51ST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

PROCEEDINGS

Seattle, Washington
August 29th -September 1st 2000

Sponsored By
Washington State Department of Transportation

51ST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 29TH -SEPTEMBER 1ST, 2000

Seattle, Washington

Welcome to the *51st Annual Highway Geology Symposium.* The local organizing committee extends a very warm welcome to the Evergreen State, and we hope you enjoy our state and particularly your stay in the Seattle area. We have planned this years meeting to provide you with valuable information concerning engineering geology and geotechnical engineering, and we believe that you will find this collection of work useful for future projects and in your present practices.

Washington State abounds with geologic wonders and scenic beauty that are one of a kind. We hope that your plans will enable you to enjoy these natural wonders.

Thank you for joining us and, once again, welcome to Washington.

Steve Lowell
Tom Badger
Symposium Co-Chairmen

51ST HIGHWAY GEOLOGY SYMPOSIUM

SEATTLE, WASHINGTON
AUGUST 29TH - SEPTEMBER 1ST, 2000

LOCAL ORGANIZING COMMITTEE

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HIGHWAY GEOLOGY SYMPOSIUM

HISTORY, ORGANIZATION, AND FUNCTION

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on March 14, 1950, in Richmond, Virginia. Attending the inaugural meeting were representatives from state highway departments (as referred to at the time) from Georgia, South Carolina, North Carolina, Virginia, Kentucky, West Virginia, Maryland, and Pennsylvania. In addition, a number of federal agencies and universities were represented. A total of nine technical papers were presented.

W.T. Parrott, an engineering geologist with the Virginia Department of Highways, chaired the first meeting. It is Mr. Parrott who originated the Highway Geology Symposium.

It was at the 1956 meeting that future HGS leader, A. C. Dodson, began his active role in participating in the Symposia. Mr. Dodson was the Chief Geologist for the North Carolina State Highway and Public Works Commission, which sponsored the 7th HGS meeting.

Since the initial meeting, 50 consecutive annual meetings have been held in 31 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona where the 13th annual HGS meeting was held. Since then, it has alternated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Annual Symposium moved to different locations as follows:

List of Highway Geology Symposium Meetings

NO.	Year	HGS Location	NO	Year	HGS Location
1st	1950	Richmond, VA	2nd	1951	Richmond, VA
3rd	1952	Lexington, VA	4th	1953	Charleston, W VA
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX

15th	1964	Rolla, MO	16th	1965	Lexington, KY
17th	1966	Ames, IA	18th	1967	Lafayette, IN
19th	1968	Morgantown, WV	20th	1969	Urbana, IL
21st	1970	Lawrence, KS	22nd	1971	Norman, OK
23rd	1972	Old Point Comfort, VA	24th	1973	Sheridan, WY
25th	1974	Raleigh, NC	26th	1975	Coeur d'Alene, ID
27th	1976	Orlando, FL	28th	1977	Rapid City, SD
29th	1978	Annapolis, MD	30th	1979	Portland, OR
31st	1980	Austin, TX	32nd	1981	Gatlinburg, TN
33rd	1982	Vail, CO	34th	1983	Stone Mountain, GA
35th	1984	San Jose, CA	36th	1985	Clarksville, IN
37th	1986	Helena, MT	38th	1987	Pittsburgh, PA
39th	1988	Park City, UT	40th	1989	Birmingham, AL
41st	1990	Albuquerque, NM	42nd	1991	Albany, NY
43rd	1992	Fayetteville, AR	44th	1993	Tampa, FL
45th	1994	Portland, OR	46th	1995	Charleston, WV
47th	1996	Cody, WY	48th	1997	Knoxville, TN
49th	1998	Prescott, AZ	50th	1999	Roanoke, VA
51st	2000	Seattle, WA			

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee composed of approximately 20-25 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contribution to the Symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers, except for the treasurer, may only succeed themselves for one additional term.

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

Meeting sites are chosen two or four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the steering committee meeting, which is held during the Annual Symposium. Upon selection, the state representative becomes the state chairman and a member protem of the Steering Committee.

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

technical session generally ends by noon on Friday. In recent years this schedule has been modified to better accommodate climate conditions and tourism benefits.

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples: in Wyoming (1973), the group viewed landslides in the Big Horn Mountains; Florida's trip (in 1976) included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee meeting in 1981 provided stops at several repaired landslides in Appalachia regions of East Tennessee.

In Utah (1988) the field trip visited sites in Provo Canyon and stopped at the famous Thistle Landslide, while in New Mexico in 1990 the emphasis was on rockfall treatment in the Rio Grand River canyon and included a stop at the Brugg Wire Rope headquarters in Santa Fe.

Mount St. Helens was visited by the field trip in 1994 when the meeting was in Portland, Oregon, while in 1995 the West Virginia meeting took us to the New River Gorge bridge that has a deck elevation 876 feet above the river.

In Cody, Wyoming the 1996 field trip visited the Chief Joseph Scenic Highway and the Beartooth uplift in northwestern Wyoming. In 1997 the Meeting in Tennessee visited the newly constructed future I-26 highway in the Blue Ridge of East Tennessee. The Arizona meeting in 1998 visited Oak Creek Canyon near Sedona and a mining ghost town at Jerrome, Arizona.

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

Banquet speakers are also a highlight and have been varied through the years

A Medallion Award was initiated in 1970 to honor those persons who have made significant contributions to the Highway Geology Symposium. The selection was and is currently made from the members of the national steering committee of the HGS.

A number of past members of the National Steering Committee have been granted Emeritus status. These individuals, usually retired, resigned from the HGS Steering Committee, or are deceased, have made significant contributions to the Highway Geology Symposium. A total of 20 persons have been granted the Emeritus status. Ten are now deceased

Several Proceedings volumes have been dedicated to past HGS Steering Committee members who have passed away. The 36th HGS Proceedings were dedicated to David L. Royster (1931-1985, Tennessee) at the Clarksville, Indiana meeting in 1985. In 1991 the Proceedings of the 42nd HGS meeting held in Albany, New York was dedicated to Burrell S. Whitlow (1929-1990, Virginia).

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HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

The Medallion Award is presented to individuals who have made significant contributions to the Highway Geology Symposium over many years. The award, instituted in 1969, is a 3.5-inch medallion mounted on a walnut shield and appropriately inscribed. The award is presented during the banquet at the annual Symposium.

Hugh Chase*	**1970**
Tom Parrott*	**1970**
Paul Price*	**1970**
K. B. Woods*	**1971**
R. J. Edmonson*	**1972**
C. S. Mullin*	**1974**
A. C. Dodson*	**1975**
Burrell Whitlow*	**1978**
Bill Sherman	**1980**
Virgil Burgat*	**1981**
Henry Mathis	**1982**
David Royster*	**1982**
Terry West	**1983**
Dave Bingham	**1984**
Vernon Bump	**1986**
C. W. "Bill" Lovell	**1989**
Joseph A. Gutierrez	**1990**
Willard McCasland	**1990**
W. A. "Bill" Wisner	**1991**
David Mitchell	**1993**
Harry Moore	**1996**
Earl Wright	**1997**
Russell Glass	**1998**

***Deceased**

HIGHWAY GEOLOGY SYMPOSIUM

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Emeritus Status is granted by the Steering Committee.

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David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
A. C. Dodson*
Walter F. Fredericksen
Brandy Gilmore
Joseph Gutierrez
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W. T. Parrot*
Paul Price*
David L. Royster*
Bill Sherman
Mitchell Smith
Sam Thornton
Berke Thompson*
Burrell Whitlow*
Earl Wright
Ed J. Zeigler

***Deceased**

Highway Geology Symposium

Future Symposium Schedule

Year	Annual	Location	Coordinators
2000	51st	Washington	Steve Lowell (360) 709-5460 Tom Badger (360) 709-5461 Washington DOT
2001	52nd	Maryland	David Martin (410) 321-3107
2002	53rd	California	John Duffy CALTRANS (805) 549-3663
2003	54th	Vermont	Tom Eliassen Vermont DOT (802) 828-2561
2004	55th	Missouri	Alan Miller Missouri DOT

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CASE HISTORIES OF SIX ROADWAY INVESTIGATIONS IN KARST COMBINING GEOPHYSICAL AND CONVENTIONAL METHODS

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ABSTRACT

Karst conditions are a prevalent problem impacting many roads, bridges and highways of the United States. This paper reviews four investigations which were made to determine the presence of karst (sinkholes and cavities) and their possible impact on existing or new construction.

These six case-histories represent projects which ranged from reconnaissance investigations, to identify possible areas of karst, to detailed investigations including follow-up drilling and remediation considerations. They also cover a wide range of scale (dimensions) of roadway problems.

The six sites utilized a wide range of data including aerial photos, surface geophysical measurements (on water as well as on land), drilling and geophysical logging. The selection of the methods for each of the unique site conditions is presented, including the parameters measured, the depth of measurements, how the data provide information on subsurface conditions along with site coverage.

In each investigation, the non-intrusive geophysical data provided an assessment of the subsurface conditions that when integrated with other data allowed the pieces of the puzzle to be brought together in an accurate conceptual model of subsurface karst conditions. This paper provides a summary of the geophysical methods used on these four projects. The results of this integrated information provided the highway design engineers with the necessary data for developing design and remediation strategies.

INTRODUCTION

The six case-histories include:

A detailed investigation of localized subsidence over 1000 feet on a main highway for Florida DOT;

A reconnaissance investigation of localized subsidence over 1600 feet of median for Tennessee DOT;

A reconnaissance of karst conditions over 4.2 miles of roadway for Virginia DOT prior to design of a critical road and utility corridor;

A reconnaissance to define areas of karst susceptibly over 7,000 feet of existing roadway for Maryland DOT to plan long-term maintenance;

A detailed investigation of a localized area of about one acre for Bureau of Prisons prior to design; and

A reconnaissance investigation for Florida DOT with follow-up detailed investigation along 8000 feet of a new bridge over water after design but prior to construction.

Technos responded to an emergency call from the Florida Department of Transportation to provide an assessment of sinkhole collapse on Highway 27 in Sebring, Florida. Subsidence (about 2 feet) had occurred over a portion of the northbound lanes and was centered around a surface water drainage system. An estimated 200 cubic yards of material had been lost in less than 1000 feet of roadway. During the previous week repairs for subsidence had been made to an area about 500 feet to the south. There were two possible causes of subsidence at this location: one is due to shallow cause of flushing of sand into a surface water drainage structure; another is due to a deep seated cause of collapse associated with karst conditions at a depth of more than 250 feet.

A section of Interstate 24 south of Nashville had recently underwent construction. After construction, a section of the median experienced localized sinkholes with subsidence cracks extending out to the edge of the shoulders within the median. The Tennessee Department of Transportation contracted with Technos, Inc. to conduct a karst investigation of the site. This investigation identified areas of deeper weathered rock associated with regional lineaments. These areas had become active due to diversion and concentration of surface water runoff.

Route 340 is a two lane road that runs along the eastern side of the South Fork of the Shenandoah River in northwestern Virginia. Four bridge replacements and associated road improvements are planned over 4.2 miles. Numerous sinkholes and caves known to be are present along the route. A reconnaissance investigation was carried out to characterize the geology and thickness of soil cover (cutter and pinnacles), and to identify possible karst conditions (highly weathered and fractured rock, sinkholes and cavities). These data were used to design a detailed drilling program.

A significant number of sinkholes had opened up along I-70 and nearby roadways south of the city of Frederick, Maryland. Extensive emergency grouting was undertaken at certain locations along I-70 in an effort to remediate the problem. A karst investigation was carried out to assess subsurface conditions along 7,000 feet of highway to locate areas in which potential collapse may be concentrated. The results clearly identified two zones which were highly prone to sinkholes development (Benson, et al, 1998-1999).

A major Federal Prison facility was to be constructed in an area of southwestern Virginia which a known area of extensive karst development. The main access road to the site along with main utilities for the site cross a "saddle" area between two major coalescing sinkholes. An investigation of subsurface conditions was carried out to evaluate subsurface conditions and to obtain data for site stabilization and construction (Daniel, et al., 1999).

A high four-lane bridge was planned to replace an existing two-lane draw bridge over Jewfish Creek and Lake Surprise, just north of Key Largo, Florida. Lake Surprise is a shallow circular lake with US-1 passing though its center for about 3,500 feet. Drilled shaft foundations for the bridge will be placed parallel to US-1 over Jewfish Creek and through the center of Lake Surprise. There were a few subtle indications of potential karst in the area. An investigation was carried out to determine if karst conditions existed along the proposed route. The results identified a zone of paleokarst about 1800 feet long where piles for the high portion of the bridge would be placed. These data provided the basis for a cost effective safe design (Benson, et al, 1995a and 1995b).

While all six of the sites were in karst environments: three were associated with present sinkhole activity, two were in areas with historic karst but no recent activity, and one was in an area of no known karst activity. Details of three of the investigations including the geologic setting, the strategy used and the results, are provided in the reference papers.

Table I summarizes the nature and scale of each project. The size of the sites ranged from a few acres to 4.2 miles of roadway.

Table I

Client	Reconnaissance	Detail	Area/Length	Condition
Florida DOT		X	1000 feet	Sinkhole activity
Tennessee DOT	X		1600 feet	Sinkhole activity
Virginia DOT	X		4.2 miles	Historic karst present
Maryland DOT	X	X	7,000 feet	Sinkhole activity
Virginia Prison		X	few acres	Historic karst activity
Florida DOT	X	X	8,000 feet	Unknown karst activity

Because of the range of sizes, the type of potential karst problem, the need for reconnaissance or detailed data, and cost considerations; the strategies and selection of methodology become critical factors in such investigations. This paper reviews the strategy used in selecting the methods of investigation.

SELECTING APPROPRIATE METHODS AND STRATEGIES

There are many remote sensing, non and minimally invasive methods, as well as invasive methods of investigation which are available. A suitable combination of methods must be selected to provide the appropriate data.

Geophysical methods measure the physical, electrical, or chemical property of the soil, rock, and pore fluids. To detect an anomaly, a soil to rock contact, the presence of inorganic contaminants, or a buried drum, there must be a contrast in the property being measured (for example, the target to be detected or geologic feature to be defined must have properties significantly different from "background" conditions). For example, the interface between fresh water and saltwater can be detected by the differences in electrical properties. The contact between soil and unweathered bedrock can be detected by the differences in acoustical velocity. In some cases, the differences in physical properties may be too small for detection by geophysical methods.

In each case, remote sensing and geophysical measurements played an important roll in reaching an accurate and complete interpretation of subsurface conditions of these four projects. Some of the key issues which impact the selection and use of methods are; the scale of the problem as well as the scale of measurements, data density, and the sequence of work. We will look at these issues and see how they affect our site characterization efforts.

THE ISSUE OF SCALE

The scale of the problem refers to the size of the area to be surveyed as well as the size of the expected anomalous conditions. For example continuous coverage can be economically obtained over smaller areas of a few

acres or a few miles while only reconnaissance coverage can be made over hundreds of acres and hundreds of miles.

The scale of our measurements refers to the sample volume of a measurement. A small sample volume provides grater detail while a large sample volume yields less detail but greater coverage to detect larger features. For example, a hand-held core sample is less than a cubic foot while and electromagnetic measurement (EM31) integrates data over a thousand cubic feet.

DATA DENSITY

Our data density must be sufficient to adequately define conditions. All too often, adequate spatial data is decreased for the sake of cost, resulting in a survey that can miss anomalous conditions leading to errors and incomplete interpretations of subsurface conditions. Temporal adequacy is concerned with sufficient data points over time as well as gaps in the data.

If the expected anomaly is large, it may be detected by certain methods using low density coverage. On the other hand, if it is small, we will need a method with higher resolution and greater data density to identify it.

A highly weathered (wide and deep) fracture zone will easily be detected by less dense surface geophysical measurements while a small joints in rock will require densely spaced downhole geophysical measurements.

SEQUENCE OF WORK

Since the basic hydrogeologic conditions are the foundation upon which our understanding of site conditions are based, we should have the basics in place before attempting to acquire more complicated data. The site characterization process should usually start with the regional setting, working toward the local setting, and then to the very local boring, sampling, and testing program.

Early in any site characterization process attention should be focused upon geologic and hydrologic conditions using non-invasive and minimally invasive methods. Then, as the site characterization proceeds, piezometer, monitoring wells, and detailed tests can then be accurately located so they are truly representative of site conditions.

SUMMARY OF GEOPHYSICAL METHODS USED ON SIX CASE HISTORIES

Tables II, trough VII summarize the non intrusive geophysical methods used along with the results of the measurements.

Table II – A detailed investigation of 1000 feet, Florida DOT

Method	Measured Parameter	Station Spacing	Provides
EM31	Electrical conductivity to a depth of 20 feet	Continuous	Identified variations in depth of rock and weathered zones
Microgravity	Density to a depth of more than 100 feet	20 to 40 foot	Determine if zones of low density due to subsidence exist
Resistivity	Electrical resistivity to a depth of 100 feet	20-foot electrode spacing	Detailed characterization of fractured and weathered zones
Ground penetrating radar	Dielectric constant to a depth of 10 to 15 feet	Continuous	Indications of soil piping

Table III – A reconnaissance investigation for Tennessee DOT

Method	Measured Parameter	Station Spacing	Provides
2D Resistivity	Resistivity to 80 feet	20-foot electrode spacing	Map top of rock, identify highly weathered zones
Ground Penetrating Radar	Dielectric constant, less than 10 feet	Continuous	
EM31	Electrical conductivity to a depth of 20 feet	Continuous	Identify variations in top of rock

Table IV – A reconnaissance investigation of 4.2 miles, Virginia DOT

Method	Measured Parameter	Station Spacing	Provides
EM31	Electrical conductivity to a depth of 20 feet	Continuous	Identified variations in depth of rock and weathered zones
EM34	Electrical conductivity to a depth of 50 feet	Continuous	Identified variations in depth of rock and weathered zones
Resistivity	Electrical resistivity to a depth of 100 feet	20-foot electrode spacing	Detailed characterization of fractured and weathered zones

Table V - Maryland DOT

Method	Measured Parameter	Station Spacing	Provides

EM31	Electrical conductivity to a depth of 20 feet	Continuous	Identify variations in depth of rock
EM34	Electrical conductivity to a depth of 50 feet	Continuous	Identify variations in depth of rock
Microgravity	Density to a depth of more than 100 feet	15-foot	Identify zones of low density due to dissolution of limestone

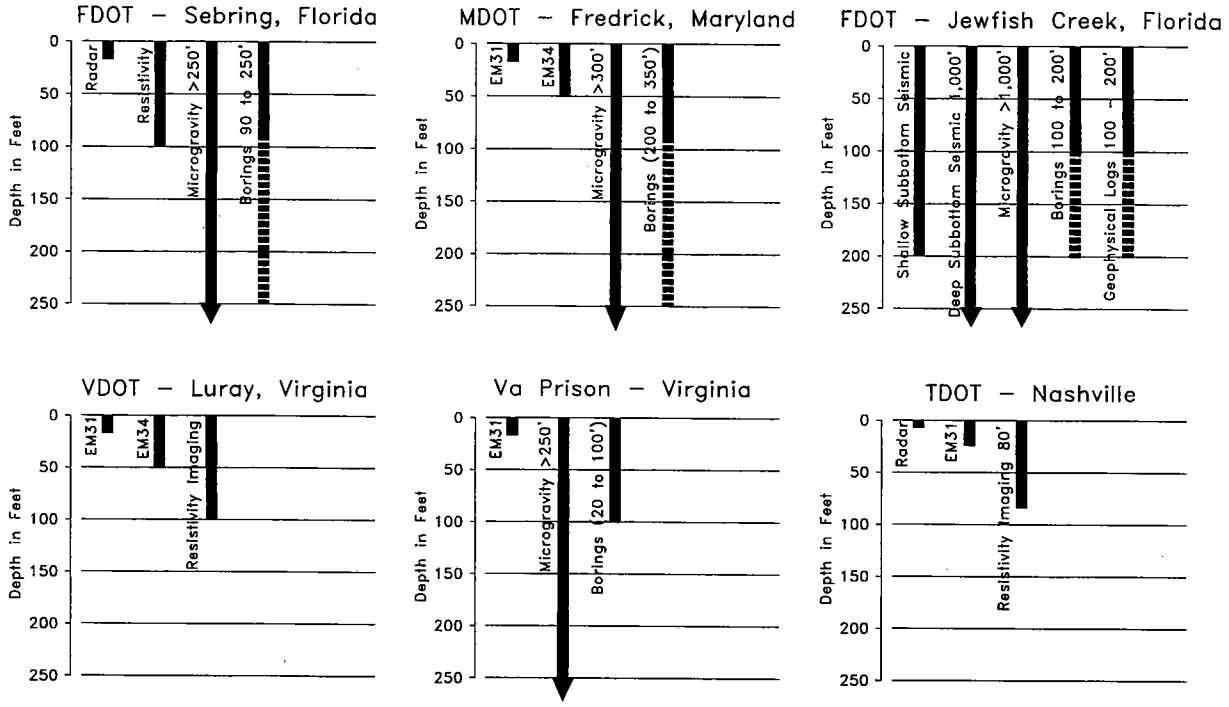
Table VI – Virginia Prison

Method	Parameter Measured	Station Spacing	Provides
EM31	Electrical conductivity to a depth of 20 feet	Continuous	Identify variations in depth of rock and weathered zones
Microgravity	Density to a depth of more than 100 feet	25- foot	Identify low density zones due to deeper rock and possible karst zones

Table VII - Florida DOT

Method	Measured Parameter	Station Spacing	Provides
Subbottom	Acoustic Impedance (velocity and density) to a depth of 200 feet	Continuous	2D stratigraphic cross-sections
Microgravity	Density to a depth of more than 100 feet	25 foot	Identify low density zones (possible karst zones)
Deep Subbottom	Acoustic Impedance (velocity and density) to a depth of 1000 feet	Continuous	Identify low density zones (possible karst zones)
Logging	Natural gamma, density and porosity over length of the borehole	Continuous	Provides details of strata, presence of voids and low density zones

Figures 1 shows the methods used and their relative depth of investigation on each of the projects.



In five cases, some measurements were 20 feet deep or less. In many cases, these reconnaissance measurements are not deep enough to detect the origin of the subsidence problem. However, most subsidence activity is associated with changes in the shallow near surface conditions, which are often readily detected (see papers by Benson and LaFountain (1984) and Benson and Yuhr (1987) which discus near-surface indicators).

Note that EM measurements are commonly used because they are continuous rapid and easily applied to many situations where data in the upper 20 to 50 feet may be important. Note that 2D resistivity imaging and microgravity are commonly used to obtain deeper data. These measurements are limited in resolution in part by station spacing and are inherently slower than continuous measurements such as EM.

CONCLUSIONS

These six case histories show that a range of methods must be considered to provide appropriate data to resolve specific project needs such as reconnaissance, or detailed investigations on land or on water. Many remote sensing, non and minimally invasive methods, as well as invasive methods of investigation can be applied to resolve such geotechnical problems. In each case, remote sensing and geophysical measurements play an important roll in reaching an accurate and complete interpretation of subsurface conditions. Selection of appropriate methods should be done by experienced hands on professionals. Although there are many method selection guidelines available such as ASTM D6429 and ASTM D5753-95, and even an expert system (Olhoeft 1988) the appropriate combination of methods can only be arrived at by experienced hands on professionals.

Two of the key issues which impacts the selection and use of methods are; the scale of the problem and measurements, as well as data density. Other factors include; the sequence of work and the use of multiple

methods. The sequence of work should start with the regional setting, working toward the local setting, and then to the very local boring, sampling, and testing program. Each step of the way should be focused upon obtaining basic hydrogeologic data. Multiple sources of data are used to confirm site specific conditions. When measurements by different methods agree, our interpretations will have a higher level of confidence.

In addition on site observations by experienced hands on professionals are essential to resolving complex problems such as cavity and subsidence problems.

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Roadway Planning and Development in Active Land Subsidence Areas near Phoenix, Arizona

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Introduction

Land subsidence has developed and continues to impact several areas within the United States from Savannah, Georgia to Sacramento, California (Figure 1). Subsidence caused by fluid withdrawal from groundwater aquifers and oil reservoirs ranges from less than three feet to more than twenty-eight feet with some occurring within heavily populated, rapidly developing urban areas. The total estimated loss from subsidence in the United States has been estimated to be many billions of dollars (Johnson, 1995).

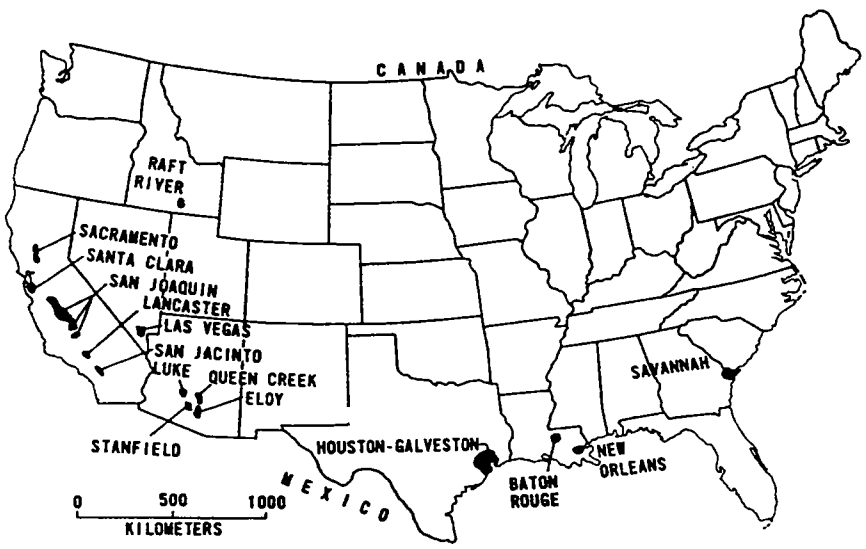

Figure 1: Major land subsidence areas within the United States (from Johnson, 1998).

Land subsidence, caused by substantial over-drafting or removal of groundwater from deep alluvium-filled basin aquifers, has impacted central Arizona for more than 50 years. Subsidence and related geologic hazards generates a variety of problems for agricultural and urban areas. Some of these problems include: roadway grade settlement or collapse; accelerated pavement deterioration; flow reversals in flood control and irrigation canals and in water and sewer pipelines; loss of foundation support; excessive erosion; and slope gradient changes.

Prior to the massive use of groundwater resources within the Phoenix metropolitan area, the water table was higher and hydrogeologic conditions were in equilibrium. Major agricultural development and associated groundwater use, beginning in the early 1930's and accelerating after World War II, caused water levels within the basin aquifers to be lowered, dewatering the basin fill sediments. In the arid southwest, the water in the aquifer is used up faster than it can be naturally recharged causing a net groundwater level decline. As a result, the weight of the soil column gradually increases as the buoyant effects and aquifer pressures, induced by the water acting on the soil column, are decreased. These conditions cause increased loading stresses to consolidate portions of the thick compressible sediment which result in the lowering (subsidence) of the land surface over a large area. Since the 1930's, measured land surface subsidence in some central Arizona basins, where excessive groundwater use has occurred, has exceeded 15 to 25 feet.

In land subsidence area where tensile stresses in the soil column that exceed the strength of the soil, the ground ruptures to create a fracture that propagates to the surface to form an earth fissure. Fissures commonly form near a buried bedrock pediment edge or buried bedrock highs where ground water levels have dropped anywhere from 150 feet to more than 500 from their original pre-pumping levels.

The Phoenix metropolitan area is one of the fastest growing areas in the United States. Land development to support this growth is beginning to encroach into agricultural lands rife with land subsidence and earth fissure problems. Many land developers, as part of their "due diligence efforts," are now requiring subsidence/earth fissure investigations for their development master planning. As part of this growth, State and county public works/highway departments need to factor land subsidence and earth fissures into road and highway design concepts.

In 1999, the Pinal County Department of Public Works, initiated a roadway realignment study to reroute a section of the County route, Thompson Road, through a proposed master planned community and replace a Y- intersection with a safer T-intersection with Hunt Highway at the north boundary of the community. During the survey of the initial right-of-way transferred to the County by the land developer, the survey crew noticed unusual and aligned depressions parallel to the centerline. Geological Consultants was contracted by the County to examine the anomalous features and assess their potential impact on the roadway. As it turned out, the original route proposed traversed through a zone of a mature, well developed earth fissured. A subsidence/earth fissure investigation, conducted by geologists experienced in dealing with similar land subsidence and earth fissure projects, rapidly and economically evaluated the alignment geologic hazards and made recommendations to deal with earth fissure prone areas.

This paper will describe the subsidence earth fissure phenomena in the study area, the field investigation and the findings and recommendations that resulted in roadway alignment modifications that avoided earth fissure prone areas and a savings of over one-half million dollars in redesign fees for Pinal County. By employing geologists experienced in dealing with a known geologic hazard in the initial planning phases of development, money in construction and future maintenance was saved.

Location & Physical Features

The Thompson Road alignment lies within northern Pinal County, south of the Town of Queen Creek about 34 miles southeast of Phoenix, Arizona (Figure 2). Regionally, the project area is situated within and near the south-central margin of the East Salt River Valley basin. This portion of the basin is bounded on the west and south by the Sacaton and Santan Mountains. Surface runoff from the site flows to the north toward Queen Creek, which flows southwest to the Gila River. The dominant topographic feature is Goldmine Mountain southwest of the alignment and rises to an elevation of about 2,375 feet. Maximum relief along the alignment is about one hundred feet.

Local Geology

The site is located within the Sonoran Desert region in the north-central portion of the Basin and Range Physiographic Province in Arizona. The Basin and Range Province is characterized by

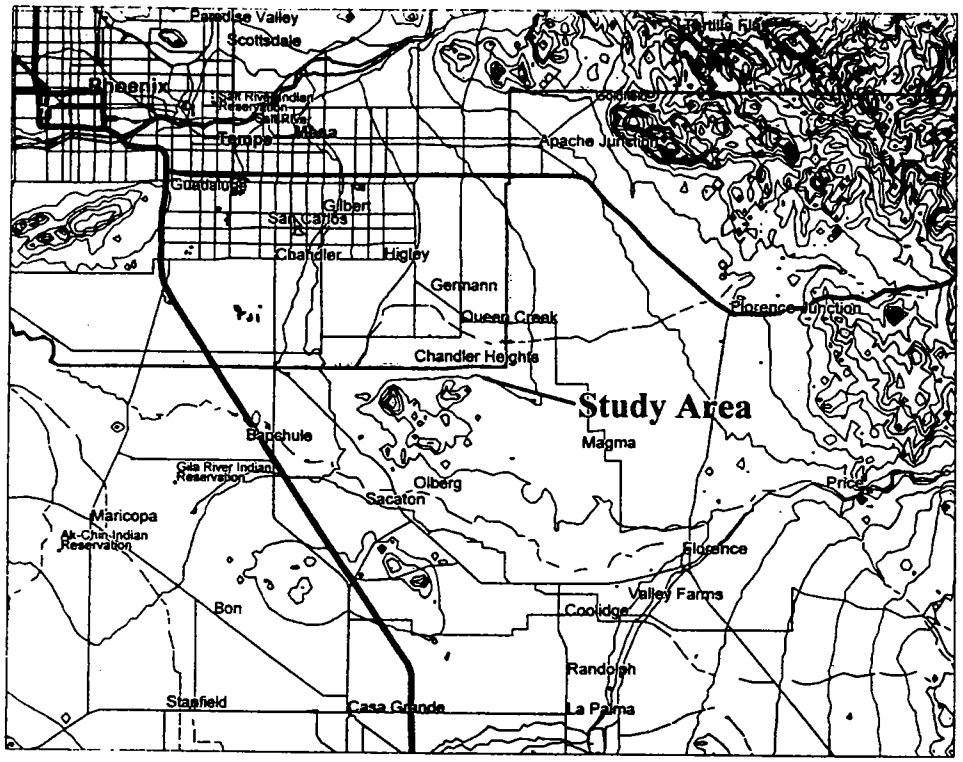

***Figure 2:** Study area location map, not to scale (from Topo USA 2.0 Copyright© 1999 DeLorme).*

northwest, north, and northeast trending mountains that rise abruptly from broad, elongated, deep sediment-filled valleys produced by block faulting and folding. The mountains and hills west of the study area, the Santan Mountains, are composed predominately of Pre-Cambrian age (570 million years ago (mya)) metamorphic schist and igneous granodiorite bedrock, intruded by younger dikes (Ferguson, 1996). The bedrock is locally overlain by Quaternary age (younger than 1.6 mya) alluvium or basin fill. The basin fill commonly makes up the principle groundwater aquifer of the region.

Structurally, the region has been uplifted to its present position by episodes of mountain/basin bounding fault movements (Cooley, 1977). The tectonic episodes and deformation, evident in the orientation of foliation planes and joint dip set discontinuities exposed in the bedrock terrain, have provided the mechanics necessary for deep intermontane basins which were subsequently filled with sediment. The alluvial deposits are expected to be a few feet near the mountains due to the presence of a shallow buried bedrock pediment. Beyond the pediment edge (to the north), the basin fill thickens rapidly to several 1,000's of feet reaching to more than 10,000 feet southeast of Gilbert, Arizona (ADWR, VII, 1994) (Figure 3).

The basin filled strata can be grossly subdivided into three zones: an upper sand and gravel unit that ranges in thickness from nil to more than 300 feet, a middle silt and clay unit that ranges in thickness from less than 100 feet to more than 1,800 feet, and a lower conglomerate unit that ranges in thickness from less than 100 feet to more than 9,000 feet (ADWR VII, 1994). Due to the close proximity of the Santan Mountains, the middle fine-grained unit may not underlie the site area.

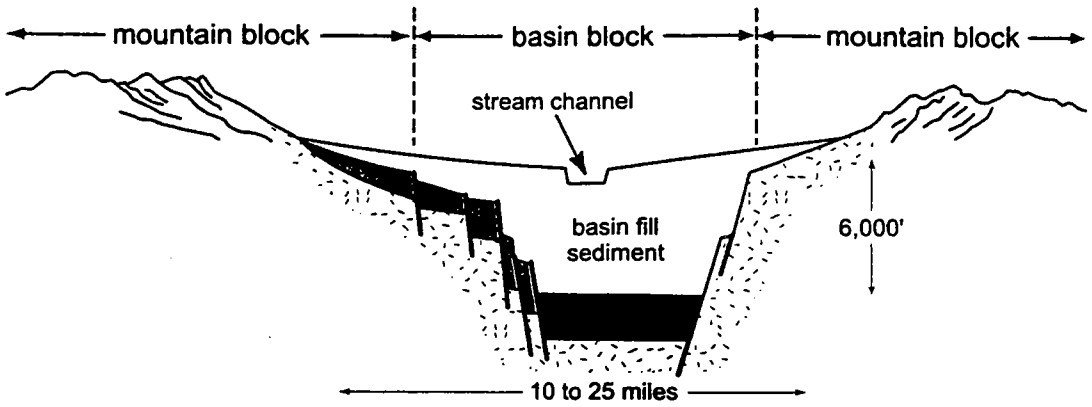

Figure 3: Generalized structural cross section of the East Salt River Valley (from Arizona Geology, Fall 1999).

Groundwater Conditions

The major human-induced factor contributing to subsidence is the large scale removal of groundwater. Nearly all of the populated southern Arizona basins from Phoenix to Tucson have experienced at least a 100+ foot drop in groundwater level, and an area surrounding the town of Stanfield, Arizona has dropped more than 500 feet. The groundwater level in the vicinity of the study area has dropped from 300 feet to 500 feet (Schumann 1986).

Groundwater Use Near Thompson Road

The Thompson Road alignment is within the East Salt River Valley Sub-Basin (ESRV), one of seven groundwater sub-basins within the Phoenix Active Management Area (AMA) as defined by the Arizona Department of Water Resources (ADWR, 1994). Groundwater pumpage estimates prior to 1984 are not readily available for the ESRV, but are available for the entire Salt River Valley (SRV). In 1915, 15,000 acre-feet of groundwater were pumped from wells in the SRV. By 1942, the annual volume of groundwater withdrawn had increased to approximately 1,000,000 acre-feet. Groundwater pumpage peaked in the 1950's with approximately 2,300,000 acre-feet per year being used. By 1992, annual usage in the SRV had decreased to 1,100,000 acre-feet. Approximately 304,900 acre-feet of groundwater were pumped from the ESRV in 1990 (ADWR, VII, 1994).

Water levels in the ESRV steadily declined from 1923 to 1976 to more than 400' below original levels near the Santan Mountain (Figure 4). Water levels rose over most of the ESRV from 1976 to 1983 due to heavy runoff and abundant surface water supply. Changes in water levels during this time ranged from a decline of 175 feet in the cone of depression near Queen Creek to a rise of 93 feet north of Queen Creek (ADWR, VII, 1994).

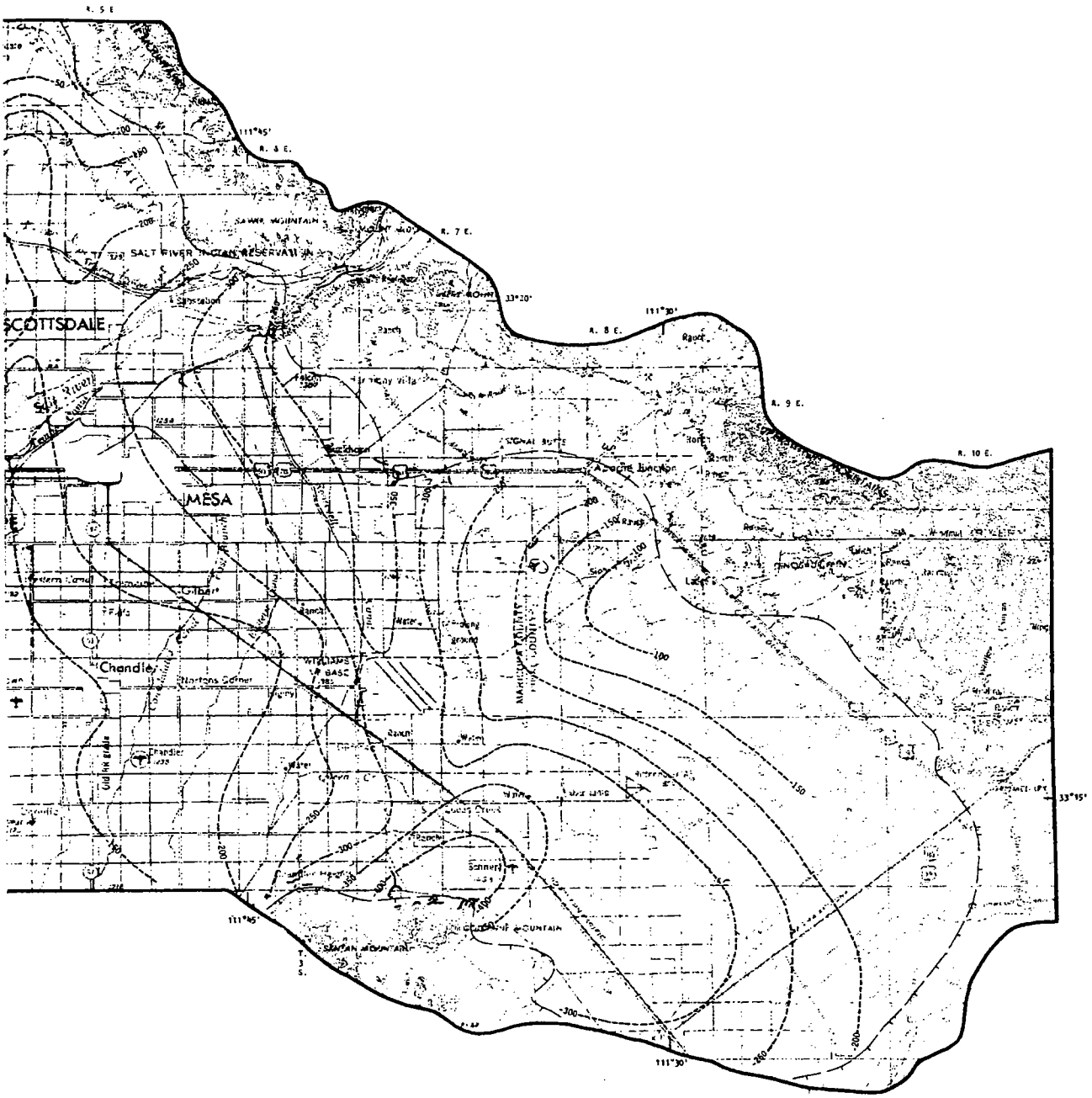

Figure 4: *Map showing groundwater decline, in feet, throughout the East Salt River Valley (from Laney, et al, 1978).*

Land Subsidence in Arizona

Land subsidence is known to occur in alluvium filled valleys of Arizona where agricultural activities and urban development have resulted in substantial over-drafting or removal of groundwater from thick basin aquifers (Figure 5). The magnitude of subsidence is directly related to the subsurface geology, the thickness and compressibility of the alluvial sediments deposited in the valleys, and the

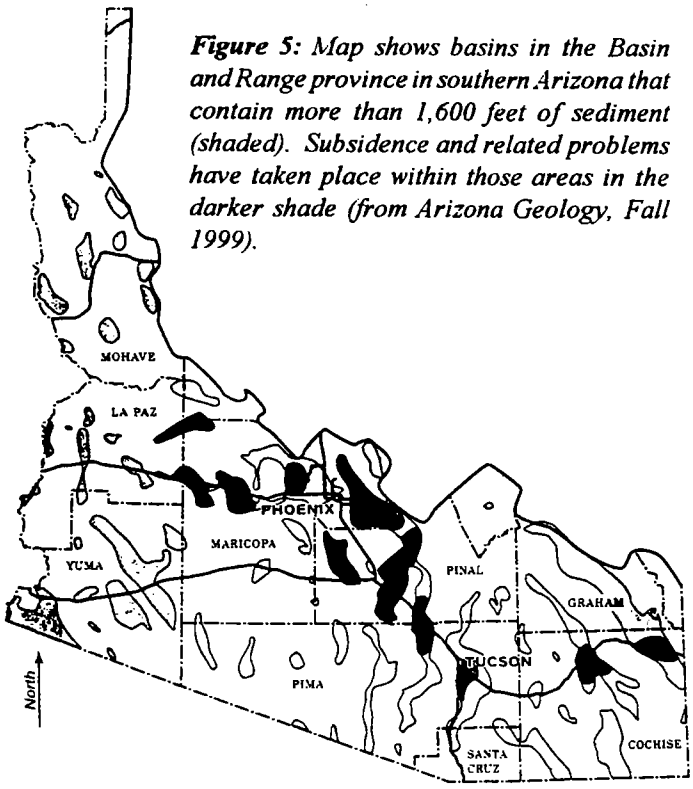

Figure 5: *Map shows basins in the Basin and Range province in southern Arizona that contain more than 1,600 feet of sediment (shaded). Subsidence and related problems have taken place within those areas in the darker shade (from Arizona Geology, Fall 1999).*

net groundwater decline. According to Bouwer (1977), ground subsidence rate range from about one-hundredth to one-half feet per 10-foot drop in groundwater level, depending on formation thickness and compressibility.

Prior to the utilization of groundwater resources within the Phoenix area, the water table was higher and hydrogeologic conditions were in equilibrium. When pumping was initiated, water levels within the aquifer began to be lowered and the basin fill sediments were dewatered. In the arid southwest, the water in the aquifer may be removed by pumping faster than it can be naturally replenished causing a net water table decline. As a result, the weight of the soil column is gradually increased as the buoyant effects and aquifer pressures induced by the water acting on the soil column are decreased. This condition causes increased loading stresses to consolidate portions of the thick compressible sediments which result in the lowering (subsidence) of the land surface over a large area.

Land subsidence was first documented in Arizona in 1934 following the releveling of first-order survey lines by the Coast and Geodetic Survey (now the National Geodetic Survey (NGS)). Subsequent levels by the NGS, the U.S. Geological Survey, the Bureau of Reclamation, and the Arizona Department of Transportation has documented substantial ground surface subsidence in south central Arizona including the Salt River Valley, the Queen Creek - Apache Junction area, and the Eloy - Casa Grande - Stanfield area as overdrafting of the aquifer continues (Figure 6). Tensile stresses induced by the subsidence continue to increase until the ground breaks to form earth fissures.

Subsidence and earth fissures in urban areas can cause a variety of problems. Structures built across fissures may be damaged, street cracks, flow in gravity water and sewer lines can be reversed, and differential subsidence (although rare) can rupture buried utilities (Winikka, 1984). However, design measures can be implemented to mitigate the effects of ground subsidence. Some of these measures can include special foundation design, additional structural reinforcement, oversized pipes, surface drainage controls and bridging the subsidence feature.

Earth Fissures

Fissures occur in unconsolidated sediments, typically near the margins of alluvial valleys or near the bedrock pediment edge where ground water levels have dropped from about 200 feet to 500 feet below ground surface (Schumann, 1986).

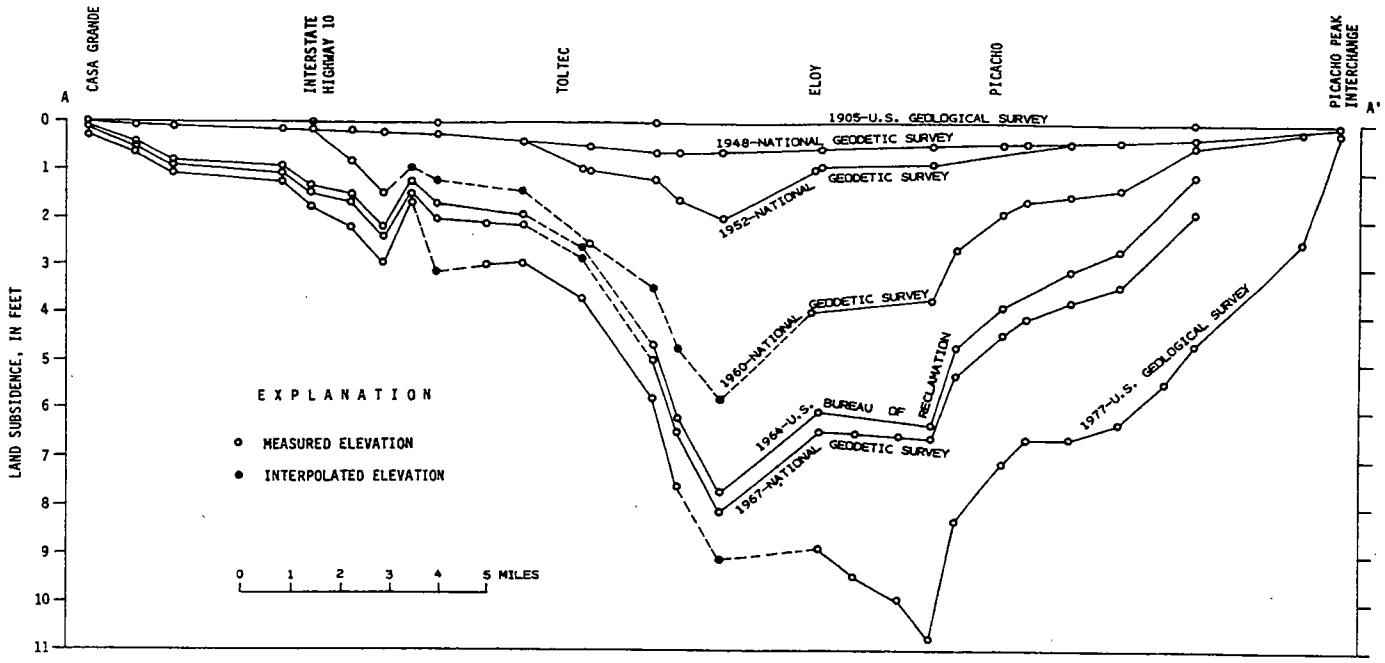

Figure 6: *Land subsidence between Casa Grande and Picacho Peak interchange along I-10 in Arizona, 1905 to 1977 (from Laney et al, 1977).*

Fissures are initiated underground when tensile stresses exceed the strength of the soils. The fissures then propagate upwards to intersect the ground surface. Early signs of earth fissuring are small en echelon hairline cracks and irregular spaced depressions. As fissures develop, the cracks grow in length to form a surface expression of the feature ranging a few feet to several thousand feet in length and from 1 foot to more than 10 feet wide. The older fissures are curvilinear and often have vegetation growing in them (Figure 7). Other physical features associated with fissures are slump-related escarpments from one inch to a few inches in height and occasionally differential displacements of one to more than four feet (Figure 8). In many instances, the surface erosion and drainage patterns associated with the fissures do not conform to the areas' local drainage pattern.

Figure 7: *Aerial photo showing fissure curvilinear surface expression and thick vegetation growing in an earth fissure located near Casa Grande, Arizona. Note stream channel truncated by the earth fissure. (Kenney Aerial Mapping, 1999)*

Figure 8: *Differential displacement along an earth fissure located 400 feet north of the Thompson Road - Hunt Highway intersection. Photo by C.Spencer.*

Subsidence in Thompson Road Vicinity

Measured subsidence in the East Salt River Valley area was first recorded in the 1960's and 1970's by comparing elevations of fixed benchmarks placed directly on bedrock, to benchmarks placed in alluvial valley fill (Péwé, 1982). No formal ground subsidence studies had been completed within or adjacent to the Thompson Road alignment until a preliminary assessment was conducted as part of a nearby master planned community (Geological Consultants Inc., 1999). Schumann (1974) estimated that subsidence for the area was between one and 3 feet as of that year. As of 1967, 3.9 feet of subsidence had been recorded in the town of Queen Creek (Strange, 1983). Assuming the rate of subsidence in the area has continued at the 1967 rate of one-tenth foot per year (Bouwer, 1977), the total subsidence in the area through 1999 is estimated to be about seven feet. The rate of subsidence would be less near the mountain front. Numerous earth fissures have also been mapped on the northwest portion of the property, indicating ground subsidence has continued in the area.

The Thompson Road alignment project is located within an area of active earth fissuring and ground subsidence as evidenced by studies conducted by the U.S. Geological Survey, the Arizona Geological Survey, and private consultants. At least 23 previously documented earth fissures (Harris, 1994; Geological Consultants, 1999) are located north and south of Hunt Highway, many of which parallel the proposed Thompson Road alignment.

Geological Reconnaissance of Proposed Thompson Road Alignment

In 1999, Pinal County entered into negotiations with the developer of a new master planned community regarding the County's acquisition of a new right-of-way to provide a dedicated arterial

road way through the new community. The initial Thompson Road alignment was to be given to the County by the land developer with the condition that the County would design and build the new roadway. In response to this proposal, Pinal County Department of Public Works, initiated a roadway design study to re-route a section of the county road through the proposed master planned community, to provide better access to a nearby regional park, and replace an existing Y- intersection with a safer T-intersection with Hunt Highway at the north boundary of the community. During the design field investigation, a survey crew noticed, and reported to the County, that unusual and aligned depressions were present along and parallel to the centerline. Geological Consultants was subsequently contracted by the County to examine the anomalous features and assess their potential impact on the roadway design and operation.

A geological reconnaissance of the proposed roadway alignment was conducted to examine subsidence-related earth fissures known to exist along portions of the subject realignment. Some earth fissures observed were segments of continuous fissures several hundred to more than a thousand feet in length (Figure 9). At the surface, the fissures have experienced the effects of erosion causing gullies several feet wide and more than 15 feet deep to form along their trace (Figure 10). The earth fissures extend from the ground surface probably to bedrock expected at depths of a few hundred feet.

Some of the observations made and discussions, held during the initial site reconnaissance with the County and their design engineering consultant, are summarized as follows:

- A series of prominent earth fissures was easily identified along the proposed new alignment. The centerline paralleled a group of at least three major earth fissures. Along some sections of the alignment, the proposed roadway centerline is directly on the trace of an earth fissure (Figure 9). Most of the new north roadway section right-of-way was located within an earth fissure zone about 100 feet or more wide.

- Various mitigation alternatives to deal with the earth fissures were discussed with the County:
  1) Over excavate the roadway section to a depth of about 10 feet and a width of about 100 feet along the affected section of the alignment.
  2) Place a 4-inch thick concrete slab, reinforced with welded wire mesh, over the bottom of the over excavated area. A synthetic geotextile filter fabric with an engineered drain fill blanket might be used instead of the rigid concrete cap.
  3) Backfill the over excavated area using appropriate lift thicknesses and compact the backfill soil to meet structural backfill specifications.
  4) Design positive drainage/runoff controls to reduce the potential for water to collect along the alignment. Lining of roadway ditches may be necessary.
  5) Design sufficient freeboard to the drainage components to compensate for estimated future potential ground subsidence.

- Because of the increased construction costs to deal with the earth fissure, Geological Consultants suggested that the County should consider renegotiating with the land developer to shift the north section of the proposed realignment to avoid as many

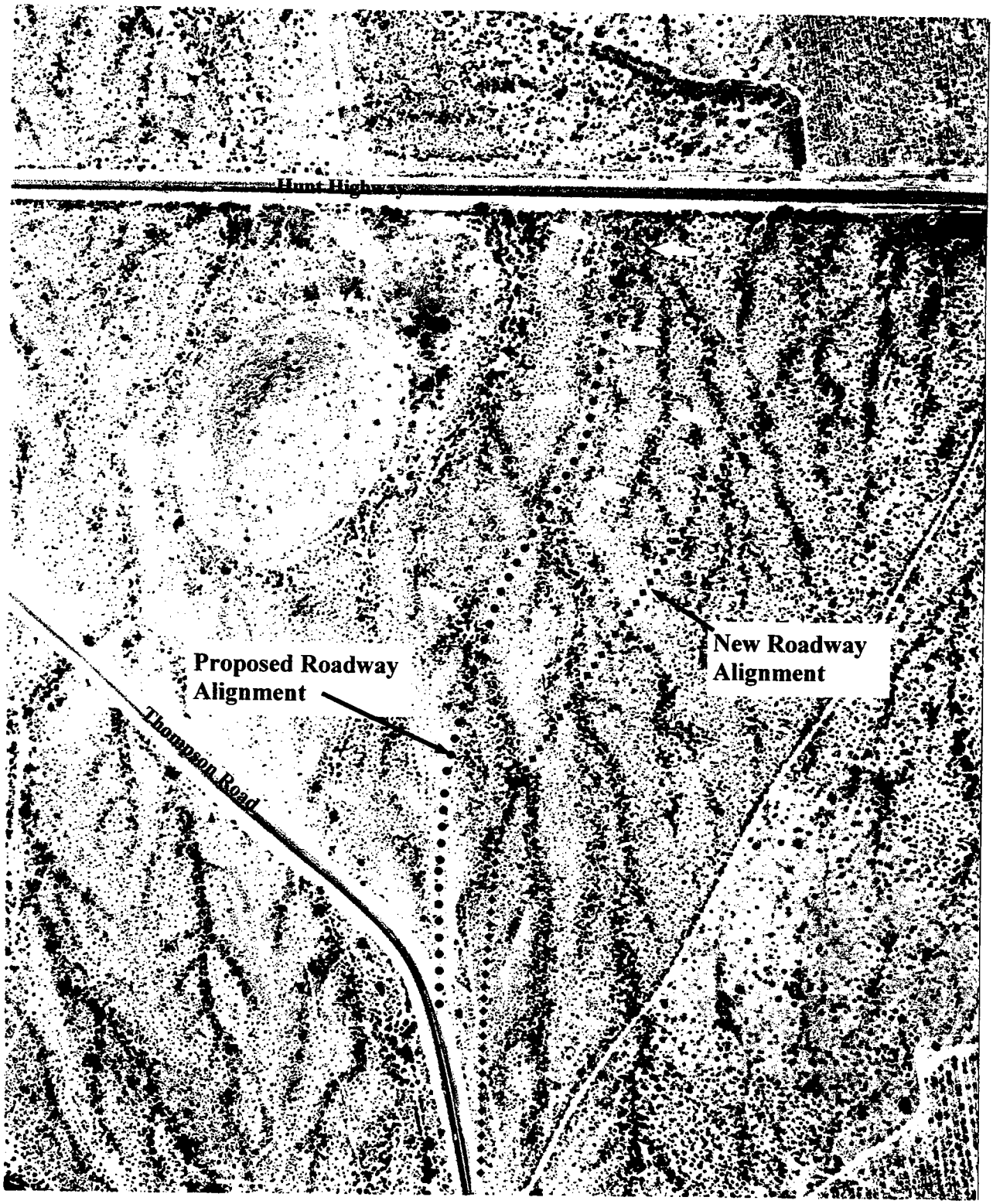

Figure 9: *Photo showing approximate location of the original proposed Thompson road alignment and the new roadway alignment, away from the earth fissure zone. White arrows point earth fissures.*

Figure 10: *Earth fissure gully located along the initial proposed roadway alignment. Photo C. Spencer*

of the earth fissures as possible. The new alignment would place the roadway alignment on more stable ground, reduce the size of the fissure crossings, and orient the improvements to cross earth fissures at nearly right angles.

Pinal County Department of Public Works pursued discussions with the land developer to consider another alignment. The County was successful in their efforts and acquired a new alignment east of the former alignment (Figure 9). A second geological reconnaissance of the new alignment was conducted to identify any actual or suspect earth fissure features that might be present. The field reconnaissance included a visual examination of the alignment within the proposed right-of-way. In some areas, examinations were made one hundred to two hundred feet outside the right-of-way limits where known earth fissures project toward the right-of-way: During the reconnaissance three "suspect" linear features, possibly representing earth fissure traces, were identified within the right-of-way and three known and previously mapped earth fissures were identified projecting toward the proposed alignment.

One of the known earth fissures (projecting between Station 199+00 and 200+50) showed evidence of very recent activity (Figure 11). The new activity consisted of a new linear ground crack open at the ground surface about 0.05 inches to 0.075 inches wide. The new ground crack projects southward toward the alignment from the south end of an established fissure. Earth fissures found near the new alignment are depicted on Figure 12. Known and "suspect" earth fissures observed during the field reconnaissance of the new alignment are summarized in Table 1.

Table 1
Recommended Subsurface Exploration Sites
Know and "Suspect" Earth Fissures Along New Thompson Road Alignment

No.	Fissure Type	Centerline Station	Offset From Centerline	Trend to Centerline	Remarks
1	Suspect	197+50	Crosses centerline	S 02° E	Linear depression 4 feet wide, 1-foot deep; aligned vegetation along trend.
2	Known	200+00	Projection crosses centerline	S 08° E	Aligned series of shallow depressions ½ to 1-foot deep; aligned vegetation along trend.
3	Suspect	204+95	40' West	S 05° E	New extension growth of previously identified fissure; hairline crack to 0.05 inches wide with some separation to 0.075 inches; some small pipes aligned along trend. No vegetation growth along trend due to recent activity.
4	Suspect	207+50	50' East	S 45° E	Prominent linear feature with depression or shallow pot holes; aligned vegetation along trend.
5	Known	177+50	Projection crosses centerline	N 08° W	Known fissure projects toward alignment; fissure well formed showing signs of recent activity with hairline crack extension west from previously identified fissure
6	Known	184+50	Projection crosses centerline	N 65° W	Known fissure projects toward alignment; fissure well formed showing signs of recent activity with hairline crack extension northwest from previously identified fissure

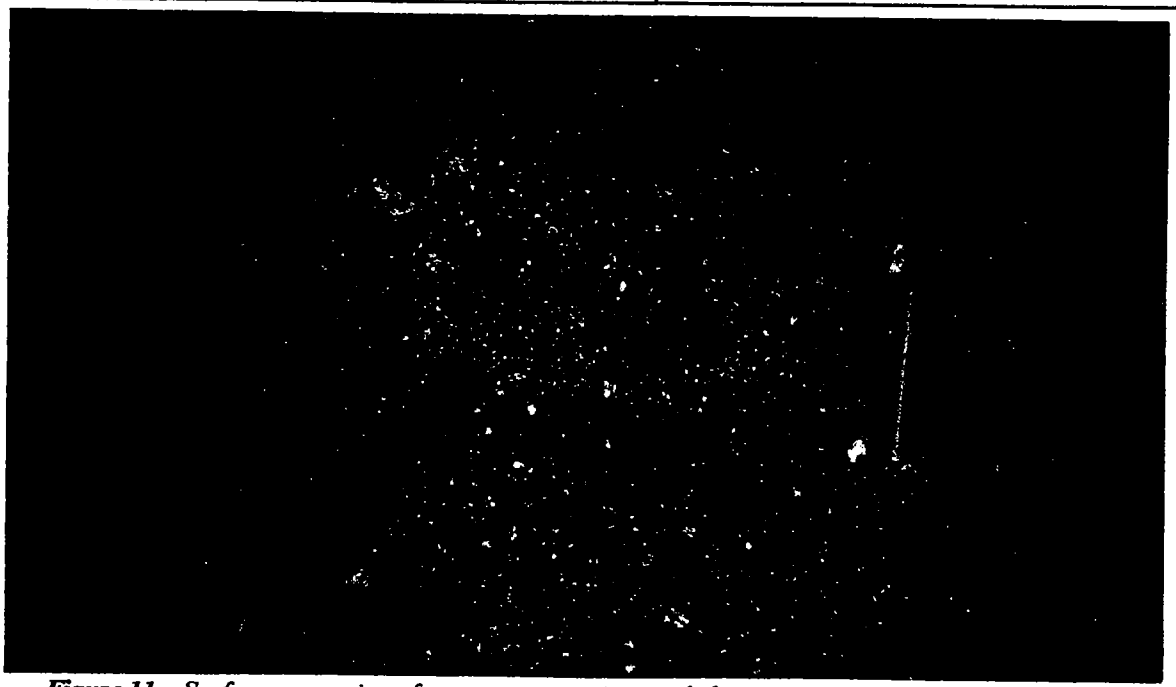

Figure 11: *Surface expression of a very young active earth fissure segment projecting within 200 feet of the new Thompson Road alignment. This fissure segment is a continuation of the relatively mature fissure depicted in Figure 10.*

Subsurface Exploration of Fissures

Following the second geological reconnaissance, subsurface exploration of known and suspect earth fissures was conducted to evaluate their potential impact on the new roadway alignment (Figure 12). Two types of earth fissure features were investigated, known or previously mapped earth fissures and features suspected for being earth fissures:

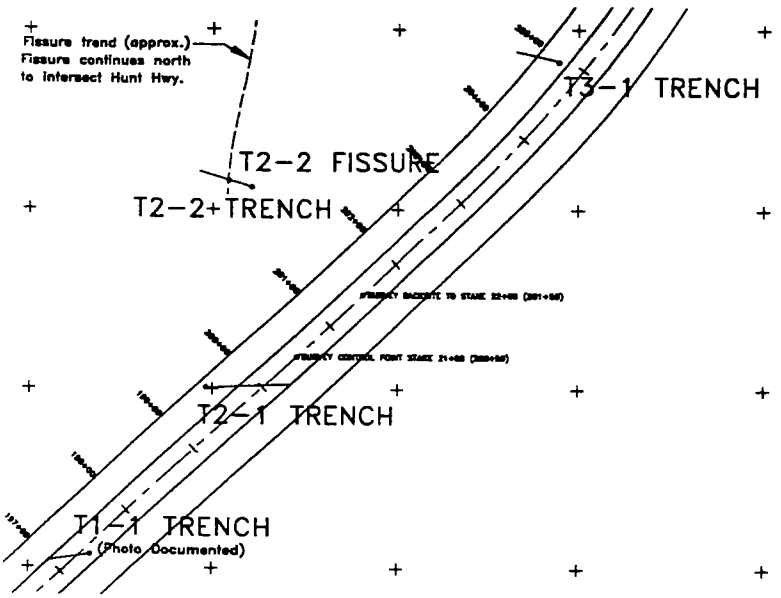

Figure 12: Portion of new Thompson Road earth fissure subsurface exploration location map depicting trench locations and fissure.

1) Where known or previously mapped earth fissures were identified, an initial exploration trench was dug across the earth fissure at its closest approach to the proposed alignment. Once the subsurface conditions of the fissure were exposed in the initial trench, a second trench was excavated about 100 feet closer to the road alignment along the axial trend of the fissure. If the fissure could still be traced, another trench was excavated another 100 feet closer to the road. This process was repeated for each known or previously mapped earth fissure until the fissures' surface extent could not be traced further. The results of the subsurface exploration permitted the definition of the present maximum surface extent of the earth fissure along its projection toward the roadway alignment.

2) Suspect features were approached somewhat differently. A suspect feature is generally defined as a surface expression that has many characteristics of an earth fissure (generally a low linear depression), but also lacks some of the distinguishing characteristics of earth fissures (large cracks, headward erosion). An exploration trench was excavated perpendicular to the approximate center of the feature. No evidence of earth fissuring was found in any of the trenches excavated across suspect features.

The trenches extended from about 40 feet to more than 70 feet in length. Trenches were excavated to a depth of at least five feet below the existing ground surface. Some trenches could not be excavated to the desired depth due to backhoe refusal in caliche cemented alluvial soil. Each exploration trench was either logged or photo-documented by a geologist:

- Trench logging required the careful cleaning of excess dirt and removing smeared soil marks from the trench walls and floors that was caused by the backhoe bucket. The cleaning was required to clearly expose the soil stratigraphy and discontinuities such as cracks or fissure features that might be present (Figure 13). Common box nails were used to mark prominent contact boundaries between soil layers and any unusual cross cutting features. Once a soil profile was defined, each soil unit was described, and the soil layer boundaries were carefully plotted on cross-section paper at their height and distance relative to a horizontal datum line established along one of the trench walls. Any soil feature that appeared to be anomalous, such as cracks, unusual soil layer transitions or pinch-outs were carefully documented (Figure 14). Exposed earth fissure features were also photo-documented.

Figure 13: *Earth fissure No. 5 discontinuity exposed exploration wall of test trench T5-1 (see Tables 1 and 2).*

- Trenches that were photo-documented also required careful cleaning and examination of soil features uncovered in the trench walls and floor. Photographs were taken to document the conditions exposed in trenches where no earth fissure-related features were found.

If an earth fissure was confirmed, its location was staked adjacent to the trench. Likewise, where the earth fissures' surface expression was identified, its location was also staked at 100-foot intervals along it length for future survey location. A summary of the observations made in the exploration trenches excavated along the Thompson Road alignment is summarized in Table 2.

Observations and Findings

Earth fissures cut across by exploration test pits can be readily identified and located by detailed trench logging. Evidence that an earth fissure is present may be:

- An individual or series of continuous vertical cracks that cut across and disrupted the continuity of the soil layering.
- Anomalous vertical or near vertical soil dikes formed from soil deposited into the open earth fissures.

- Discontinuities (earth fissure cracks are exposed in both trench walls and cut the trench floor.

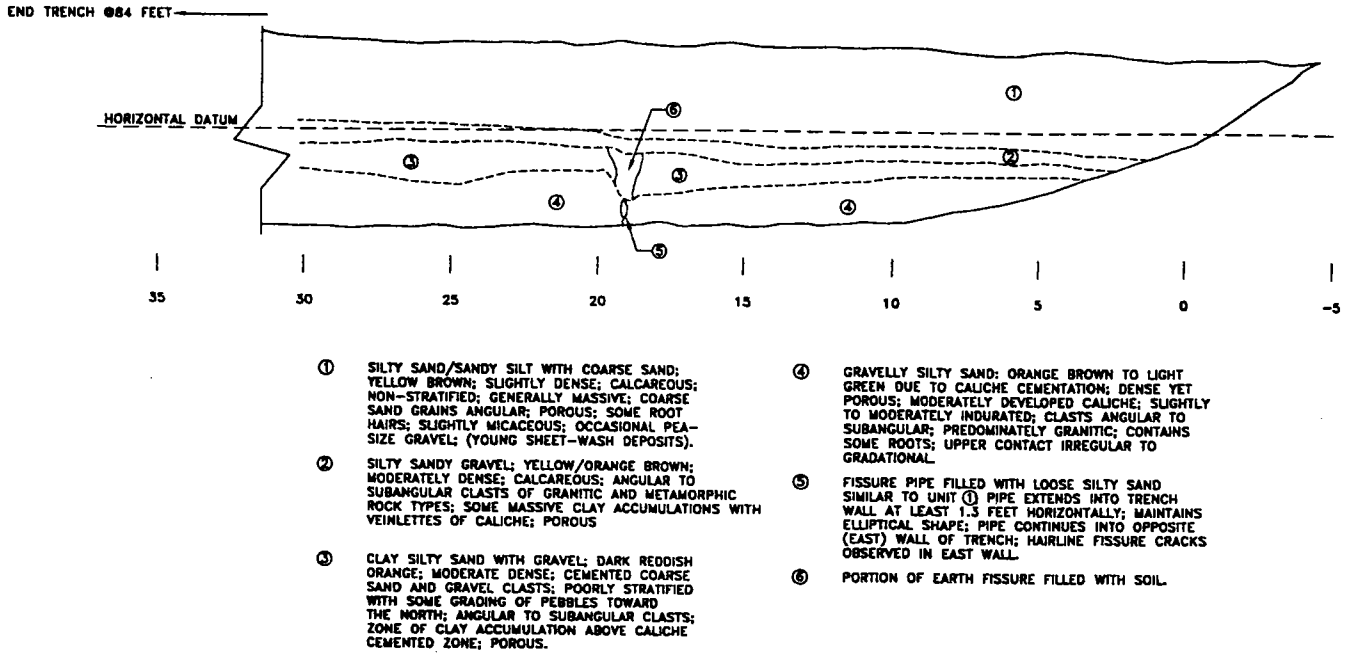

Figure 14: *Earth fissure exploration trench log for trench T6-1.*

Table 2
Description of Investigated Known Earth Fissures-New Thompson Road Realignment Pinal County, Arizona.

Earth Fissure Number	Trench Number	Earth Fissure Identified in Trench	Location (Figure 2)	Comments
2	T2-1	No	@ Station 199+50	Trench T2-2 located within highway ROW. Fissure trends toward roadway station 199+50. Last trace of earth fissure is approximately 230 feet from the proposed roadway centerline.
	T2-2	Yes	260 feet North of Sta. 199+50	
5	T5-1	Yes	500 feet East of Station 199+00	Fissure trends toward the new road between stations 185+00 and 187+00. Last trace of earth fissure is approximately 750 feet from the proposed roadway centerline.
	T5-2	Yes		
	T5-3	No		
6	T6-1	Yes	1,400 feet East of Station 175+00	Fissure projects toward the roadway near Station 174+00. Last trace of earth fissure is approximately 1,400 feet from the proposed roadway centerline.
	T6-2	No		

Conclusions

Research of available geological reports and documentation combined with geological investigation by various public agencies and private sector companies identified at least 23 previously documented earth fissures (Harris, 1994; Geological Consultants, 1999) north and south of Hunt Highway that are directly caused by ground subsidence induced by the removal of groundwater from the regional aquifer. Many of the known earth fissures were parallel to the initially proposed Thompson Road alignment. Recent aerial photography also revealed additional suspect earth fissures projecting toward the proposed alignment.

The geological reconnaissance and subsurface investigation of the known earth fissures confirmed that the fissures are active. Three earth fissures projected toward the proposed alignment, but were at least 200 feet or more away at their closest approach to the new Thompson Road right-of-way. The fresh surface expression of the earth fissures appears to indicate the fissures are active.

The earth fissures projecting toward the proposed Thompson Road right-of-way, in our opinion, do not appear to pose an immediate threat to the proposed alignment. However, the project is located in an area of active ground subsidence. Continued urban growth and development in the project vicinity, even thought that groundwater use is less than what it is presently for agricultural use, could cause a decline of the present water table that would induce increased ground subsidence rates. When this occurs, it is possible that the earth fissures that are apparently active today, will continue to grow in the future and impact the roadway . It is possible that the three fissures, as wells some new fissures that might develop, could intersect the roadway alignment. Pinal County was advised of this future potential subsidence-related concern that will require periodic monitoring.

In addition to mitigating the impacts of ground subsidence and earth fissures and highway design, construction, and operation, recommendations were also made to the County to carefully review all development plans for parcels that may be built along the Thompson Road right-of-way. Because of the active ground subsidence and earth fissures in this area, proper land use, grading, and drainage of lands adjacent to the roadway, will be critical for mitigating future potential effects of ground subsidence and earth fissures. Some of the potential adverse effect would likely include changes in land surface gradient, differential settlement, reversal of normal surface runoff, ponding of surface runoff, blockages of culverts and drainage ditches, and excessive erosion or sedimentation.

Likewise, builders and developers must provide assurances to the County that the future potential adverse effects of groundwater pumping and induced ground subsidence and earth fissures are accommodated in their development design plans and eventually construction. Likewise, assurances should also be provide that Thompson Road and it appurtenances will not be adversely impacted by nearby land development.

Hiring knowledgeable geologists experienced with local geology and geologic hazards at early stages of highway planning and development can often lead to simple, straightforward money saving solutions. Continuous interaction and communication between the agency, their design engineer and geological consultant are necessary components of a project to rapidly and economically resolve critical highway design issues such as ground subsidence and earth fissures. According to the Pinal

County Department of Public Works, the geological consultants initial recommendation to simply shift the proposed Thompson Road alignment to a new alignment east of the known earth fissure zone, resulted in the County saving an estimated one-half million to three-quarters of a million dollars that would have been needed for subsidence/earth fissure mitigation.

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The Interstate Technical Group On Abandoned Underground Mines

by

Thomas E. Lefchik, P.E.[1] and L. Rick Ruegsegger, P.E.[2]

ABSTRACT

Safety problems for travelers caused by sinkholes and slides initiated by abandoned underground mines are a growing concern as the mines in the United States deteriorate with age. Transportation agencies are faced with a potentially large number of sites that need to be addressed and a lack of knowledge and experience in dealing with these sites. The potential liability that these sites present can be significant.

The Ohio Department of Transportation (ODOT) developed an Abandoned Underground Mine Inventory and Risk Assessment process as a proactive method of locating and assessing the safety of mine sites underlying highways in the state of Ohio. The process is documented in an ODOT technical manual (ODOT, 1998) and a Federal Highway Administration (FHWA) publication (FHWA, 1999).

ODOT was in contact with several states and mine related agencies throughout the development of the inventory and risk assessment process and the investigation and remediation of several sites. During the development of the process, it was discovered that no states or other entities had a process to systematically locate and assess the safety of roadways underlain by abandoned underground mines. In addition, the knowledge and experience of the state DOT's was not being widely shared among the states.

Consequently the Interstate Technical Group on Abandoned Underground Mines was conceived. This group is composed of technically oriented individuals responsible for remediation of underground mines beneath highways in fourteen states and one Canadian province. The goals of the group are to 1.) generate and disseminate information and 2.) obtain outside funding or cooperatively share in the costs of related mutually beneficial efforts.

The group is growing and sharing information through workshops, a group interactive web site, and informal communication.

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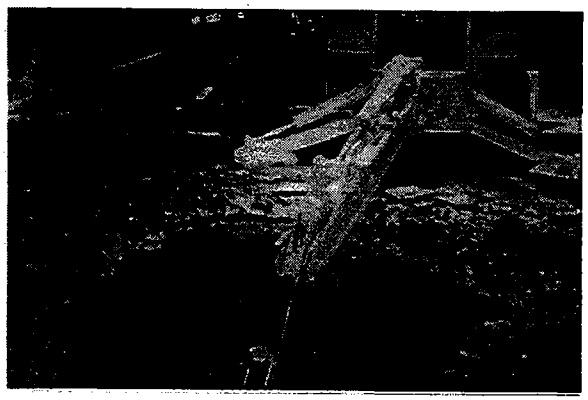
INTRODUCTION

A twelve foot (4 meter) section of Interstate Route 70 in Guernsey County, Ohio suddenly collapsed due to abandoned underground mine subsidence on March 5, 1995. This collapse resulted in near tragedy and the subsequent closure of this vital roadway for four months.



I-70

This incident was startling and attracted statewide and nationwide attention. However, it was not the first ODOT encounter with mines beneath roadways nor would it be the last. Discussions with ODOT maintenance personnel following the I-70 collapse revealed that subsidences due to mine collapse at other locations were periodically filled by maintenance crews. One location requiring periodic attention due to these subsidences was the interchange of I-70 and I-77, at one time the largest interchange in the United States.

It was becoming evident that mines beneath and adjacent to roadways were a past, present, and unavoidable future problem that was growing in proportions due to the increasing age of the mines. The Ohio Department of Natural Resources, Division of Geological Survey has detailed abandonment maps for approximately 4,200 mines in Ohio and estimates that there are approximately an additional 2000 more abandoned mines for

Map of mines beneath I-70/77

which no detailed maps are available. (Crowell, 1997)

Mines present a danger to the traveling public due to the potential for the collapse of the pavement, the formation of sinkholes in the shoulder and side slopes, the settlement of bridges and other structures, and the initiation of slides by mine collapse or mine drainage. The I-70 site in Guernsey County underscored the potential for severe danger to the traveling public and the potential liability to the state DOT.

These revelations caused ODOT personnel to realize that a systematic process was needed to address the issue. The number of sites was unknown but could be as many as 1000. Therefore it would be impossible to work on every potential site in the state.

Consequently, ODOT developed an Abandoned Underground Mine Inventory and Risk Assessment process as a proactive method of locating and assessing the safety of mine sites underlying highways. The process is documented in an ODOT technical manual (ODOT, 1998) and a FHWA publication (FHWA, 1999) which can be purchased by the general public.

STATE CONTACTS

During the development of the manual, ODOT contacted numerous state DOT's, state agencies and federal agencies to determine what those states and agencies were doing to locate, investigate, rate, remediate, and monitor sites where abandoned mines are located beneath roadways. Many state DOT's and state and federal agencies were very helpful in providing guidance and in sharing their knowledge and experience.

However, it became evident that the knowledge and experience of the state DOT's was not being widely shared among the states. The various state DOT's were wrestling with the same issues and problems that the Ohio DOT was now facing. They each had made some attempts, both successful and unsuccessful, in various methods and procedures. Some of the successes and failures were repeated in some states without the knowledge of the experiences of other states.

The personnel dealing with the mine issues in the states were also encumbered with numerous other responsibilities and pressing issues. Consequently the personnel in each state could only expend a limited effort on mine related responsibilities.

Collapse on a county road in Missouri

The problems of mines beneath roadways is widespread. The United States Department of the Interior, Office of Surface Mining Reclamation and Enforcement (OSMRE) maintains a repository of mine maps (mostly coal mines). This repository contains maps for mines in 45 states. (OSMRE 1997) Maps are not available at the repository for all mines. Consequently the number of mines and the number of states with mines is larger.

Many transportation agencies are faced with a potentially large number of sites where mines lie beneath roadways. These sites need to be assessed and/or addressed. However, these agencies are hindered by a lack of information, knowledge, experience, resources, and time. This has hampered progress in adopting and developing methods to systematically locate, inventory, investigate, rate, remediate, and monitor the sites.

WORKSHOP

The Ohio DOT decided to host a workshop to bring together the state DOT's and state and federal agencies to share their knowledge and experiences. Funding for the workshop was provided by the Federal Highway Administration. The workshop was held on September 16-18, 1997.

The following ten states and one turnpike authority participated in the workshop:

- Illinois
- Kansas
- Kentucky
- Maryland
- Missouri
- New York
- North Dakota
- Ohio
- Pennsylvania
- Pennsylvania Turnpike

In addition the following seven state and federal agencies participated:

- United States Department of Interior, Office of Surface Mining
- United States Geological Survey
- United States Department of Transportation, Federal Highway Administration (FHWA)
- Ohio Department of Natural Resources
- Ohio Mine Subsidence Inventory Underwriting Association
- Ohio Environmental Protection Agency
- Ohio Department of Development

The primary focus of the discussions was on procedures to inventory and assess sites. However, other aspects of dealing with mines beneath roadways; such as methods of locating mine voids, monitoring methods, and remediation methods; were discussed as well.

The discussions at the workshop resulted in a lively exchange of information and all participants benefitted. As a result of the success of the workshop, the participants expressed a desire to continue their cooperation and sharing of information.

GROUP FORMATION

Consequently the Interstate Technical Group on Abandoned Underground Mines was conceived.

It was agreed that the group would be comprised of technically oriented individuals responsible for the remediation of underground mines beneath highways. The group expressed a desire to continue to exchange information individually and as a group with other state and federal agencies, private industry and firms, as well as other organizations. But it was believed that group goals could best be achieved with a focused membership.

The following are the current group members:

- Arizona
- FHWA
- Illinois
- Indiana
- Iowa
- Kansas
- Kentucky
- Maryland
- Michigan
- Missouri
- New York
- North Dakota
- Ohio
- Ontario, Canada
- Pennsylvania
- Pennsylvania Turnpike
- West Virginia

The goals of the group are to:

- Generate and disseminate information.
- Obtain outside funding or cooperatively share in the costs of research or other related mutually beneficial efforts.

The expected benefits to be derived from the achievement of the above goals are:

- Increased efficiency and effectiveness of each state's operations
- Enhanced safety of the traveling public.

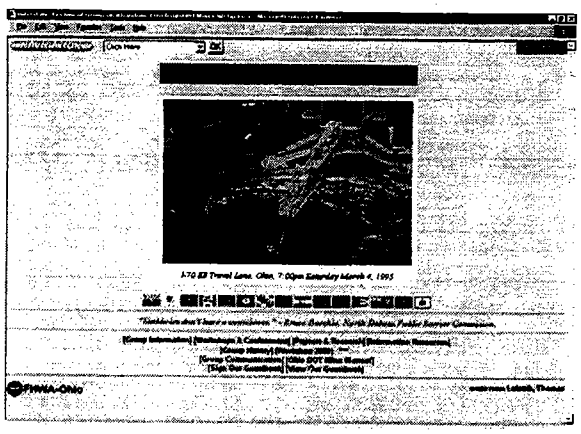
GROUP COMMUNICATION

Various means are being pursued to fulfill the goal of generating and disseminating information. First, and perhaps most important, the group members developed contacts with each other through the workshop in Ohio and subsequent workshops. This has led to informal communication between

various members through telephone and e-mail communications.

GROUP WEB SITE

Another means of generating and disseminating information is an interactive internet web site currently hosted by the Federal Highway Administration at: www.fhwa-ohio.org/mine.html. This web site features contact information for all members, information regarding current and completed projects and research, a discussion board for active group communication, summaries of completed workshops and information for the upcoming workshop, a copy of the Ohio Department of Transportation's mine manual, links to related web sites, and a multimedia page. As money and time become available it is hoped that other features can be added to the site.



Group Web Site Home Page
www.fhwa-ohio.org/mine.html

The most significant features of the site are the interactive pages.

The page containing information regarding current and completed research and projects is updated by the individual state members. This helps to assure that the information is accurate and up-to-date. The page includes the type of work, location, date completed, degree of success, and whether a report is available.

The discussion board is used by the group members to post questions and allows the other members to post responses. Not only does this allow group members to draw on the knowledge and experience of the other members but it also will provide a library of useful information that can be accessed through a search function so that members and the public can draw upon the stored responses.

ADDITIONAL WORKSHOPS

Another workshop was hosted by the Missouri Department of Transportation in St. Louis in August 1998. This workshop focused on geophysical methods of investigation and monitoring but also included discussions on various other issues. Participants included six states - Iowa, Illinois, Kansas, Missouri, Ohio, and Pennsylvania and one state and one federal agency. In addition, there were presentations and participation by seven private practice firms. At this workshop the group expressed a desire to meet every two years in workshops.

Another workshop was hosted by the Kansas Department of Transportation in Kansas City in April 2000. Nine member states plus federal and state agencies, private consulting firms, industry, and academia participated in this workshop.

Some significant issues were discussed at the Kansas workshop. The workshop opened with an afternoon of presentations by the various participating transportation authorities. The balance of the workshop presentations focused on investigating, monitoring, and remediating abandoned underground mines beneath roadways. Presentations were made regarding the roadway applications of geophysical

techniques such as ground penetrating radar, seismic reflection, borehole tomography, time domain reflectometry (TDR), surface wave inversion and attenuation, micro-seismic monitoring, and other technical subjects. All of these presentations included case studies demonstrating the application of the different geophysical techniques to roadway locations underlain by abandoned underground mines.

The workshop also included a field trip to an abandoned underground limestone mine which is being converted for use as commercial space and as a local college's library.

The exchange of information is a primary goal of the group. At the Kansas workshop it was noted that other modes of transportation, specifically railroads, face the same problems with abandoned mine hazards. Consequently the group agreed to invite other modes of transportation to become involved with the group. In addition, other mine related agencies and firms are involved with many of the same issues and use many of the same methods of monitoring, investigation and remediation. Therefore, it would be mutually beneficial to involve these groups to a greater degree.

Other organizational affiliations are being investigated but it appears that the group will remain independent and informally organized.

The next workshop will be held in 2002.

RESEARCH

There has not yet been any active sharing of research cost or effort by the group. However a significant benefit has been achieved through the sharing of information which prevents states from pursuing unsuccessful efforts and concentrating on trying untried methods, making modifications to unsuccessful methods, or using successful methods in untried conditions. This is an informal method of sharing the research effort which is already showing a benefit.

It is likely that group members will generally each choose research topics that suit their state needs or interests and share the results with other states. The use of pooled funds for a common research project remains a possibility.
No specific research projects have yet been identified for the application of pooled funds or outside funding. Also no sources of outside funds for research have been identified.

SUMMARY

The Interstate Technical Group on Abandoned Underground Mines is meeting its original goals to
1.) generate and disseminate information and
2.) obtain outside funding or cooperatively share in the costs of research or other related mutually beneficial efforts.

The expected benefits of:
1.) increased efficiency and effectiveness of each state's operations and
2.) enhanced safety of the traveling public
are currently being realized and are expected to be realized in a greater degree in the future.

The membership of the group has grown to 17 members and remains open to interested agencies.

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Transition to Pro-active Practices for Applying Geophysical and NDT Methods

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The presentation covers an overview of the current FHWA and AASHTO guidelines, and the state of practice for subsurface investigations and site characterizations. Emphasis will be placed on the rational for change to pro-active practices.

A review of innovative contracting, construction fast-tracking, current construction management and engineering practices will show that over 50% of all construction claims and change orders are the result of insufficient subsurface information, unintentional discovery of obstructions, voids, utilities, abandoned or active mines, and "unknown or changed" conditions. The related costs of these is well over $100 million annually on transportation projects only. Expanding this criteria to other areas in private and public sectors will generate substantially more savings.

A rational program to develop referenced improvements in safety and implementation of substantial cost savings will be illustrated by accomplishing a better understanding of the applications and limitations of each method, and with the appropriate selection and use of the various geophysical technologies.

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Highway Site Characterization Using Seismic Tomographic Imaging

by

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ABSTRACT

Geotechnical engineers have struggled for years with the difficult task of characterizing geologic conditions that affect the design and stability of various transportation infrastructure, including road cuts, slope stability and slides, tunnels, and bridge abutments. Commonly, designs are based on data from borings that may or may not reflect the true character or continuity of the materials present and may not detect the presence of structural defects (faults, shear zones, slip planes) or previous excavation activity. Geophysical techniques such as seismic reflection, seismic refraction, and ground penetrating radar have been used with some success to expand the knowledge of subsurface conditions prior to excavation; however, these techniques typically interfere with the construction process and are time-intensive.

To improve the quality, continuity, and timeliness of geotechnical data for site characterization, NSA Engineering, Inc., has developed the RockVision3D™ tomographic imaging system. This system is based on the principle that acoustic waves travel through different materials at different speeds and with different degradation rates. By recording the waveforms from known acoustic sources, three-dimensional tomograms representing the relative competence of the materials that the waves travel through can be constructed. Although tomographic techniques are not new, the RockVision3D™ system incorporates advancements in the technology that allow for near-real-time imaging, making tomography a practical tool for geotechnical site characterization, both in the design stages and as excavation proceeds. With the information that this technique provides, the risks and expenses associated with unknown geologic conditions are greatly reduced.

This publication describes the seismic refraction tomography and seismic reflection tomography systems and gives several case studies describing their application for highway site characterization.

INTRODUCTION

The underlying principle of seismic tomographic imaging is that the interior of a volume can be characterized by propagating waves from an energy source through the volume and examining the transmitted and scattered wavefields detected at the boundary of the region. Direct arrival seismic tomography exploits the fact that seismic energy travels through different material types with different velocities. Seismic waves will travel faster and with less attenuation through competent or highly stressed rock than through broken rock or voids (Nur, 1987; Shea-Albin, et al., 1991; Yu, 1992). Geologic anomalies such as voids, structural anomalies such as faults, or compositional anomalies such as hard inclusions can be tomographically imaged when surveyed using the proper techniques and instrumentation.

Seismic energy sources may be dedicated seismo-acoustic sources such as detonators, pneumatic fired cavitating devices, or magnostrictive. With proper filtering, the

sounds produced as part of the construction process by drilling, excavation, or tunneling equipment can also be used. Analysis of the seismic waveforms may be performed with either shear (S-wave) or compressional (P-wave) energy. In practice, data sets are collected as full waveform signals that are recorded on a seismograph using a sampling rate and duration appropriate for the signal source, the earth materials anticipated, and the physical scale of the study. Source frequencies must be considered in this process because smaller scale targets or smaller physical dimensions require higher frequency sources.

The tomographic inversion algorithm used in RockVision3D™ models seismic energy transmission and dispersion using a variation of the eikonal equation, a finite-difference approximation to the wave equation. Waves are propagated through a finite-difference mesh to model energy transmission through the medium. Mesh properties are iteratively adjusted until simulated energy transmission measurements match field measurements, using a variation of SIRT (Simultaneous Iterative Reconstruction Technique). SIRT uses the concept of a ray packet, the region through the mesh corresponding to the highest probability of energy transmission. In the case of direct arrival transmission velocity, the ray path region through the mesh must satisfy:

$$t_L = \oint_{Ray} \frac{L}{V} dL$$

where t_L is the computed travel time from the energy source to receiver, L is path length, and V is the velocity of the medium along the ray path. This ray path is not normally straight, but rather undergoes significant refraction from velocity contrasts between lithologic units. The ray path is used to adjust the velocity model according to:

$$S_{i+1} = S_i + \sum_c \frac{t_m - t_c}{L_c} Gauss_R(Min_D[P_n, L]),$$

where S_i is the slowness (inverse of velocity) at a given mesh node point for iteration i, t_m is the time measurement from the field, t_c is the time computed from the simulation, L_c is the length of the simulated ray path, $Gauss_R$ is the Gaussian probability distribution as a function of resolution and transmission frequency, and Min_D is the minimum distance from mesh node point P_n to ray path L.

Tomographic inversion techniques are often divided into straight-ray versus curved-ray techniques. Straight-ray techniques are usually considered faster but are valid only for low-velocity contrast mediums. Curved-ray techniques produce better results for geotechnical applications, but have traditionally employed computationally intensive shooting, relaxation, or conjugate gradient techniques for determining the ray path location. A curved-ray technique developed by NSA is used in the RockVision3D™ software that computes the ray path location using steepest descent on the travel-time mesh, nearly matching the computational efficiency of straight-ray techniques. Curved-ray techniques also occasionally suffer from problems such as instabilities or shadow-zones. Use of a node-based mesh, rather than a pixel-based mesh, eliminates these inconsistencies.

Flexibility in the application of the tomographic imaging method is essential to conform to various site conditions ranging from rock to dry and unconsolidated soils to saturated clays and flowing sands along coastal regions. Traffic noise, nearby utilities, dense clays, brine, and other factors that can often affect standard geophysical methods and GPR can be handled effectively with seismic tomography methods, however.

The software iterative process is illustrated in Figure 1.

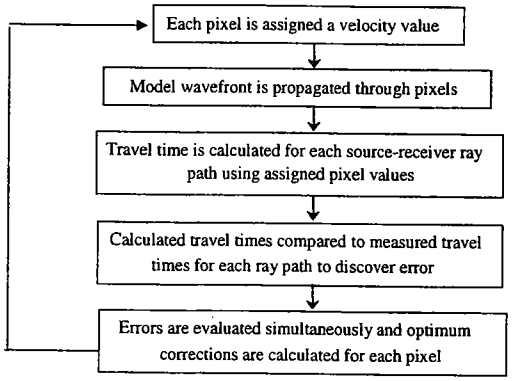

Figure 1. Summary of RockVision3D™ processing for velocity tomography.

SLOPE STABILITY

The design of excavated slopes is of major importance in the mining and civil construction industries. Whether the slope design is for an open pit mine, a highway or railroad cut, or as part of building site preparation, there is a tradeoff between excavation cost and the potential for slope instability. As the angle of the slope is increased, the volume of material that has to be excavated is decreased, resulting in faster excavation and lower excavation cost. At the same time, the probability of slope failure is increased. Conceptually, there is an optimum slope angle for every project that satisfies the following equation:

$$C_e = C_f \times P_f \times SF,$$

where C_e = cost of excavation, C_f = cost of failure, P_f = probability of failure, and SF = safety factor.

Uncertainties are associated with each of the variables in this equation. The cost of excavation is fairly straightforward to estimate and has the least uncertainty associated with it. The costs of slope failure, both economic and human, are harder to estimate, as is the probability of failure. To compensate for these uncertainties, a judgment is required as to an appropriate safety factor. Thus, slope design is essentially an exercise in risk management.

The probability of failure for any given slope design depends on the character of the ground mass being excavated. The uncertainty associated with that probability lies in the fact that it is impossible to obtain a set of data that completely describes the ground mass. Rock and soil types vary throughout the excavation, as does the degree to which they are weathered and altered. In rock slopes, the orientation, spacing, persistence, and frictional characteristics of discontinuities (faults and joints) normally control slope stability; in soil slopes, hydrology and soil properties dominate. Although efforts are made to characterize the ground mass through geologic mapping of surface exposures, core drilling, etc., unexpected conditions are invariably encountered during the course of excavation.

Also at issue, irrespective of the slope angle, is planning for the stabilization of isolated areas of weakness in the slope wall. Loose rock and/or isolated wedge block failures require stabilization using combinations of fixtures such as soil anchors, rock bolts, cables, wire mesh, and shotcrete. The types and quantities of stabilization fixtures are difficult to accurately plan for, again due to unknown subsurface conditions.

System Installation and Data Collection

For a slope stability analysis project near Blackhawk, Colorado, an array of source and receiver locations was laid out at the site, as shown in Figure 2. The array was designed to allow for the calculation of a tomogram of the entire excavation, with coverage particularly concentrated on the west wall. A total of 36 geophone receivers were attached with epoxy

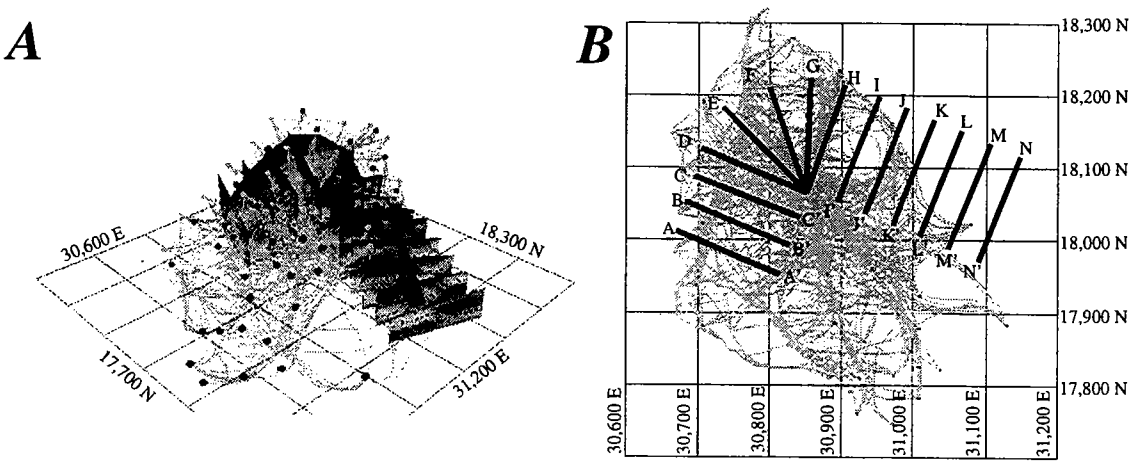

Figure 2. Orthogonal (A) and plan (B) views of ray path coverage between sources and receivers, showing good coverage of the area of primary interest, the west wall, and the west end of the north wall.

to bedrock or large boulders on the surface. The seismic source consisted of a standard blasting cap and two primers; this charge was set off at each source location, one location at a time. As a shot was fired, the time of the shot was recorded using a geophone trigger located 1 to 2 ft from the shot. For each shot, the trigger signal and signals from the other 36 receivers were recorded with a 48-channel Geometrics seismograph. Source and receiver locations were surveyed using a GPS instrument.

A three-dimensional velocity tomogram of a volume slightly larger than that defined by the limits of source and receiver locations was produced from the seismic survey data.

The initial cut slope angle of 53° was selected based on examinations of other excavations in the area. The tomograms generally supported this cut slope angle with certain exceptions at cross-sections B-B' and I-I.' The rock structure also was favorably oriented for this cut angle except for a potential planar failure along foliations in the west cut face. The tomograms were invaluable in determining the extent of the weak and strong zones in the cross-sections for slope stability analyses and for stabilization planning in estimating quantities of rock bolts, ground nails, wire mesh, and reinforced shotcrete, Figure 3.

The stabilization plans prepared for the overburden, weathered bedrock, and intact, non-weathered bedrock addressed potential translational and rotational global slope failures, as well as toppling and wedge failures in each of five segments comprising the excavation. A pattern of permanent ground nails attached to a reinforced shotcrete section was designed to cover the overburden and weathered rock. The intact, non-weathered bedrock stabilization plan consisted of patterned rock bolts to stabilize potential wedge failures and double-twisted wire mesh to capture smaller wedges likely to dislodge during storm events and freeze-thaw cycles.

The design parameters used to characterize the ground materials were estimated from classification tests of overburden samples, peak and residual direct shear test results (along foliations) of rock specimens taken at the site, and published reports on similar materials. Critical cross-sections and elevation views were generated from the tomograms to determine the extent of the various ground materials and to estimate the

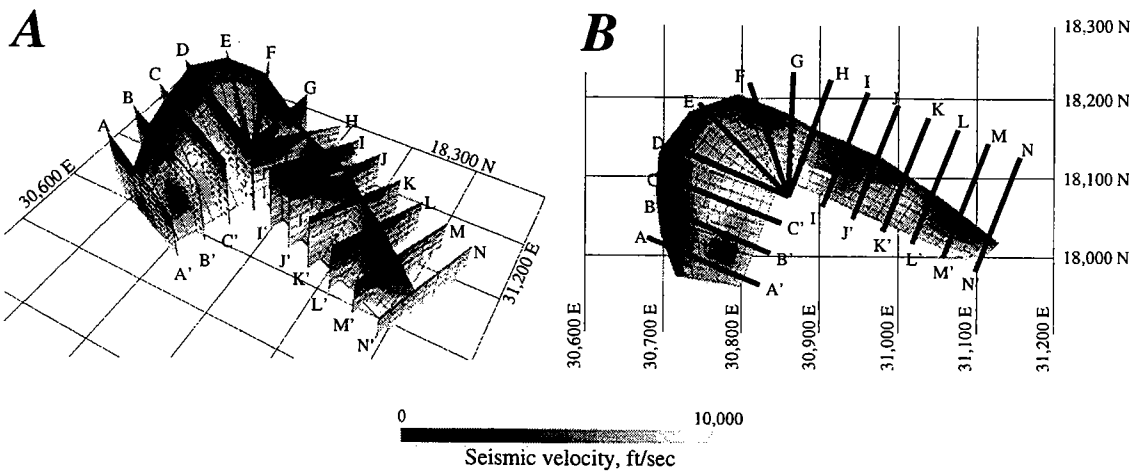

Figure 3. Orthogonal (A) and (B) views of slope cut at 53° along both the west and north walls.

stabilization quantities. The permanent ground nails and reinforced shotcrete ranged in height from 15 ft to 80 ft, covering 17 percent of the total excavated face area, or 28,000 square feet. This quantity extended to cover the weak zones illustrated in sections J-J' and I-I', Figure 4.

To analyze the critical cross-section for the planar failure, conservative shear strength estimates for the foliation planes that represent a peak (intact) and residual strength condition were determined. Assuming that the high wave speeds shown in the tomograms represent intact peak strength values and the slower wave speeds represent the residual strength values, a limiting equilibrium slope stability analysis with the typical patterned rock bolts (needed for wedge failures) produced a factor of safety of 2.0. This factor of safety was above the desired value of 1.5.

LOCATION OF KARST AND VOIDS

Abandoned works from old mining activities, limestone karst, and voids associated with basalt flows and volcanic activity can affect many locations in the western United States. Site investigation by drilling is often difficult and expensive. The application of seismic tomography in locating and delineating these features has proven successful on multiple projects in the last two years.

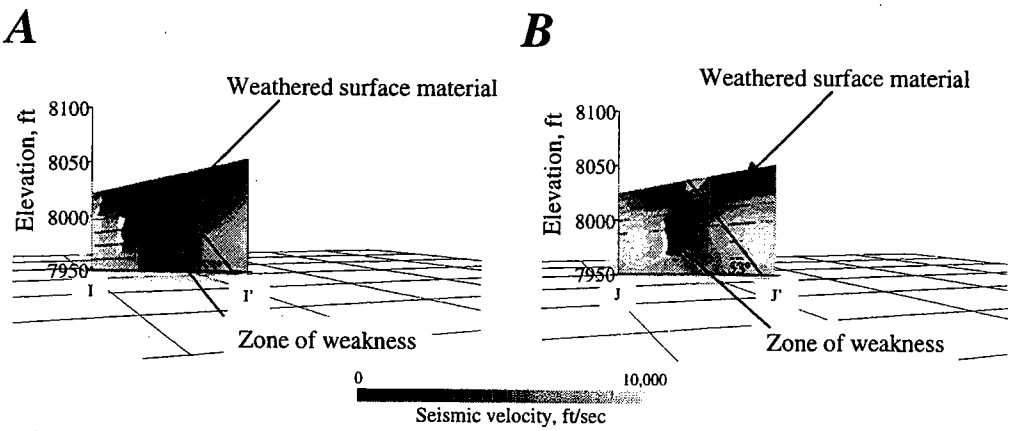

Figure 4. View looking northeast perpendicular to section A-A', showing 40° (A) and 53° (B) slopes along the west wall.

During the planning and construction of a new highway overpass on Interstate 25, the Colorado Department of Transportation (CDOT) became concerned that the foundational stability of the bridge support caissons could be compromised by the existence of old mine works beneath the caissons (Hanna and Haramy, 2000). Information from the Colorado Geological Survey (CGS) indicated the existence of a rock tunnel connecting two abandoned coal mines and crossing the interchange below Pier 3, which is currently under construction. If the tunnel has collapsed, its cave zone may extend to the bearing horizon, reducing the bearing capacity of the foundation and the skin friction along the caisson. In an attempt to better define the location and alignment of the tunnel and to assess its condition or state of collapse, a three-dimensional cross-borehole tomographic survey was performed. The purpose of the study was twofold:

(1) To image the subsurface geologic structure including strata and voids or rubble resulting from past coal mining operations.
(2) To evaluate the extent of the subsidence zone that may occur due to mine entry/tunnel collapse.

A total of six vertical boreholes, defining a zone/volume approximately 34 m wide by 84 m long by 61 m deep, were used to conduct a cross-borehole seismic survey. All boreholes were drilled and cased with 100-mm perforated PVC. The field survey was conducted according to the procedure and approach discussed in the previous section. As a result, over 30 colored 2-D and 3-D tomographic images were constructed, including horizontal and vertical cross-sections, at 1.5-m increments to delineate the location of the tunnel.

Tomographic imaging results are depicted in several cross-sections, as shown in Figure 5. The location of the suspected tunnel was established at a depth between 27 m to 30 m. The results from the three-dimensional images indicate that the tunnel is not intact, and that a caved zone extends from the tunnel horizon to near the ground surface. As shown in Figure 6, three distinct zones were delineated: (1) unconsolidated, low-velocity zone (< 20 blows); (2) intermediate consolidated zone (>30 blows); and (3) consolidated, high-velocity zone (> 50 blows). Based on field studies of sinkhole subsidence previously performed in the area (Matheson, 1987), the expected extent of the rubble zone and angle of draw were superimposed on the tomography results and related to the planned location and bearing horizon of the bridge caissons. Figure 7 shows that if further sinkhole-type subsidence is to occur, the bearing capacity of Column 1 will be affected.

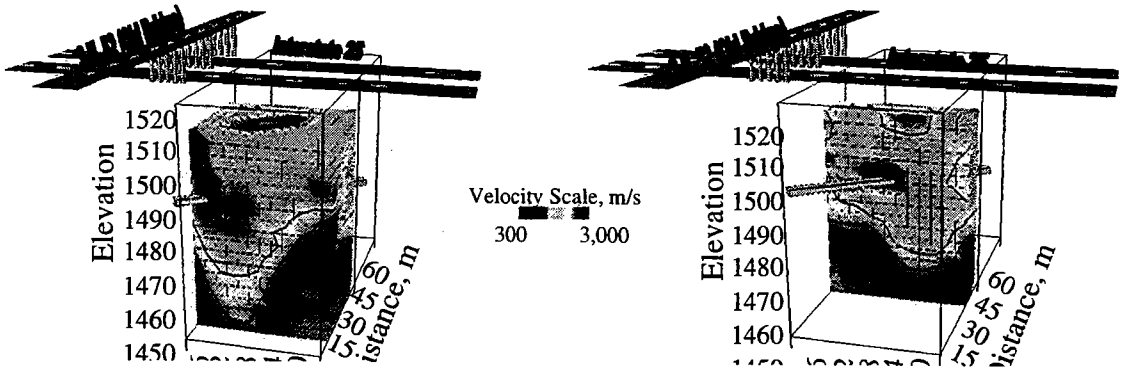

Figure 5. Tomographic image showing tunnel location at various cross-sections.

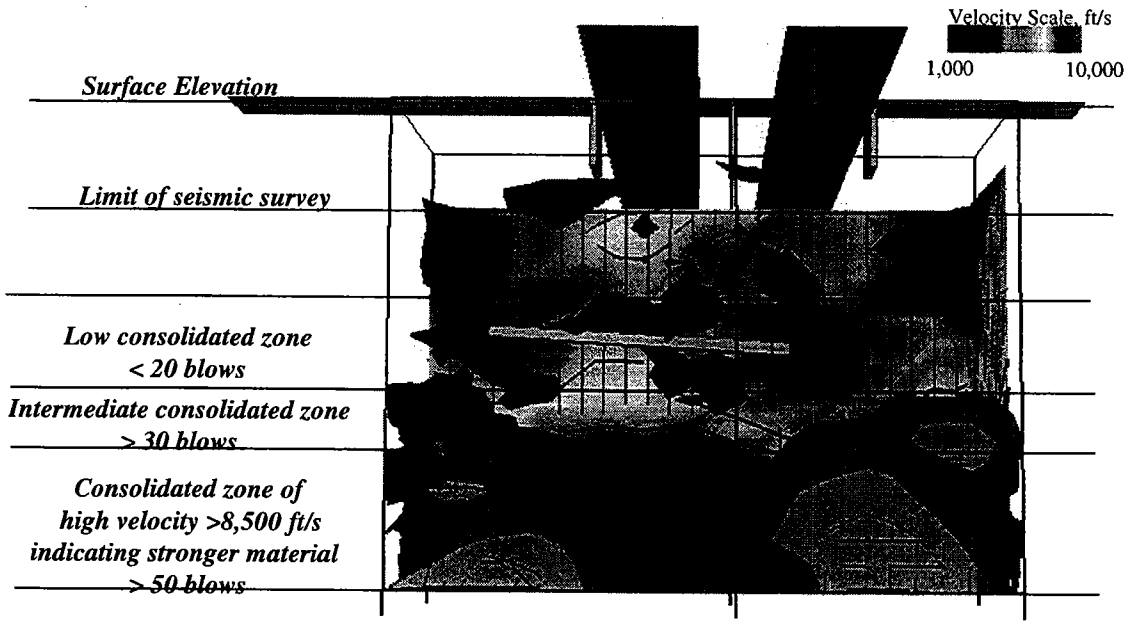

Figure 6. Three-dimensional tomographic image showing tunnel location, cross-section 100+613.2.

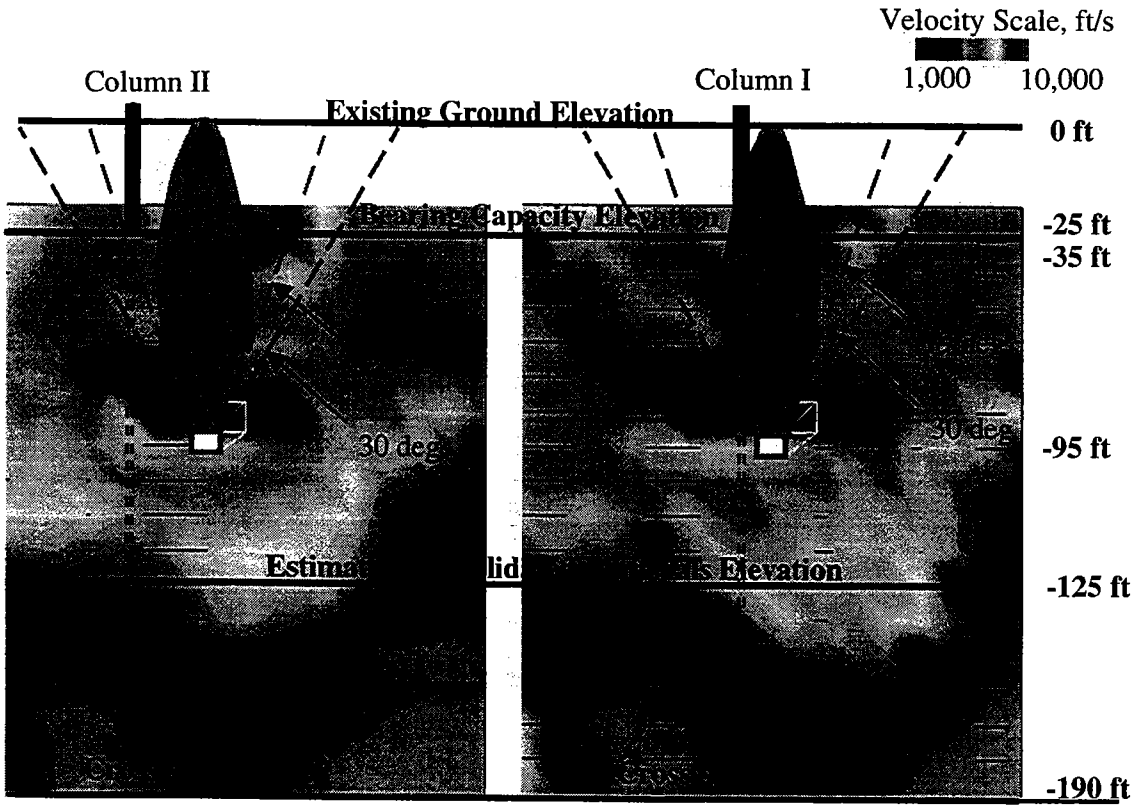

Figure 7. Two-dimensional cross-sections perpendicular to the tunnel direction superimposed with the subsidence limit lines and sinkhole profile line.

Grouting and Grout Effectiveness

Figure 8. On-site data reduction.

The Bill Emerson Memorial Bridge is being constructed over the Mississippi River between Missouri and Illinois. The river bottom consists of karstic limestone; and voids. Structural defects within the limestone necessitated reinforcement of the foundation by high-pressure jet grouting. To evaluate the effectiveness of the grouting program, NSA performed cross-borehole tomographic surveys before and after jet grouting.

From a construction platform in the river (Figure 8), six boreholes were drilled in an area approximately 3.2 m wide by 3.4 m long to a depth of about 22 m below the river bottom. The survey was conducted in a manner that allowed a three-dimensional tomogram to be produced. The well-log geologic data were inputted to the RockVision3D™ program as known information. The results were used to delineate clay-filled zones and characterize the effectiveness of grouting in a complex geologic environment.

Figure 9 shows the interpolated well-log images and sections of the resulting tomogram before and after jet grouting operations. Results of the survey showed that jet grouting significantly increased seismic velocities throughout the treated volume and allowed the Missouri Department of Transportation (MDOT) to confirm that the jet grouting effort was adequate for the pier foundation as designed.

Highway Remediation Grouting

After the construction of Colorado Highway 287, the presence of a relatively hard clay deposit beneath the highway caused differential settlement resulting in cracks and vertical displacement in several sections of the roadway.

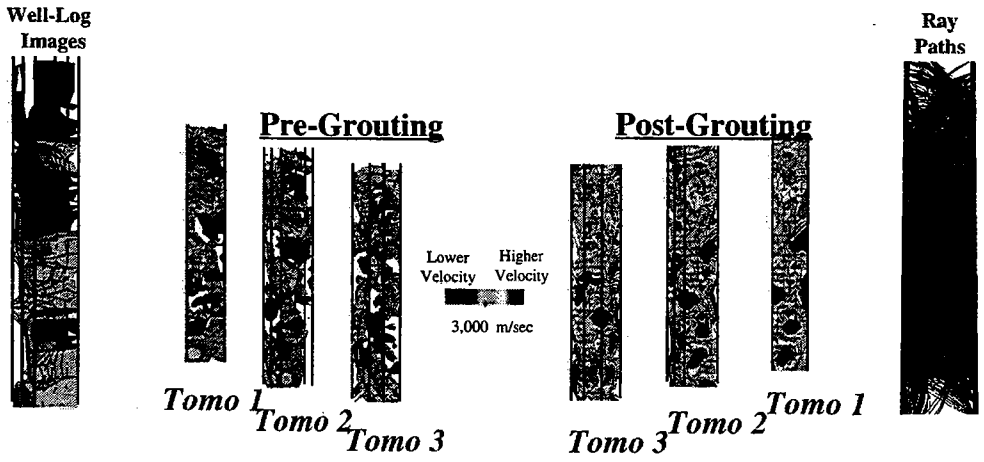

Figure 9. Interpolated well log and pre- and post-grout images beneath river bottom formation underlying the foundation of a pier for a bridge structure.

Tomographic imaging using NSA's RockVision3D™ system was applied to determine the nature of the subsurface ground condition and assess remedial grouting effectiveness. A three-dimensional cross-borehole tomography survey was conducted using five boreholes drilled to a depth of about 8 m below a selected section of the damaged roadway.

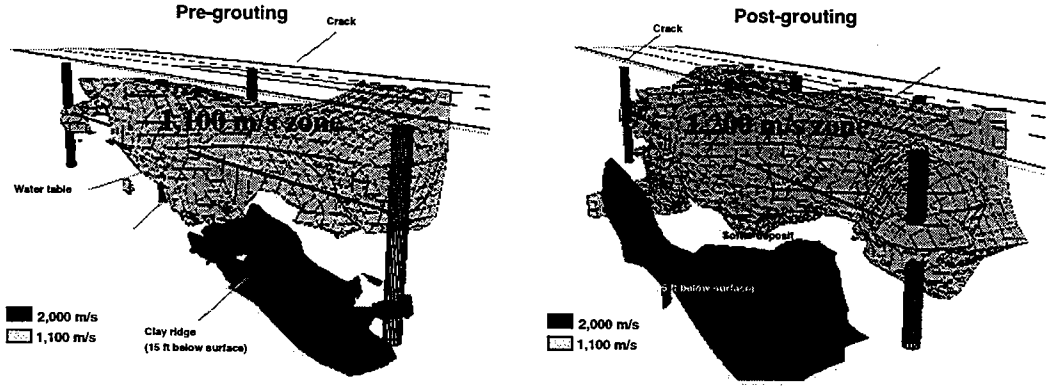

Figure 10. Three-dimensional velocity contour images show comparison between pre-grouting and post-grouting.

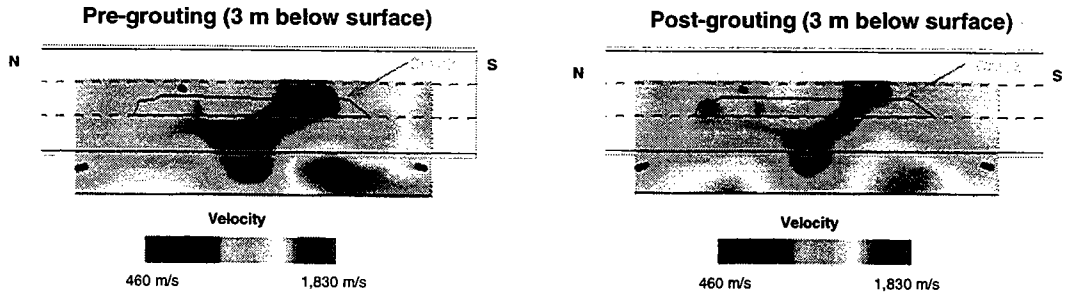

Figure 11. Tomograms at 3 m below surface for pre- and post-grouting show improvement of the ground conditions.

Tomographic images were produced before and after grouting. Three-dimensional velocity contour images, as shown in Figure 10, provide comparison between pre-grouting and post-grouting conditions. The low-velocity area (1,067 m/s) in the pre-grout survey is altered to a higher velocity area (1,220 m/s) in the post-grout survey due to a change in density. The velocity increase in the grouted area indicates that the ground condition is improved by the grouting. Figure 11 shows tomograms at 3 m below the surface for pre- and post-grouting, indicating improvement of the ground conditions. After grouting, the density of the low-velocity area (762 m/s) has been increased by the grout.

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Application of Specialized Drilling Technology and Equipment for Difficult Access

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The need to access difficult and logistically complicated areas to obtain informative data presents a challenge within the geotechnical industry. Specialized drilling technologies and equipment such as small, powerful, componentized drill rigs, versatile rubber track drill rigs, low displacement barges and platform systems are being utilized and are constantly advancing to improve the proficiency of engineers to acquire this information at the exact point of interest. This paper will describe the appropriate drill rig technologies to access difficult terrain along with the correct sampling devices. Case histories of several projects ranging from Highway 101 landslides in Washington, drilling retaining walls in Glacier National Park, barge access on Henry's Fork of the Snake River and road alignment projects in several states will illustrate the ability of this technology to safely access challenging sites while retrieving valuable data and installing monitoring instruments. Challenges associated with these projects and their solutions will also be discussed.

Quality and Durability of Aggregates Determined by Multivariate Statistical Analysis.

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Karol Grunner, Technical University, Bratislava, Slovakia

ABSTRACT

The quality and durability of aggregates depends on the sum of their properties, yet the current methods of determining these properties is based on their performance in individual tests. For aggregate to pass, the test results must be less than some predetermined limit for the given test. If it does not pass a given test, it may be rejected, or an 'exception' may be granted. Rather than depend on the results of the individual tests, an overall performance of the aggregate should be considered, as well as its performance similarity to other aggregates. The techniques best suited for this are multivariate statistical analysis, especially those of stepwise regression and K-means cluster analysis. Two databases of aggregate properties were statistically analyzed: One database of over 650 aggregates from Slovakia, and another database of 27 well studied aggregates from Ontario, Canada. The aggregates in these databases were subjected to at least 8 different quality and durability tests. The Ontario set additionally contained a quality record estimate for each aggregate type. K-means cluster analysis applied to the databases allowed the grouping of aggregates into durability or quality categories that reflected their behavior in service. It is concluded that aggregate quality determined by multivariate statistical analysis is a reliable predictor of their long-term durability.

INTRODUCTION

Evaluation of aggregate quality is based on the methods used for decades. Some tests, devised to predict the aggregates' performance in service in the early part of 1900's are still used today, with relatively minor modifications. The aggregate must pass each of the specified tests in order to be accepted. Although there is nothing inherently wrong with most of the tests, they are often blindly prescribed, even if the aggregate will never be exposed to the conditions that the tests simulate. For instance, L.A. Abrasion test is run for aggregate that may be used in mass concrete, and never exposed to abrasion or impact. Some tests are almost useless: Magnesium or sodium sulfate tests are run routinely, and relatively inflexible passing limits are assigned to the losses, even though the material may never be exposed to freezing conditions. Different rock types have significantly different response to the test, and the test is barely reproducible in inter-laboratory tests on the same aggregate.

Once the aggregate is tested, the results are filed, and rarely consulted again. Databases of aggregate test results are rare. In this age of powerful desktop computers and sophisticated data storage and analysis software, burying the data in filing cabinets is tantamount to waste on Pentagon scale! This paper illustrates how the test data can be used to advantage to not only classify the aggregate, and provide insight into their properties.

For this paper, two databases, each consisting of somewhat different sets of tests, and different testing philosophies are used to illustrate the power of multivariate statistical approach in classifying coarse rock aggregate. The first set of data is the result of the tests done by the first author on rock types identified and collected by the Ministry of Transportation Ontario (MTO)

petrographers as part of the Petrographic Analysis Test for Coarse Aggregate (MTO Test Method LS-609). Each sample represents one of the major petrographic types or categories, and is assigned a 'quality' designation of 1, 3, or 6, 1 being well performing rock type, and 6 a relatively poor quality rock. The tests performed on this rock group included:

- Adsorption at 95% Relative Humidity, 72 hours
- Absorption, 24 hours
- Vacuum Saturation, 4 hours
- Density
- Point Load Strength
- British Impact Test, Dry sample
- British Impact Test, Saturated sample
- Freeze-Thaw Loss, 5 cycle, 3%NaCl Solution

The second set of data from central European country of Slovakia is provided by the second author, and represents compilation of aggregate testing results over the last 30 years. The database consists of over 650 test results on a variety of rock types. 427 of these, representing the major rock types, were selected. The tests selected for analysis include:

- Polishing Factor
- LA Abrasion
- MicroDeval Abrasion
- Freeze-Thaw in water, 200 cycles
- Na Sulfate, 15 cycles
- Absorption, 24 hrs
- Absolute Porosity
- Density, SSD
- Density, apparent

MULTIVARIATE STATISTICAL ANALYSIS CONCEPTS

The classification of construction materials by a variety of standard tests is used to assure a quality product, especially in the field of concrete technology. A crushed stone source is expected to pass a required set of tests before being accepted. In some cases, it passes most, but not all tests. This puts the project manager or specifying agency in a quandary - should the source be used, or should the material be imported from a more distant area at additional costs? Does the failure of one test imply that the material is of low quality? In such cases, exemptions may be granted on case-by-case basis, perhaps based on service record, economic considerations, or political pressures. Statistical classification may be the preferred tool in such cases.

Most researchers are familiar with and apply uni- or bivariate statistical methods to analyze their data. Comparisons of means and variances to emphasize that one set of data or results are better, worse, different, or similar are common examples of univariate statistical applications. However, even in these simple statistics cases, the claims are often not supported by 'significance of means' test. Likewise, simple bivariate correlation is routinely applied to data sets and scatter diagrams to show the 'goodness' of fit of data obtained from different sources or tests. A good relationship is often implied by the high correlation coefficient, but not always with a significance test. The term 'multivariate' implies that more than two variables, or in this case, test methods or samples are being compared. The technique is usually applied to a sample population of more than 30 samples that has undergone testing using three or more methods (variables).

Rock materials and concrete have certain properties in common: they have specific range of porosities, degree of saturation, density, compressive, tensile and impact strengths, hardness, and reactivity, among others. These properties combine to give the material a specific response to a given test. The multivariate statistical technique that finds the above groupings is FACTOR ANALYSIS. Factor analysis determines the degree to which clusters of inter-correlated variables may represent fewer underlying, more basic, hypothetical variables. In terms of rock testing, factor analysis will group those tests that produce similar response in the rocks tested, and will suggest which properties of the rock are involved in the response.

The multivariate statistical technique that groups samples rather than variables is called K-MEANS CLUSTER ANALYSIS. In practical terms, each cluster represents samples with similar properties. The properties are often dissimilar enough to enable the classification of the cluster; in rock or concrete terms, the classification can be as "excellent", "good", "fair", or "poor" aggregate. Theoretically, each new sample added to the model would fall within one cluster and be so classified. It is this method that was used to classify the samples in the two databases. The data was normalized to equalize the numerical disparity of the test results.

STATISTICAL SOFTWARE The statistical software used in the analysis of the data was Systat version 8, running on Windows 98. Systat is produced by SPSS Inc., and is a subset of the well-known SPSS statistical package. The data was imported from Excel spreadsheet into Systat, and the results of Systat analysis was transferred back to Excel and Quattro Pro spreadsheets for further analysis and graphing.

STATISTICAL ANALYSIS

MTO Data Set

The results of K-Means Cluster analysis on the MTO data set are presented in Table 1. Three groups or categories were chosen for the K-Means test to reflect the number of quality categories assigned by the MTO petrographers. The rock type and the quality categories are given in the first two columns, and the results of tests in subsequent columns.

Each group represents rocks that have similar responses in the tests, and that differ significantly from the rocks in the other groups. Table 2 gives the results of the t-test analysis between groups. It shows that rocks in Group 1 are all significantly different from those in group 2 and in Group 3 in all but one test. The rocks in Group 2 are mostly weathered, soft, and shaley. The rocks in Group 2 differ from those in Group 3 in adsorption and strength properties: Point Load and wet and dry impact. Note that Group 3 consists mostly of weak, friable rock types. The differences and similarities between groups are shown in Figure 1.

Note that the MTO quality classification transcends groups to a certain extent. In group 1, which contains high quality rock types, two quality or 'factor' '3' rock types are found. This suggests that their quality is perhaps higher than originally assigned. There is a mix of quality category rocks in groups 2 and 3. Group 3. Both of these groups can be considered as fair to poor material, distinctly and significantly different from the rocks found in Group 1. Rocks in Group 3, in addition to being less durable and sound, are also significantly weaker and friable.

Table 1. Results of K-Means Cluster Analysis, MTO Data Set

	Factor	Ads95RH	Abs24h	VacSat	Dens	PointLoad	FTloss	IMPdry	IMPwet
Group 1									
Gneiss/Schist. (Hard)	1	0.058	0.12	0.27	2.717	9	0.1	25	27
Crystalline Carbonate	1	0.175	0.04	0.37	2.787		1.9	22	26
Volcanic (Hard)	1	0.283	0.3	0.48	2.81	11.4	1	8	10
Sandstone (Hard)	1	0.262	0.82	1.29	2.636	4.29	5.1	16	29
Granite-Diorite-Gabbro	1	0.19	0.12	0.48	2.752	7.81	2.2	24	27
Carbonate (Med.Hard)	1	0.5	1.33	1.73	2.732	3.36	3.7	17	22
Quartzite	1	0.202	0.16	0.22	2.667	4.58	2.2	19	21
Sandy Carbonate (Hard)	1	0.75	0.48	0.96	2.631	2.79	2.4	21	25
Greywacke-Arkose	1	0.19	0.09	0.19	2.734	8.3	0.4	8	12
Trap (Basalt)	1	0.45	0.05	0.06	3.083	12.4	1.5	9	9
Carbonate (Hard)	1	0.375	0.55	1.2	2.717	4.53	3.4	19	21
Cherty Carbonate	3	0.39	1.62	1.76	2.622	6.02	3.2	17	17
Carbonates (Encrusted)	3	0.517		0.62	2.675		4.9	17	20
Mean		**0.33**	**0.47**	**0.74**	**2.74**	**6.77**	**2.46**	**17.08**	**20.46**
St. Dev.		**0.19**	**0.53**	**0.59**	**0.12**	**3.25**	**1.56**	**5.69**	**6.69**
Group 2									
Carbonate (Med.Hard)	1	0.957	0.2	1.84	2.69		8.9	19	22
Sandy Carbonate (Med.Hard)	1	0.98	2.12	2.32	2.605	3.33	4.7	21	17
Crystalline Carbonate	3	0.827	1.24	1.69		1	6.6	25	24
Carbonate (Slightly Shaley)	3	0.487	1.95	2.12	2.666	2.32	11.6	25	27
Carbonates (Deeply Weathered)	3	0.772	2.74	3.3	2.62	2.51	9.7	25	32
Sandy Carbonates (Soft)	3	0.662	3	3.42	2.525	2.49	7.1	25	28
Carbonates (Shaley)	6	1.087	3.15	4.2	2.586	2.13	12	30	33
Siltstone (Friable)	6	0.277	2.48	2.35	2.579	6.75	3.3	19	27
Chert (Leached)	6	0.22	4.23	5.08	2.467	4.55	8.2	14	13
Mean		**0.70**	**2.35**	**2.92**	**2.59**	**3.14**	**8.01**	**22.56**	**24.78**
St. Dev.		**0.31**	**1.16**	**1.16**	**0.07**	**1.78**	**2.93**	**4.75**	**6.59**
Group 3									
Sandstone (Med.Hard)	1	0.28	1.52	1.96	2.582	2.46	6.3	29	32
Granite (Brittle)	3	0.325	0.75	0.72	2.636	0.39	5.4	35	36
Gneiss (Brittle)	3	0.16	0.49	0.27	2.73	4.01	4.2	36	36
Sandstone (Friable)	6	0.572	3.19	4.32	2.483	0.54	10.4	46	41
Gneiss (Friable)	6	0.477	1.57	2.24	2.677	0.1	18.5	50	53
Mean		**0.36**	**1.50**	**1.90**	**2.62**	**1.50**	**8.96**	**39.20**	**39.60**
St. Dev.		**0.16**	**1.05**	**1.58**	**0.09**	**1.68**	**5.82**	**8.58**	**8.14**

Table 2. t-Test Results of date in Table 1. Bold characters indicate significant difference.

	Ads95RH	Abs24h	VacSat	Density	PointLoad	FTloss	IMPdry	IMPwet
T-Test between Group 1 (top) and Group 2 (middle)								
t Stat	-3.425	-4.964	-5.848	3.047	2.855	-5.792	-2.368	*-1.497*
P(T<=t) one-tail	0.001	0.000	0.000	0.003	0.005	0.000	0.014	*0.075*
t Critical one-tail	1.725	1.729	1.725	1.729	1.740	1.725	1.725	*1.725*
T-Test between Group 2 (middle) and Group 3 (bottom)								
t Stat	2.212	*1.337*	1.395	*-0.636*	1.644	*-0.413*	-4.743	-3.719
P(T<=t) one-tail	0.024	*0.103*	0.094	*0.269*	0.064	*0.344*	0.000	0.001
t Critical one-tail	1.782	*1.782*	1.782	*1.796*	1.796	*1.782*	1.782	1.782
T-Test between Group 1 (top) and Group 3 (bottom)								
t Stat	*-0.302*	-2.736	-2.346	1.901	3.383	-3.849	-6.431	-5.136
P(T<=t) one-tail	*0.383*	0.008	0.016	0.038	0.002	0.001	0.000	0.000
t Critical one-tail	*1.746*	1.753	1.746	1.746	1.761	1.746	1.746	1.746

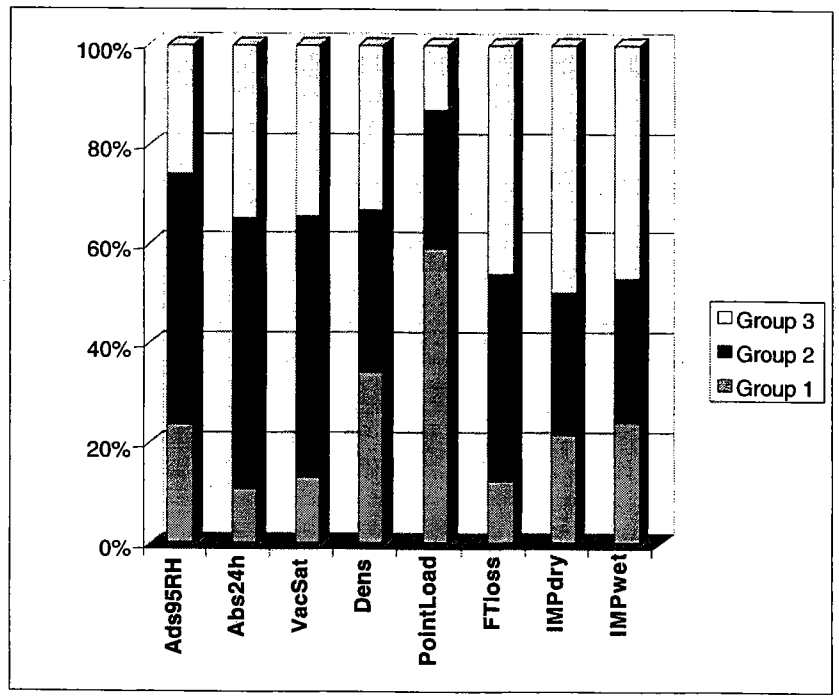

Figure 1. Illustration of the proportion of given property assigned to a group.

SLOVAK DATA SET

K-Means Cluster Analysis and t-test of group significance

The 430 samples of the Slovak data set were analyzed using the identical methods of K-means Cluster analysis as those applied to the MTO set. However, rather than three groups in the MTO set, four were stipulated for the Slovak set, considering the large size of the set. Table 3 shows the results of the analysis. The first part of the table shows the means and standard deviation of the four groups, and the remainder of the table shows the t-test of significance between groups 1 and 2, 1 and 3, and 3 and 4. Figure 2 illustrates the distribution of test results assigned to each of the groups.

Groups 1 and 2 are similar, and can be classed as a relatively good aggregate. The difference between them is largely in their density, and response to frost durability tests (freeze-thaw and Na sulfate). Group 1 has significantly lower freeze-thaw loss, and somewhat higher, though not significantly so, Na sulfate loss. Group 1 rocks can withstand severe environments of weathering. Group 2 rocks may be susceptible to breakdown if frozen in saturated state.

Groups 3 can be considered as fair to poor aggregate. The results of all tests are significantly worse than group 1, and 2 (t-test for group 2 and 3 were done, giving similar results, but are not shown in the table for lack of space). The rocks of Group 3 can be characterized as relatively soft, low density, large porosity rocks that break down relatively easily during freezing and thawing cycles. They may be suitable for use in non-severe, low abrasion, well-drained environments.

Group 4 rocks can be classed as generally poor aggregate. All the test results, with the exception of LA Abrasion, are poorer than those of Group 3, and Freeze-Thaw and Na Sulfate results significantly so. The rocks of this group should be used with caution.

Rock Type Distribution within the Groups

Table 4 gives the distribution of the rock types within each of the groups and Figure 3 illustrates their relative percentage in terms of the total for the rock type. It can be seen that the andesites, are the most abundant source rock, and are also a good quality rock. Conglomerate, dolomite, limestone, granite, granodiorite, quartzite, gneiss, and gravel are also a good quality rock. Basalt, basalt porphyry, porphyry, amphibolite, and greywacke are of somewhat lower quality. Among all of these rock types there are some that are of fair to poor quality, mainly because the rock may be tectonically disturbed or weathered.

Among the rocks that fall mostly into the fair to poor categories (3 and 4) are rhyolite and sandstone. Rhyolite is found here because the rock is quite often highly altered and soft; sandstone is relatively poorly cemented, with and has large porosity.

Table 3: Means, Standard Deviations, and t-Test Analyses of Groups in the MTO Data Set.

Locality	Polish End	LA Abras	microDeval	FrzThw200	NaSul15	Absorb	Porosity	DensSSD	DensApp
Group 1 Mean	**0.47**	**21.96**	**13.68**	**7.06**	**17.19**	**1.25**	**3.14**	**2.71**	**2.62**
St.Dev. 1	0.04	5.14	6.44	3.70	13.03	0.53	1.92	0.05	0.07
Group 2 Mean	**0.51**	**17.34**	**13.64**	**11.71**	**12.67**	**1.23**	**2.98**	**2.55**	**2.51**
St.Dev. 2	0.05	3.55	7.80	6.24	9.10	0.48	1.84	0.04	0.06
Group 3 Mean	**0.60**	**37.45**	**33.68**	**22.57**	**54.13**	**3.40**	**10.40**	**2.67**	**2.40**
St.Dev. 3	0.071	12.842	19.874	8.722	23.981	1.337	4.581	0.092	0.161
Group 4 Mean	**0.575**	**28.592**	**34.120**	**66.300**	**86.675**	**4.135**	**11.925**	**2.695**	**2.365**
St. Dev. 4	0.053	3.546	7.805	6.235	9.099	0.481	1.841	0.042	0.062

Group 1 and Group 2									
	Polish End	LA Abras	microDeval	FrzThw200	NaSul15	Absorb	Porosity	DensSSD	DensApp
t Stat	**-7.851**	**6.501**	*-0.003*	**-8.525**	*1.153*	*0.564*	*0.662*	**-16.124**	**-11.513**
P(T<=t) one-tail	**0.000**	**0.000**	*0.499*	**0.000**	*0.126*	*0.287*	*0.255*	**0.000**	**0.000**
t Critical one-tail	**1.651**	**1.651**	*1.658*	**1.649**	*1.660*	*1.659*	*1.658*	**1.658**	**1.658**

Group 1 and Group 3									
	Polish End	LA Abras	microDeval	FrzThw200	NaSul15	Absorb	Porosity	DensSSD	DensApp
t Stat	**-15.190**	**-11.165**	**-7.818**	**-19.895**	**-8.210**	**-10.713**	**-10.789**	**2.392**	**9.229**
P(T<=t) one-tail	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.009**	**0.000**
t Critical one-tail	**1.653**	**1.653**	**1.658**	**1.650**	**1.660**	**1.659**	**1.658**	**1.658**	**1.658**

Group 3 and Group 4									
	Polish End	LA Abras	microDeval	FrzThw200	NaSul15	Absorb	Porosity	DensSSD	DensApp
t Stat	*0.950*	***1.605***	*-0.046*	**-12.354**	**-2.561**	*-0.902*	*-0.592*	*-0.496*	*0.349*
P(T<=t) one-tail	*0.173*	***0.059***	*0.482*	**0.000**	**0.011**	*0.191*	*0.281*	*0.313*	*0.366*
t Critical one-tail	*1.677*	***1.697***	*1.740*	**1.672**	**1.761**	*1.761*	*1.746*	*1.746*	*1.746*

*** Results in bold indicate significant difference between groups.**

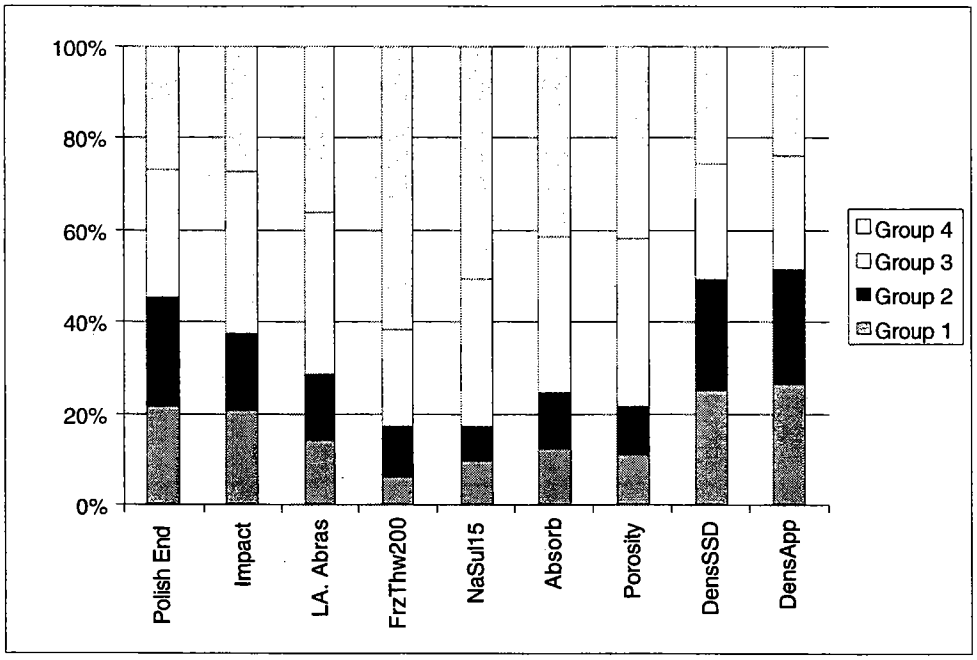

Figure 2. Distribution of test results among the four groups.

Table 4. Rock type distribution among the four groups.

ROCK TYPE	Group 1	Group 2	Group 3	Group 4	TOTALS
amphibolite	6	7	1		14
andesite	76	25	8	3	112
basalt	12	14		1	27
basalt porph	11	15			26
conglomerate	6	2			8
diorite	8	2	1		11
dolomite	14	3	1	2	20
granite	8	2			10
granodiorite	16	1	1		18
gravel	6	1			7
greywacke	10	12	4		26
limestone	35	5	3	1	44
orthogneiss	11	1	2		14
paragneiss	19	3			22
porphyry	5	5			10
quartzite	8	1			9
rhyolite	1	4	5	4	14
sandstone	4	8	21	2	35
TOTALS	256	111	47	13	427

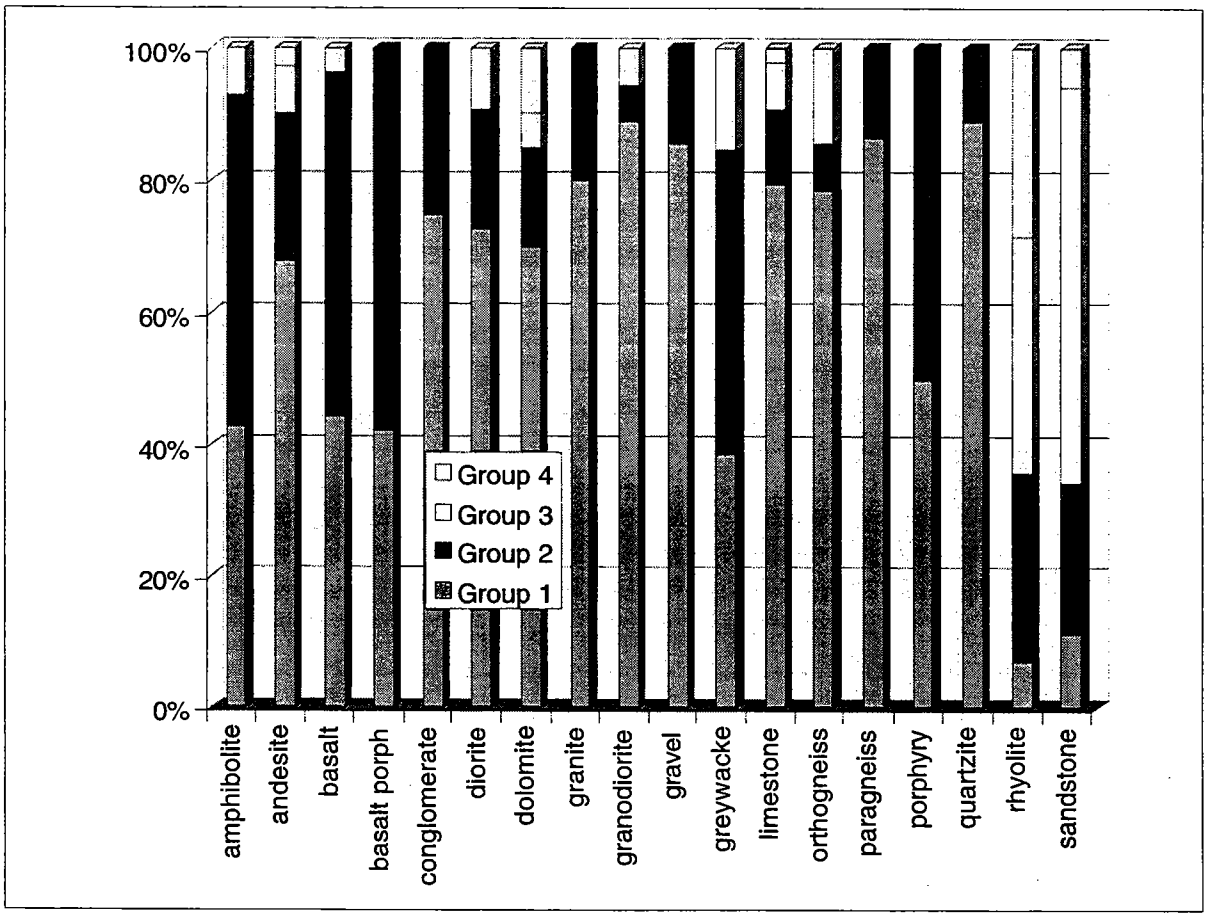

Figure 3. Distribution of rock types within the four groups, expressed as percent of total of each rock type.

DISCUSSION

The multivariate statistical techniques are useful in classifying rock types based on several tested properties. The K-means cluster analysis, in particular, is able to group rock types into affinity groups in which the rock have similar properties. Statistical classification of this type is preferable to manual rock classification based on any one or a combination of tests alone. It is particularly useful if rock types of know service record or properties are part of the database. The placement of these into a particular category identifies the other, less well-known rocks found in the same category as having similar properties, and therefore similar behavior. In similar way, the quarry or sample locality can be identified and classified; the groups of quarries producing lower quality rock can be related to regional geologic, structural or weathering patterns or features.

CONCLUSIONS

One of the most useful multivariate statistical techniques for classifying rocks is K-means Cluster Analysis. When applied to the two rock sets described above, the analysis produced the following results:

MTO data set:

- The analysis was able to clearly separate rock high quality rock types from those of lower quality.
- The lower quality rocks were further subdivided into weak and friable category.
- The grouping produced suggested that the quality designation used by MTO for the lower quality rock types needs some adjustment.

Slovak data set:

- The analysis divided the rocks into two acceptable and two fair to poor groups. Quality estimate is based on the overall test results.
- The two good quality groups differ only slightly, mostly in their density and LA abrasion, which in turn affect their freeze-thaw behavior.
- The two poorer groups differ mostly in the degree of durability; the last group being distinctly less durable than any of the other groups.
- The procedure identifies rock types that populate the four groups, and their location or quarries.
- The information can be used to imply regional geologic controls that are responsible for rock quality.

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Deleterious Constituents in Indiana Aggregates: A Challenge for Non-Geologists to Identify

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Abstract

Aggregates for highway construction in Indiana are obtained from two basic sources: stream gravels, and crushed stone from quarries in carbonate rocks. Blast furnace slag and fine sandstone (from outside the state) contribute a much smaller quantity. The stream gravels are valley train deposits formed by outwash from Pleistocene glaciers. A complete petrographic analysis of a typical gravel sample indicates that about 65% of the constituents are sedimentary (and mostly native to Indiana), whereas the remainder are igneous/metamorphic rocks transported from the north by glaciers.

Deleterious materials designated by INDOT Specifications include friable particles, clay lumps, soft sandstone, shale, limonite concretions, coal, weathered schist, cemented gravels, ocher, shells, wood and other objectionable materials. Chert with low specific gravity (less than 2.45) is also considered to be detrimental. A maximum allowable percentage is designated in the Specifications based on the aggregate use classification.

Indiana has recently installed the CAPP program, Certified Aggregate Producer Program. Technicians from current aggregate producers and INDOT districts are certified to determine whether materials continue to qualify under state specifications. One of the tasks that technicians must perform is the test for deleterious constituents in aggregates. Training of technicians to identify deleterious constituents is an on-going process, taught by INDOT, the state mineral aggregates association and geologists from state universities (includes T.R. West) and other agencies. Details of the training program are provided in the paper.

Introduction

Natural aggregates used in Indiana consist of crushed carbonate rocks and glacial river gravels. Typically, gravels in the State are composed of 50 to 85% carbonates, 1 to 8% other sedimentary rocks, 8 to 35% igneous and 3 to 17% metamorphics. The combined igneous and metamorphic portions tend to be stronger and more durable than the sedimentary fraction. For the carbonates, dolomite usually is a higher quality than is limestone. Rocks that comprise Indiana aggregates are related to glaciation and bedrock geology (textbook by West, 1995(1), Chapters 3-6, 11, 12; Wayne, 1956).

Blast furnace slag is a by-product of the steel making industry in Indiana. A small quantity of fine, quartz-sandstone is shipped in from Illinois for specialized road construction. Both slag and sandstone have a higher friction resistance than carbonates; this resistance is needed for bituminous overlays on high traffic volume roads. They provide added frictional resistance for wet, bituminous pavements (Shakoor and West, 1979).

Approximately 2/3 of northern Indiana has been glaciated by a series of Pleistocene ice advances. Wisconsinian age deposits pre-dominate, but near the glacial boundary to the south,

areas of older, Illinoisian deposits are exposed at the ground surface. Bedrock in southern Indiana consists of Paleozoic sedimentary rocks ranging in age from Ordovician (on the east) to Pennsylvanian (on the west). The rocks dip gently toward the Illinois Basin yielding progressively younger rocks from east to west across the State (Camp and Richardson, 1999).

Crushed stone is obtained from the carbonate bedrock units consisting of limestone and dolomite. Older Paleozoic rocks (Ordovician and Silurian) tend to be dolomitic, whereas for younger Paleozoics (Devonian and Mississippian), limestones predominate (Deiss, 1952).

Glacial gravels occur in valley train deposits which comprise the flood plains of the major rivers in Indiana. These streams extend from north to south, originating in the glaciated area, but continuing into the nonglaciated portion to the south. Gravels are used to supply aggregates for concrete and hot mix asphalts (HMA) as well, as for base courses and general fill material (West and others, 1970).

Both crushed stone, consisting of dolomite or limestone, and stream gravels are used as highway construction materials. The quality of these aggregates is evaluated based on the current specifications for INDOT, 1999 (Indiana Department of Transportation).

Acknowledgements

Information presented here is based on the author's knowledge and experience gained while conducting research on Indiana aggregates (for example, report by Bruner and others, 1995; West and others, 1999). It is strictly the author's interpretation of available information and not necessarily that of the Indiana Department of Transportation.

Determination of Aggregate Quality

INDOT requirements for coarse aggregate are shown in Table 1, Classification of Aggregates. Coarse aggregate is defined as aggregate that has a minimum of 20% retained on the 4.75 mm sieve. Crushed stone, crushed or uncrushed gravel, and crushed blast furnace slag and steel slag, all fall within this category.

Referring to Table 1, the characteristic classes are designated: AP, A, B, C, D, E and F. AP is required for portland cement concrete constructed on grade. This includes pavements and other flat work construction. Class A is acceptable for all INDOT uses including exterior concrete except when AP is specified. Classes B, C, D, E and F are aggregates having restricted use. Classes A or B are acceptable for portland cement concrete bases and Classes A, B, C or D are acceptable as hot mix asphalt (HMA) base courses. Refer to Table 2 for further details.

The quality requirements are listed on the left side of Table 1. Physical test methods are 1) Freeze-and-Thaw Beam Expansion, 2) Los Angeles Abrasion, 3) Sodium Sulfate Soundness, 4) Brine Freeze-and-Thaw Soundness, and 5) Absorption (ASTM Standard Specifications, 1999 and INDOT, 1999). Maximum allowable percentages are listed for each characteristic class, AP, A, B, C, D, E and F.

In the second part of Table 1, additional requirements are listed. The maximum allowable deleterious materials are designated. These relate to the primary purpose of this paper (evaluation of aggregate quality) and are considered in detail in a subsequent section. They are the constituents that non-geologists are required to recognize in an evaluation of a crushed stone or gravel sample.

Table 1. Requirements for INDOT Classes of Aggregates (after INDOT Specifications, 1999).

Characteristic Classes	AP	A	B	C	D	E	F
Quality Requirements							
Freeze-and-Thaw Beam Expansion, % Max. (Note 9)	.060						
Los Angeles Abrasion, %, Max. (Note 1)	40.0	40.0	40.0	45.0	45.0	50.0	
Sodium Sulfate Soundness, %, Max. (Note 2	12.0	12.0	12.0	16.0	16.0	20.0	25.0
Brine Freeze-and-Thaw Soundness, % Max. (Note 8)	30	30	30	40	40	50	60
Absorption, %, Max. (Note 3)	5.0	5.0	5.0	5.0			
Additional Requirements							
Deleterious, %, Max.							
Clay Lumps and Friable Particles	0.2	0.2	0.2	0.2	0.2		
Non-Durable (Note 4)	4.0	4.0	4.0	6.0	8.0		
Coke				(See	Note 7)		
Iron				(See	Note 7)		
Chert (Note 5)	3.0	3.0	5.0	8.0	10.0		
Mass Per Cubic Meter for Slag, kg,	1200	1200	1200	1120	1120	1120	
Weight Per Cubic Foot for Slag, (lbs), Min.	(75.0)	(75.0)	(75.0)		(70.0)	(70.0)	
Crushed Particles, %, Min. (Note 6)							
Asphalt Seal Coats		70.0	70.0				
Compacted Aggregates		20.0	20.0	20.0	20.0		

NOTES:
1. Los Angeles abrasion requirements shall not apply to blast furnace slag.
2. Aggregates may, at the option of the Engineer, be subjected to 50 cycles of freezing and thawing in accordance with AASHTO T 103, Procedure A, and may be accepted, provided they do not have a loss greater than specified for Sodium Sulfate Soundness.
3. Absorption requirements apply only to aggregates used in portland cement concrete and HMA mixtures except they shall not apply to blast furnace slag. When crushed stone coarse aggregates from Category I sources consist of production from ledges whose absorptions differ by more than two percentage points, the absorption test will be performed every three months on each size of material proposed for use in portland cement concrete or HMA mixtures. Materials having absorption values between 5.0 and 6.0 that pass AP testing may be used in portland cement concrete. If variations in absorption preclude satisfactory production of portland cement concrete or HMA mixtures, independent stockpiles of materials will be sampled, tested, and approved prior to use.
4. Non-durable particles include soft particles as determined by ITM 206 and other particles which are structurally weak, such as soft sandstone, shale, limonite concretions, coal, weathered schist, cemented gravel, ocher, shells, wood, or other objectionable material. Determination of non-durable particles shall be made from the total mass (weight) of material retained on the 9.5 mm (3/8 in.) sieve. Scratch Hardness Test shall not apply to crushed stone coarse aggregate.
5. The bulk specific gravity of chert shall be based on the saturated surface dry condition. The amount of chert less than 2.45 bulk specific gravity, shall be determined on the total mass (weight) of material retained on the 9.5 mm (3/8 in.) sieve for sizes 1 through 8, 53, and 91 and on the total mass (weight) of material retained on the 4.75 mm (No. 4) sieve for sizes 9 and 11.
6. Crushed particle requirements will apply to gravel coarse aggregates used in HMA mixtures, compacted aggregates, and asphalt seal coats except seal coats used on shoulders. Crushed particle requirements for HMA mixtures are set out in 904.02(c). Determination of crushed particles shall be made in accordance with ASTM D 5821.
7. Air-cooled blast furnace slag and steel slag coarse aggregate shall be free of objectionable amounts of coke and iron.
8. Brine freeze-and-thaw soundness requirements are subject to the conditions stated in note 2.
9. Freeze-and-thaw beam expansion shall be tested and retested in accordance with ITM 210.

Table 2. INDOT coarse Aggregates versus Type of Construction (after INDOT Specifications, 1999).

CLASS OF COARSE AGGREGATES REQUIRED FOR VARIOUS TYPES OF CONSTRUCTION	
TYPE OF CONSTRUCTION	***REQUIRED QUALITY CLASSIFICATION***
Compacted aggregate base, surface and shoulder	Class A, B, C, or D
Portland cement concrete base	Class A or B
Sodium chloride stabilized aggregate base coarse	Class A, B, C, or D
HMA base course	Class A, B, C, or D
HMA intermediate course	Class A, B, or C
HMA surface course	Class A or B
Asphalt seal coat	Class A or B
Portland cement concrete pavement	Class AP
Portland cement concrete structural– exposed	Class A
Portland cement concrete structural– non-exposed	Class A or B
Cover (choke) aggregates coarse aggregate	Class A or B

Coarse aggregate size gradations for INDOT are shown in Table 3. Ten sizes or gradations are designated, Numbers 1, 2, 5, 8, 9, 11, 12, 53, 73 and 91. For Numbers 1 through 12, the small number indicates a larger size distribution, and for Number 53 to 91, the smaller the number the larger the size distribution. Aggregate gradations are obtained by aggregate producers at their plant site and placed in stockpiles according to these gradation numbers. For example, Number 8s are required for concrete aggregates and Number 11s for surface courses of HMA pavement overlays. Numbers 53 and 73 are compacted aggregates and Number 91s are used for aggregates in pre-cast concrete. In Table 4 the size gradations for different types of construction are listed.

Four sizes or gradations are designated for fine aggregates, Numbers 23, 24, 15 and 16. These gradations are shown in Table 5. Fine aggregate is defined as material that has 100% passing the 9.5 mm sieve (3/8 inch) and a minimum of 80% passing the 4.75 mm sieve (No 4 sieve). Referring to Table 4, Number 23s are required for a portland cement concrete base and as aggregate for the portland cement concrete itself. For an asphalt seal coat (HMA), Number 23s or 24s are required.

Deleterious Materials

For natural aggregates the deleterious materials that occur in Indiana are 1) clay lumps and friable particles, 2) non-durable particles, and 3) low density chert. The maximum allowable percentages of these constituents are listed in Table 1 in regard to the different Aggregate Classes (AP through F).

Clay lumps and friable particles are materials that are easily crumbled or mashed with the fingers. Non-durable particles consist of two types; the first being soft materials as determined by ITM 206, Scratch Hardness of Coarse Aggregates (ITM-206, 1998). The second consists of structurally weak materials as determined by visual inspection. Structurally weak materials include 1) conglomerate and cemented gravels, 2) soft sandstone, 3) shale or laminated rock of clay sized minerals, 4) limonite, an iron oxide brown to black in color, frequently occurring as a concretion around a soft core, 5) weathered schist that is structurally weak, 6) ocher, a soft clay to sand cemented with iron oxide, 7) fresh water shells that is, unfossilized clam shells, 8) coal, wood and other foreign materials, and 9) materials with loosely cemented grains and/or a weathered coating.

In addition, there is a maximum allowable percentage of chert which has a bulk specific gravity less than 2.45. For exposed concrete and concrete pavements (classes A and AP respectively), a maximum of 3% of this low density chert is allowed. The reason for this limitation is that low density chert in exposed concrete is subject to freeze-and-thaw damage in the severe Indiana climate. Popouts occur in low density chert, when exposed at the surface where they undergo freezing and thawing cycles under saturated conditions. Other low density particles can cause popouts when exposed at the surface of exterior concrete. The nine weak materials discussed in the previous paragraph are subject to the popout problem.

Chert, a sedimentary variety of SiO_2, has a specific gravity similar to quartz when the chert is unweathered. This value of 2.65 is reduced by the formation of pores and pits in the chert surface during weathering, eventually yielding a value below 2.45.

Specifications for coarse aggregates in portland cement concrete are also designated according to ASTM Standards (ASTM, C33, 1999). These ASTM specifications are shown in Table 6. For Indiana concrete pavements, category 4S would apply. A comparison between

Table 3. INDOT Coarse Aggregate Gradations (after INDOT Specifications, 1999).

Sieve Sizes	COARSE AGGREGATE SIZES (PERCENTS PASSING)									
	1	2	5	8	9	11	12	53[1]	73[1]	91
100 mm (4 in.)	100									
90 mm (3 1/2 in.)	90-100									
63 mm (2 1/2 in.)	25-60	100								
50 mm (2 in.)		95-100								
37.5 mm (1 1/2 in.)	0-15		100					100		
25 mm (1 in.)		0-20	85-98	100				80-100	100	100
19 mm (3/4 in.)	0-5	0-5	60-85	75-95	100			70-90	90-100	
12.5 mm (1/2 in.)		0-2	30-60	40-70	60-85	100	100	55-80	60-90	
9.5 mm (3/8 in.)			15-45	20-50	30-60	75-95	95-100			
4.75 mm (No. 4)			0-15	0-15	0-15	10-30	50-80	35-60	35-60	
2.36 mm (No. 8)			0-10	0-10	0-10	0-10	0-35	25-50		
600 μm (No. 30)							0-4	12-30	12-30	
75 μm (No. 200)(2)								5.0-10.0	5.0-10.0	
Decant Conc[3]			0-1.5	0-1.5	0-1.5	0-1.5				0-1.5
Other	0-1.0	0-2.5	0-2.5	0-3.0	0-2.5	0-2.5	0-2.0			0-2.5

NOTES:
1. The fraction passing the 75 μm (No. 200) sieve shall not exceed 2/3 the fraction passing the 600 μm (No. 30) sieve. The liquid limit shall not exceed 25 (35 if slag) and the plasticity index shall not exceed 5. The liquid limit shall be determined in accordance with AASHTO T 89 and the plasticity index in accordance with AASHTO T 90. Unless otherwise specified, when these materials are not to be surfaced or sealed under the contract, the amount passing the 75 μm (No. 200) sieve shall be 5% to 12% and the plasticity index shall not exceed 7.
2. Includes the total amount passing the 75 μm (No. 200) sieve as determined by AASHTO T 11 and T 27.
3. When the material is stone or slag, the decant may be 0 to 2.5.

Table 4. INDOT Aggregate Gradations Related to Type of Construction (after INDOT Specifications, 1999).

TYPE OF CONSTRUCTIONREQUIREMENTS	GRADATION
Compacted aggregate base * Coarse aggregate	 No. 53
Compacted aggregate surface and shoulders * Coarse aggregate	 No. 73
Portland cement concrete base * Fine aggregate * Coarse aggregate	 No. 23 No. 8
Calcium chloride stabilized Aggregate base course * Coarse aggregate	 Nos. 73
Asphalt Seal Coat * Fine Aggregate * Coarse Aggregate	 Nos. 23 or 24 Nos. 8, 9, 11, 12
Portland cement concrete pavement/structural * Fine aggregate * Coarse aggregate	 No. 23 No. 8

Table 5. INDOT Fine Aggregate Gradations (after INDOT Specifications, 1999).

Sieve Sizes	SIZES (PERCENTS PASSING)			
	23	24	15	16
9.5 mm (3/8 in.)	100	100		
4.75 mm (No. 4)	95-100	95-100		
3.35 mm (No. 6)			100	
2.36 mm (No. 8)	80-100	70-100	90-100	
1.18 mm (No. 16)	50-85	40-80		
600 μm (No. 30)	25-60	20-60	50-75	100
300 μm (No. 50)	5-30	7-40	15-40	
180 μm (No. 80)				95-100
150 μm (No. 100)	0-10	1-20	0-10	
75 μm (No. 200)	0-3	0-6	0-3	65-100

Note: The fine aggregate shall have not more than 45% retained between any 2 consecutive sieves.

Table 6. Requirements for Portland Cement Concrete Aggregates (after ASTM, C33).

Class Designation	Type or Location of Concrete Construction	Maximum Allowable, %						
		Clay Lumps and Friable Particles	Chert[C] (Less Than 2.40 sp gr SSD)	Sum of Clay Lumps, Friable Particles, and Chert (Less Than 2.40 sp gr SSD)[C]	Material Finer Than No. 200 (75-μm) Sieve	Coal and Lignite	Abrasion[A]	Magnesium Sulfate Soundness (5 cycles)[B]
		Severe Weathering Regions						
1S	Footings, foundations, columns and beams not exposed to the weather, interior floor slabs to be given coverings	10.0	...	...	1.0[D]	1.0	50	...
2S	Interior floors without coverings	5.0	...	...	1.0[D]	0.5	50	...
3S	Foundation walls above grade, retaining walls, abutments, piers, girders, and beams exposed to the weather	5.0	5.0	7.0	1.0[D]	0.5	50	18
4S	Pavements, bridge decks, driveways and curbs, walks, patios, garage floors, exposed floors and porches, or waterfront structures, subject to frequent wetting	3.0	5.0	5.0	1.0[D]	0.5	50	18
5S	Exposed architectural concrete	2.0	3.0	3.0	1.0[D]	0.5	50	18
		Moderate Weathering Regions						
1M	Footings, foundations, columns, and beams not exposed to the weather, interior floor slabs to be given coverings	10.0	...	...	1.0[D]	1.0	50	...
2M	Interior floors without coverings	5.0	...	...	1.0[D]	0.5	50	...
3M	Foundation walls above grade, retaining walls, abutments, piers, girders, and beams exposed to the weather	5.0	8.0	10.0	1.0[D]	0.5	50	18
4M	Pavements, bridge decks, driveways and curbs, walks, patios, garage floors, exposed floors and porches, or waterfront structures subject to frequent wetting	5.0	5.0	7.0	1.0[D]	0.5	50	18
5M	Exposed architectural concrete	3.0	3.0	5.0	1.0[D]	0.5	50	18
		Negligible Weathering Regions						
1N	Slabs subject to traffic abrasion, bridge decks, floors, sidewalks, pavements	5.0	...	...	1.0[D]	0.5	50	...
2N	All other classes of concrete	10.0	...	...	1.0[D]	1.0	50	

[A] Crushed air-cooled blast-furnace slag is excluded from the abrasion requirements. The rodded or jigged unit weight of crushed air-cooled blast-furnace slag shall be not less than 70 lb/ft^3 (1120 kg/m^3). The grading of slag used in the unit weight test shall conform to the grading to be used in the concrete. Abrasion loss of gravel, crushed gravel, or crushed stone shall be determined on the test size or sizes most nearly corresponding to the grading or gradings to be used in the concrete. When more than one grading is to be used, the limit on abrasion loss shall apply to each.

[B] The allowable limits for soundness shall be 12 % if sodium sulfate is used.

[C] These limitations apply only to aggregates in which chert appears as an impurity. They are not applicable to gravels that are predominantly chert. Limitations on soundness of such aggregates must be based on service records in the environment in which they are used.

[D] This percentage may be increased under either of the following conditions: (*1*) if the material finer than the No. 200 (75-μm) sieve is essentially free of clay or shale the percentage may be increased to 1.5; or (*2*) if the source of the fine aggregate to be used in the concrete is known to contain less than the specified maximum amount passing the No. 200 (75-μm) sieve (Table 1) the percentage limit (L) on the amount in the coarse aggregate may be increased to $L = 1 + [(P)/(100 - P)]\ (T - A)$, where P = percentage of sand in the concrete as a percent of total aggregate, T = the Table 1 limit for the amount permitted in the fine aggregate, and A = the actual amount in the fine aggregate. (This provides a weighted calculation designed to limit the maximum mass of material passing the No. 200 (75-μm) sieve in the concrete to that which would be obtained if both the fine and coarse aggregate were supplied at the maximum tabulated percentage for each of these ingredients.)

[E] Weathering Index. The effect of weathering is related to the weathering index, which for any locality is the product of the average annual number of *freezing cycle days* and the average annual *winter rainfall* in inches (or centimetres), defined as follows: *A Freezing Cycle Day* is any day during which the air temperature passes either above or below 32°F (0°C). The average number of freezing cycle days in a year may be taken to equal the difference between the mean number of days during which the minimum temperature was 32°F (0°C) or below and the mean number of days during which the maximum temperature was 32°F (0°C) or below. *Winter Rainfall* is the sum, in inches (or centimetres) of the mean monthly corrected precipitation (rainfall) occurring during the period between and including the normal date of the first occurrence of freezing (32°F, 0°C) in the fall and the normal date of the last occurrence of freezing (32°F, 0°C) in the spring. The winter rainfall for any period is equal to the total precipitation less one tenth of the total fall of snow, sleet, and hail. Rainfall for a portion of a month is prorated.

Table 1 and Table 6 indicates that in most situations, INDOT specifications are somewhat more stringent, for example Los Angeles Abrasion, Sodium Sulfate Soundness and Absorption. Also of interest is a comparison of the deleterious category. For ASTM C33, clay lumps, friable particles and non-durable materials are combined and a single total allowable of 5% is designated. This is considerably less stringent than the INDOT specifications with the 0.2% limitation on clay lumps and friable particles. Also note that the specific gravity for low density chert is different between the two specifications. In ASTM C-33, a density of 2.40 rather than 2.45 is indicated. Also, the allowable percentage of chert is greater under ASTM C-33, 5% rather than 3%.

As a further concern, the total amount of low density chert plus soft and non-durable particles is limited to 5% under ASTM C33. No such combined value is designated by INDOT Specifications. For ASTM this yields an additional condition for aggregate quality.

Evaluation of Deleterious Constituents by Non-Geologists

Conducting a complete, megascopic evaluation of crushed stone or gravel samples is a challenging exercise for a professional geologist until he/she becomes familiar with the procedure. Identification of rocks of hand specimen or cobble size is far easier than accomplishing full identification of aggregates that are less than 25 mm (1 inch), ranging to the No. 4 sieve (4.75 mm) size (Federal Highway Administration, 1991). The author has given this assignment to students in his Petrography of Aggregates class over the past 30 years and never fails to observe how tedious and difficult it is for students to extend rock identification skills to rock particles so much smaller than hand specimen size. A complete petrographic, megascopic evaluation of an Indiana gravel sample is provided in Table 7 as an example. The petrographic procedure to accomplish this complete evaluation is discussed in detail in the paper by West, 1995(2).

INDOT has recently instituted the CAPP, Certified Aggregate Producer Program. Technicians from aggregate producing companies and INDOT districts are certified to determine whether materials continue to qualify under state specifications. One of the tasks that technicians must perform is the test for deleterious constituents in aggregates.

Referring again to Table 1, the deleterious materials involved are: clay lumps, friable particles, soft sandstone, shale, limonite concretions, coal, weathered schist, cemented gravel, ocher, shells, wood and other objectionable materials. Performing this task can be a challenge for a trained geologist with limited experience in the identification of small sized rock pieces. For a non-geologist, the process is a much greater challenge. In a heterogeneous sample such as a stream gravel, a sizeable amount must be included to obtain a representative evaluation. ASTM specifications call for a minimum of 200 to 300 pieces in the evaluation sample (ASTM, 295, 1999).

First we consider the chert category. To begin the process, all chert particles must be identified and set aside from the total sample. For the following size gradations, all particles retained on the 9.5 mm (3/8") sieve are evaluated: Nos 1, 2, 5, 8, 12, 53, 73 and 91. For gradations No. 9 and 11, all particles retained on the 4.75 mm (No. 4 sieve) are evaluated.

It is not a simple process to distinguish chert from limestone, dolomite or fine, hard siltstone. A useful procedure is to test the hardness of individual aggregate pieces using a steel nail. Chert is harder than steel, whereas limestone and dolomite are softer than steel. Hence, the nail will leave a streak of steel on the chert, but will scratch the limestone or dolomite. Hard

siltstone typically is also harder than steel and will show a steel streak much like chert. Other properties of the chert are used to distinguish it from siltstone. Chert is aphanitic (very fine grained) and shows conchoidal fracture (circular breaks), whereas siltstones typically differ in both these characteristics. In the author's experience, the beginning petrographer tends to designate a much greater percentage of chert particles in a gravel than is actually present. Fine limestones and dolomites are commonly mistaken for chert if consistent use of the steel nail is not followed.

After all chert constituents have been segregated from the sample, they are checked for low specific gravity. The chert is placed in heavy liquid having a specific gravity of 2.45. Particles less than 2.45 will float and those with a greater specific gravity will sink in the heavy liquid. The floating portion comprises the low density constituents which are compared to the 3% allowable for AP and A classes.

Distinguishing clay lumps and friable particles should not be difficult in a thorough inspection. These pieces will crumble when compressed and only the weakest of materials fall into this category. Non-durable particles are a greater challenge to distinguish. Soft sandstone and shale must be identified. Shales typically are thinly bedded and have a flat shape. Equi-dimensional, hard siltstones should not be included in the shale category. Limonite concretions and ocher are readily identified as is wood and coal. Weathered schist (and other weathered rock types) are another matter. It is the deeply weathered rocks with a lower specific gravity that are properly singled out. They should be pieces weak enough to fail in freeze-and-thaw much like low density cherts do. Rounded, slightly weathered rocks, so common in Indiana gravels, do not fall in this category. Non durable, deleterious rocks should be obviously quite low in compressive strength.

Modern day shells of freshwater clams are soft, weak particles and are considered to be deleterious. Invertebrate fossils found in limestones and dolomites do not qualify as deleterious shells. Clams, brachiopods, coral, sponges and bryozoons are common fossils which in themselves are not deleterious particles. The scratch hardness test ITM, No 296-98T is applied as an aid to determine non-durable particles in gravels (INDOT, 1998). It is used on materials retained on the 9.5 mm (3/8 inch) sieve. A brass rod, 1.6 mm (1/16 inch) in diameter with a rounded point is used to scratch rock particles. The rod will scratch a U.S. Lincoln copper penny, but fail to scratch a U.S. Jefferson nickel. A copper penny has the hardness of 3 on the Mohs hardness scale. The scratch hardness test shall be conducted on gravel samples only and not on crushed stone.

Apparently there is a tendency for non-geology trained technicians to consider some slightly weathered rock pieces to be non-durable when such is not the case. A rock particle should be weak and deeply weathered for it to be considered as non-durable. Gravel particles can be subdivided into three categories: satisfactory, fair and poor. Only the poor portion should be considered as non-durable. Refer to Table 7 showing the megascopic, petrographic evaluation of an Indiana gravel sample. Those rocks listed as poor are the deeply weathered constituents and would qualify as non-durable particles. For the gravel sample described in Table 7, non durable particles total 2.7% which is less than the maximum allowable of 4% for INDOT requirements for AP, A and B classes (Table 1). However, the deeply weathered chert at 3.9% exceeds the allowable 3% for INDOT. Relative to Table 6, ASTM C33, the total of 2.7 plus 3.9 or 6.6% exceeds the maximum allowable total of 5% for the 4S class designation.

Table 7. Megascopic Petrographic Evaluation, Northern Indiana Gravel, 12.5-9.5 mm (1/2-3/8 inch) Fraction (after West, 1995(2)).

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

Conclusions

The training of technicians to identify deleterious constituents is an on-going process. Short courses are given for CAPP training where all aspects of the certification process are reviewed. On occasion, specialty courses on the identification of deleterious materials are taught for CAPP inspectors by experts from INDOT, the State Mineral Aggregates Association, and geology professors from state universities in Indiana, who are experienced in aggregate studies. Rock identification is presented along with actual examples of deleterious materials to provide a hands-on experience for the technicians. The author, with his experience in teaching a specialty course in petrography of aggregates for 20 years or so, serves as one of the instructors for this short course.

Background on the physical properties of aggregates is provided in the course. The relationship between deleterious materials in aggregates and distress of portland cement concrete is discussed. The origin of rocks and their relationship to surficial and bedrock geology are presented. Finally, the subtleties of rock identification are pursued. Distinguishing chert from limestone or dolomite is considered and how weathered, low density chert looks different from sound, unweathered chert. Pieces of soft rocks are examined to illustrate the difference between moderately weathered, rounded rock fragments, which have considerable compressive strength and a truly weak soft and non-durable particle.

This training course for CAPP inspectors related to deleterious materials in aggregates provides a valuable service to individuals untrained in the geological sciences. It is a major undertaking, however. As stated early in this paper, rock identification of gravel and crushed stone particles 25 mm (1 inch) to 4.75 mm (No 4 sieve) in size is a challenging undertaking for geology graduates applying their coursework in mineralogy and petrology. For non geologists who have not observed the range of rock types and weathering conditions, identification of deleterious particles in natural aggregates is a major undertaking.

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USE OF A GEOGRAPHICAL INFORMATION SYSTEM AS A SLOPE MANAGEMENT TOOL

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Abstract

Aside from pavement maintenance, the rehabilitation and repair of highway slopes represents one of the single largest maintenance expenditures for many state DOTs. These major expenditures could be significantly reduced if proper preventive measures are taken during design and construction, or, at least, at some time before slopes fail. The evaluation of slope safety is normally based on some form of stability analysis, which unfortunately, requires vast quantities of field and laboratory data. The generation of this data makes the safety analysis of slopes, on a global basis, cost prohibitive for DOTs. A more cost efficient procedure for predicting slope stability, on a global basis, is to develop a screening tool, which would rank regions by instability potential. The ranking system should utilize index parameters that are already in existence or which require only minimal collection efforts. Those regions ranked with the highest instability potential would receive more scrutiny during design for new construction, or in programming work effort for maintenance operations. Geographical Information Systems (GIS) serve as an excellent screening tool for this purpose because of their capability of manipulating large quantities of spatial data and providing visual results of the manipulation. This paper describes the creation of a GIS screening tool to predict slope instability potential in the State of Arkansas. Fieldwork consisted of cataloging the location and characteristics of over 140 slope failures throughout the state. This failure information was combined with a wide variety of other publicly available spatial data to develop a model of failure prediction parameters. Not surprisingly, the spatial attributes that provided the strongest correlation to slope instability were soil, geology, gradient, soil cover and moisture. When this model is applied to a region, the spatial data is displayed in such a fashion, on county or state level maps that a manger can distinguish five levels of slope instability potential. Based on the results of this rapid screening tool, managers can program appropriate measures to insure slope safety.

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Introduction

The topic of slope stability is one that often receives only limited attention during the design phase of major highway construction projects. Increasingly, however, state and federal transportation agencies are finding that the remediation of failed slopes is becoming a costly alternative to proper up-front design. Based on the results of a five-year survey of maintenance and repair costs associated with the U.S. highway system, Walkinshaw (1992), concluded that over $100 million in 1990 dollars was expended annually on the repair of slope failures. Significantly, the state and federal highways included in this survey represented only 20 percent of the total road network in the United States. Thus the annual cost of remediating all failures associated with the highway system could be much more than the figure cited by Walkinshaw.

The recent construction of I-540 through the Ozark Plateau of Northwest Arkansas has heightened that State's awareness of the cost of slope repair. Historically, the Arkansas Highway and Transportation Department has performed only limited slope design. Instead, they have relied on some calibrated rules of thumb and on experience from previous construction in like terrain. Examples of these rules include, fill height restrictions for embankments based on the quality of fill material or the common practice of constructing cut slopes on a 3:1 ratio with only limited soil testing or analysis. While these practices have been reasonably successful in many areas of the state they proved disastrous for the construction of I-540. The project was plagued from the beginning by slides that occurred almost continuously during the construction of the route from Ft. Smith to Fayetteville. The end result was only minor delays in the completion of the project, but substantial cost overruns. While many of the slides were small and easily repaired a significant number were of the magnitude of the repaired cut slope depicted in Fig. 1. This failure alone cost over a million dollars to repair.

Slides continue to occur regularly along this 60-km section of highway resulting in large, unprogrammed, maintenance expenditures for the Highway Department. While most failures have been located in cut slopes located a safe distance from the traveled way at least one failure has encroached on the pavement structure of the highway. Fig. 2 depicts the temporary repair of the southbound lanes of I-540 after a failure in a side hill fill caused a portion of the pavement to break away and subside approximately 230 mm.

Conventional wisdom says that many of these slope failures could have been avoided by conducting field investigations to quantify the soil properties and other environmental factors, which are important in the design of stable slopes. However, the back analysis of some of these failures using routine field measures of soil strength gathered after the fact, reveal that normal exploration and testing techniques, coupled with traditional stability analyses, predict very stable slopes in this area. Additionally, the cost of conducting extensive field investigations to quantify all of

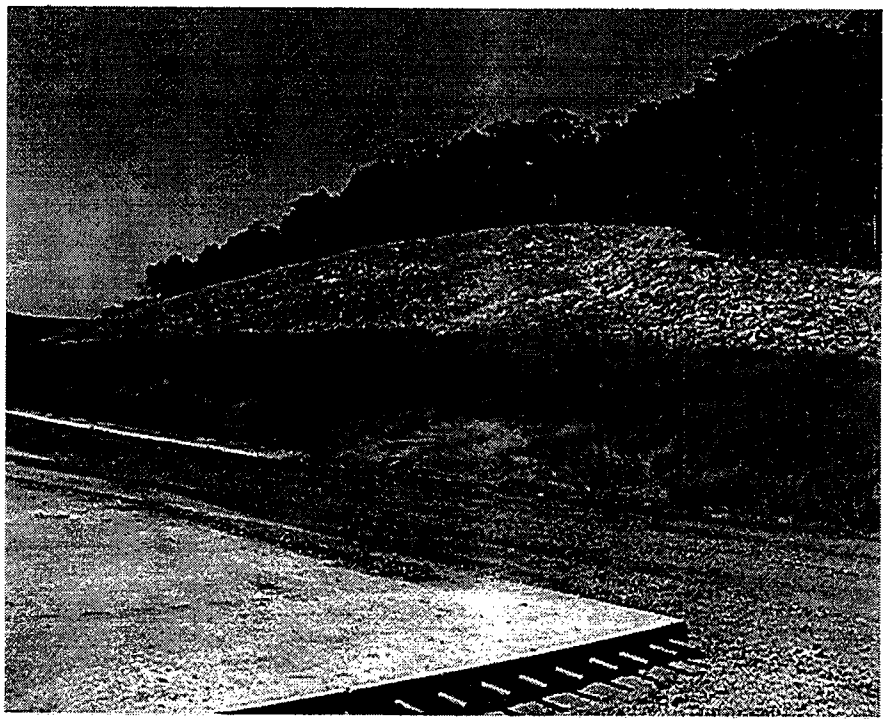


Figure 1. Slope repair during construction of I-540 using "repair by replacement" method.

the factors causing instability could easily exceed the cost of slope repair if no methodology is used to direct investigative efforts to those areas having the highest potential for failure. In an attempt to assist planners in identifying areas requiring some measure of sub-surface exploration and testing, a Geographical Information System (GIS) has been developed to relate slope failures to causative factors. Using this planning tool, managers can devote limited field exploration and testing resources to the areas with the highest potential for failure.

Background

Geographical Information Systems (GIS) are a relatively recent outgrowth of the field of cartography. Burrough and McDonnell (1998) trace the origins of GIS to the mid 1960's, when the availability of the digital computer spawned both the conceptual methods for spatial analysis and the actual potential for quantitative thematic mapping. Essentially, GIS allows the linkage of attribute or thematic data to a geographical coordinate system. While this linkage provides the potential for creating a very powerful multi-variable modeling tool, Berry (1995) contends that the major use of GIS today is still the creation of map products. Indeed, the use of GIS as a modeling tool in the area of geotechnical engineering is truly in its infancy. One of the earliest uses of GIS in landslide susceptibility mapping and perhaps the first

Figure 2. Temporary repair of pavement section on I-540 following a slope failure in a side hill fill.

statistical approach is reported by Carrara et al.(1978). His research team applied multivariate analysis techniques to grid cells with 200 by 200 m resolution, representing a region in central Italy. Their method was founded upon a single conceptual model that requires three steps. First, landslides over a target region or a study area are mapped. Then, the identification and mapping of various sets of geological-structural factors, such as rock composition and structure, steepness of slope and structure, stream evolution and vegetative cover is conducted. These factors are directly or indirectly correlated with slope instability. Finally, the method requires an estimate of the contribution of these factors in generating slope failures, and the classification of the land surface into domains of different degrees of hazard. Carrara et. al., suggested that their models imply that slope failures in the future will be more likely to occur under those conditions which led to the past and present instability.

More recently, researchers have employed qualitative techniques to produce landslide susceptibility maps for land use planning. Brabb (1987, 1995) reports the use GIS in predicting slope hazards resulting from seismic activity in the San Francisco Bay area while Hansen (1995) reports a similar effort in the Hong Kong area. Both studies integrate a landslide inventory with topographic, geologic, soil, ground cover, hydrologic, and economic maps to produce susceptibility maps that assist manages in controlling population density in high risk areas.

In the 1990s researchers began to use deterministic models coupled with GIS to create hazard maps. The most recent of these studies is reported by Khazia and

Sitar (2000). These researchers used a simple infinite slope analysis, with soil properties, gradients and ground accelerations extracted from digital maps, to predict instability in the San Francisco Bay area from seismic activity.

The common thread of all of the studies reported here is that the landslide or failure inventory was generated primarily from the interpretation of aerial photography and the application of the methods was restricted to a relatively small geographical area.

Procedures

Unlike the previous studies reported from the literature, the purpose of the current study is to determine landslide or failure activity over the expanse of an entire state through actual field investigation. This information is then related to publicly available spatial data, using simple Boolean techniques, to predict areas of potential instability throughout the entire study area.

As a first step in this study a simple survey was created and distributed to the ten district offices of the Arkansas Highway and Transportation Department (AHTD) requesting information about the quantity and magnitude of slope failures in each district. Based on the responses a prioritized work plan was developed to visit all areas of the state in the order of failure density. Each of the 75 county maintenance supervisors served as a guide to locate the failure sites in their area and to provide as much background information about each failure as possible. All information regarding the failure area was recorded on the Field Investigation Checklist illustrated in Appendix A. Ideally, every field in the checklist would have been complete at each site,. However, the form was normally only half-complete for the majority of the sites. Unfortunately, the data most frequently missing related to soil strength and water conditions. A mapping grade GPS receiver with post processing capability was used to record the center of mass, aerial extent, and elevation change of each failure site to 1-meter accuracy. As a back up, the gradient, length and breadth of each failure were measured using a meter tape and several digital photographs were taken of each site. Over the course of two years 140 failure sites were investigated throughout the state with a significant majority being located in the northern half of the state. The information from the field investigation was transcribed into a Microsoft Access® database for incorporation into a GIS.

Three different GIS engines were used in this study. Geomedia® Professional, an Intergraph product was used for vector data manipulatoin and as a general viewer while MFWorks for Geomedia®, a Thinkspace product, was used for raster based analyses. These two products were selected because the AHTD had settled on Intergraph as the vendor for their GIS software. However, a substantial amount of public domain data is available only in the ArcView® shapefile format so Arcview® was used to manipulate and clip that data before it was imported into a Geomedia® data warehouse to preserve the proper spatial referencing. Theoretically, the use of Arcview as an intermediate data manipulator should not have been required. However, we found through the course of the study that many operations on shapefiles could not be completed successfully or properly using GeoMedia®. Table 1 provides a brief summary of the native characteristics of the data used in this

study. With the exception of the digital elevation data all of the areal data shown in Table 1 required manipulation in ArcView® to clip it to county boundary lines and to convert its native coordinate system to Universal Transverse Mercator Projection (UTM) before it was imported into MFWorks® for rasterization and analysis.

Table 1 Summary of data characteristics for publicly available data used in this study.

Data Layer	Data Type	Format	Source	Coordinate System
Failure Site	Vector	MSAccess	GPS Rover File	Lat, Long NAD 27
Digital Elevation Model (DEM)	Raster	SDTS	USGS	UTM
County Boudaries	Vector	Arc/Info	1999 Tiger/line	Lat, Long NAD 83
Highway System	Vector	Arc/Info	1999 Tiger/line	Lat, Long NAD83
Streams	Vector	Arc/Info	1999 Tiger/line	Lat, Long NAD83
Soil Type (STATSGO) (SSURGO)	Vector	DLG-3	NCRS	UTM
Geology (STATSGO)	Vector	DLG-3	NCRS	UTM
Springs/Seeps	Vector	ASCII	USGS, Little Rock, AR	Lat, Long NAD 27
Vegetative Cover	Vector	ArcView	Center for Spatial Technology, AR	UTM NAD 27

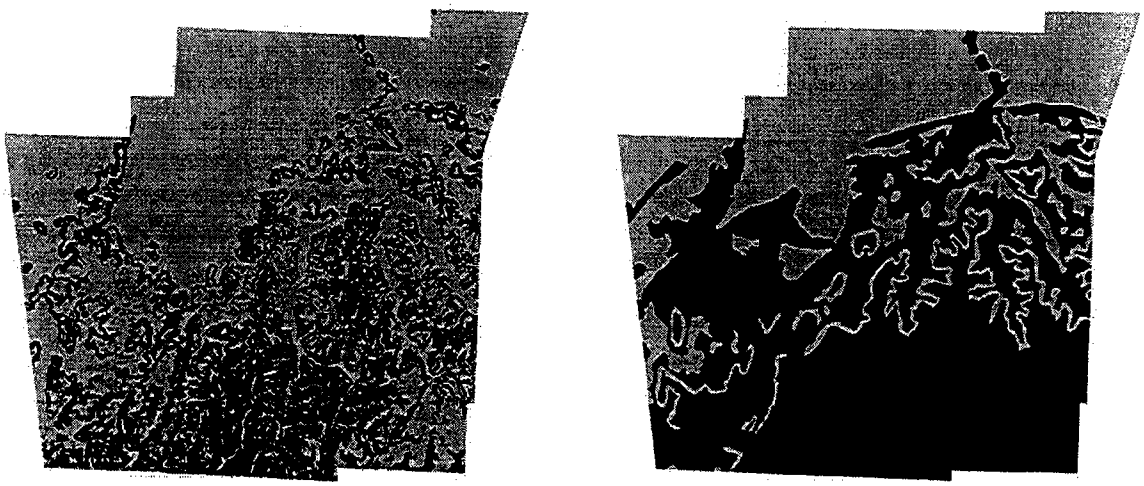
After insuring that all the layers had the same grid size and were properly georeferenced, the MFWorks® *Score* operation was performed using the rasterized failure sites layer as the *Zone/Region Map* to determine the range of elevation and gradients based on the known locations of the failures. This operation was also used to determine the soil, geology and ground cover types based on the presence of slope failures. The *Score* operation gives the attribute value of a data layer at each particular failure location. Using the result of this process one can then make use of relational operators, such as greater than, less than or equal to, to create individual Boolean layers. Each Boolean layer consists of two values, one representing a true condition and the other representing a false condition. For example, after determining the appropriate slope or elevation thresholds within Washington County, a Boolean script was executed to create the Boolean layers for slope and DEM. Examples of the Boolean scripts are given in equation 1.

$$DEM_B=(\text{“DEM”}>=382)AND(\text{“DEM”}<=657) \quad (1a)$$

$$Slope_B=(\text{“Slope”}>=1)AND(\text{“Slope”}<=18) \quad (1b)$$

The first script indicates that elevations between 382 and 657 meters would represent a true condition while values outside this range or threshold would be a false condition. A true condition for any data layer in this research also indicates a potential for having slope failures.

To obtain the Boolean layers for soil and geology, and ground cover the R*ecode* operation was performed after *Score* to reassign a value of 1 to all the soil, geology or ground cover types at failure locations while the remaining were assigned a value of 0. The *Recode* operation was used because the result of performing the *Score* operation indicated that there was more than one soil or geology type associated with failure locations. All of the qualifying types grouped together to represent a true condition. For the springs and streams layers, the MFWorks® *Spread* operation was used to create buffered regions for the rasterized springs and streams layers. Each output layer had a value of 0 that represented the county background while a value 1 represented a positive association with a slope failure. Figure 3 gives an example of two Boolean layers for Washington County, AR.



(a) (b)

Figure 3 Example results of a Boolean analyses for Washington County, Arkansas. Black areas indicate (a) soil types, (b) surface geology types that are found at known failure locations.

Once all six Boolean layers were created, the script shown as Equation 2 was executed which added the Boolean layers together to create a failure potential model for Washington county.

$$\text{Model} = \text{DEM_B} + \text{Geology_B} + \text{Soil_B} + \text{Slope_B} + \text{Streams_B} + \text{Springs_B} \quad (2)$$

Based on Equation 2, each data layer was assumed to carry an equal probability of contributing to a failure. The resulting prediction model for Washington County is illustrated in Fig. 4. Areas with the darkest shading represent the highest failure potential, while the small white circles represent failure locations. This analysis was

repeated for every county in Arkansas where failure data was available in order to establish a range of threshold values for each dependent data layer.

Figure 4. Spatial model to predict slope instability based on the equal weighting of causative factors including slope, elevation, geology, soil type, ground cover and water.

Results

Sixteen counties had enough failures to produce a Boolean model of limited usefulness. Virtually all of the Boolean models show that failures do not correlate well to all data layers. Most notably, the layers dealing with water showed very low correlation to failure. Thus, one might deduce that slope failures are not affected by the nearness to water, which is contrary to field evidence at failure sites. This poor correlation between failure sites and streams and springs is more likely due to the fact that the locations of the investigated slope failures were near roads instead of streams, springs, or recorded seeps. The strongest correlation appeared to be related to soil and geology, followed by slope and elevation.

Most of the investigated failure sites were found to be located in formations that consisted of different types of shale mixed with some fractured chert and limestone. Failures seem to occur in the northern part of Arkansas where Boone, Chattanooga or Cason geologic mapping units are encountered. The central Arkansas slope failures seem to correlate with Womble, Collier, Missouri, and Everton geology types. Most of these formations are fairly young geologically, especially the terrace deposits located near streams. Most of the soils in the sixteen counties belong to the Allen, Enders, Linker, Mountainburg, Nella, Carnasaw, Spadra and Moko soil series. These soils are closely associated to each other in terms of their engineering properties. At the time of the study, Soil Survey

Geographic (SSURGO) data which provides county level soil mapping, was only available for seven counties in Arkansas. As a result, state level soil mapping data (STATSGO) was used in the Boolean analyses and only general soil groupings could be discerned. At this point, one can only conclude that these types of soil can be found on gently sloping to steep terrain. They are usually made up of a thin surface layer of loamy material and an underlying clayey material. They are fairly well drained materials at the surface, but the permeability values can go up to 4x10-6 m/s within the clayey material. The range of values for plasticity index is between 5 to 15 for surface material to material below one foot in depth, respectively. Virtually all slope failures occurred when highway cuts were made in previously flat or gently sloping terrain. Threshold gradients ranged from 1 to 18 percent.

A universally applicable Boolean model could not be created for the entire state. However, when a cross-examination was conducted between failure locations, geology, soil type and a landform map some interesting relationships developed. The landform map generally divides the entire state into five major landforms--Boston Mountains, Arkansas Valley, Ouachita Mountains, Mississippi Delta, and Coastal Plain. Counties within a given landform tended to exhibit similar geology and soil type. For example, Washington, Crawford, Madison, Newton, Boone, Stone, and Van Buren Counties are all located within the Boston Mountain landform, and possess similar geology and soil types. Yell and Hot Springs Counties, which are part of the Ouachita Mountains landform, also exhibited similar behavior. A Boolean model created for one county in a particular landform had similar threshold values as the models created for other counties of the same landform. Thus a semi-universal model could be created by landform. Fig 5 illustrates the results of applying the Boolean model created for Washington County to Crawford County. The relationship of failure sites to causative factors appears to be as good or better in Crawford County as it was in Washington County.

Figure 5. Results of the application of the Washington County Boolean model; to Crawford County.

The darkest shading in Fig. 5 represents the highest potential for slope instability, based on the Washington County model, while the white circles represent the actual failure sites.

Conclusions

A useful screening tool to predict slope instability can be developed by using simple Boolean operators and publicly available spatial data. The strongest correlation to slope instability and causative factors was with soil and geology. The accuracy of the model is currently limited by the coarseness of the mapping of soil and geologic properties. As a result, the aerial extent of those zones predicted to be unstable is quite large. As more refined data becomes available zones of instability can be pinpointed more precisely. While the models have some portability from county to county the user must be aware of landform limitations, e.g. a model developed for the Boston Mountains is not applicable to the Arkansas Valley.

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Appendix A: Field Investigation Checklist

FIELD INVESTIGATION OF SLOPE FAILURE

Date of Invesitigation: _____ Failure ID #: _____

County: _____

Investigator(s) Name: _____

Date of Slope Failure: _____

Location of Slope Failure: _____

History of Past Slope Failures: _____

Causes of Past Slope Failures:				Slope Cover and Structures:				
Construction	Ground Water	Pipe Failure	Excavation	Retaining Wall	Natural	Cut	Fill	Grass

Geologic Conditions

Form of Failure: _____

Soil Properties

	Soil 1	Soil 1	Soil 1	Soil 4	Soil 5	Soil 6
s						
c						
ϕ						
γ						

Water Conditions	Water seeping from				Fluidity of Debris	Saturated	Unsaturated

	Yes	No		Yes	No
Back Analysis:			Data Present		
Previous FOS					

	Yes	No
Drainage Improvements		
Is the Drainage working		

Reason if not working: _____

What drainage Features are Present: _____

Slope Geometry			
	height	breadth	angle
Original			

Max. Depth of Failure Below Original Slope: _____

Stabilization Measures when a Landslide and Rockfalls Occur on the Same Slope

by
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Abstract

A section of rural roadway in north-central Pennsylvania was being improved by widening and realignment. This work involved a major rock cut, with a thick soil zone in the transition area to fill. This transition area was further complicated by a veneer of boulders on top of the undisturbed soil. The Department had concerns about the slope in this area and called for an emergency placement of a rock buttress.

The contractor placed a rock-faced embankment against the slope, however it started to fail before the buttress was completed to its proposed height. In addition, with winter rains, boulders had been dislodged from the top of the cut and had been found in traffic lanes of the roadway. Between Christmas, 1997, and New Years Day, 1998, Baker was contacted by PennDOT to assess the rock buttress and determine the need for other special stabilization measures in this area.

After an initial site visit, available project information was reviewed. A series of test rock rolls were performed on the slope to observe actual rockfalls at the site and collect data to aid calibration of the CRSP model. After this calibration, existing conditions were modified to consider various options. Concurrently, the stability of the buttress was evaluated using STABL. The configuration of the buttress was found to be marginally stable after its height was reduced. However, its stability was a factor in the rockfall remediation alternative, as it was desirable to increase its factor of safety.

The boulder veneer on the undisturbed portion of the soil was the source of the rockfall problem, and it was triggered by erosion of the steep soil slopes. Flattening of the soil slope was not a practical option in this area. Therefore, a variety of geosynthetic and biological techniques were evaluated. A combination of these techniques was designed and implemented to minimize erosion potential and thereby improve stability.

The rockfall mitigation design involved a break-away post Brugg fence mounted on a concrete barrier system. This system allowed easy set-up on the shoulder of the roadway and the potential for easy removal upon stabilization of the soil slopes and minimization of the rockfall hazard. After more than 2 years, the vegetation on the slope has become established and the rockfall fence has successfully protected the motoring public. In addition, the rockfall fence system has served as a model for other applications.

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Two individuals also warrant acknowledgment. David Frey of Michael Baker Jr., Inc, was the structural engineer on this project, responsible for design of the concrete blocks and coordination with Brugg, the rockfall containment system vendor. Frederick Gaffney, conservation agronomist with the USDA Natural Resource Conservation Service, provided plant material recommendations for bio-stabilization.

INTRODUCTION

Project Overview and Start of Baker Involvement

In 1997, a section of U.S. Route 6 in Bradford County, Pennsylvania was being improved by widening and realignment. This work involved a major rock cut, with a thick soil zone in the transition area to fill. This 200-foot long transition area was further complicated by a veneer of boulders in the root mat on top of the undisturbed soil. The Department had concerns about this area and contracted for placement of a rock buttress as emergency construction. Before the buttress was completed, it started to fail. Failure was detected through field surveys as bulging of the lower buttress and slumping of the upper buttress surface. In addition, several rocks had come off the slope during non-construction periods. These rock falls had stopped in the traffic lanes, so maintenance crews were called to protect traffic whenever the contractor was not available.

On December 29, 1997, Michael Baker Jr., Inc., was issued a work order for the site under an open-end, agreement with the PennDOT. Baker was directed to assist the Department with three tasks: determination of the need for a rockfall barrier, analysis of the rock buttress to verify a final stable configuration, and investigation of erosion protection for the steep soil slopes.

Information from the Department

Records provided by the Department included an overall topographic site plan, records of borings taken during design, roadway typical details and cross sections from the original roadway reconstruction contract, and plans and cross sections from the emergency buttress contract. Final "As-Built" cross sections for the buttress were not developed until January 9, 1998, and were received by Baker on January 14, 1998.

For stability analysis, soil classification and strength test results were also provided by the Department. Two soil samples were collected from the buttress embankment and one sample was from the natural ground. All three samples had comparable test results, indicating the finer material in the buttress is essentially the same as the surrounding soil, as would be expected.

PennDOT's Publication 408 and the construction plans for this section of roadway offered the first steps in control of the steep soil slopes. The plans called for seeding of the soil cuts with Formula C (crownvetch and annual ryegrass). Sections 804 and 805 of Pub. 408 provide default seeding, soil supplements, and mulching requirements. Section 806 provides specifications for geosynthetic material use for slope erosion protection.

PennDOT also provided a table of possible alternative treatments according to "live construction," "mixed construction," and "inert construction" from TRB Special Report 247: "Landslides: Investigation and Mitigation," as a suggested direction for investigation of treatments for the steep soil slopes. In addition, PennDOT's Bulletin 15 provided a list of approved geosynthetics producers to contact.

TASK 1 - NEED FOR A ROCKFALL BARRIER

Field Observations

Rockfall field testing was conducted on January 8, 1998. The purpose of the field testing was to observe actual rockfall results at the site and to gather information required to increase the accuracy of computer modeling. Test rolls were performed from various elevations using typical rocks found on the slope. Observations of rock type, shape, roundness, dimensions, origin of roll and test results were recorded for each test rock or group composed of similar rocks. General observations of slope surface roughness, composition and firmness were also made to assist in subsequent computer modeling.

The typical behavior observed in most rolls of the blocky specimens was to:

1) roll short and stop, or
2) roll to the toe and be stopped by the standard height barrier.

The typical behavior observed in most rolls of the flaggy or platy specimens was to:

1) roll some distance (often curving to a side), fall over and stop, or
2) roll bouncing to the toe, over the standard height barrier and onto the roadway.

A total of 93 test rolls were recorded, ten of which reached the roadway (10.8%). Nine of the rocks reaching the road were flat-shaped and rolled on edge. The eight rocks reaching the roadway that originated from the top of the scarp comprise 8.6% of the total rolls and 19% of the roll from that origin area. While there is very little chance that material from the buttress will

reach the roadway by rockfall or rolling, the upper flat rocks rolling on edge present a hazard at the site. If rocks immediately above the cut are mobilized by continued failure of the soil mantle, a significant percentage will reach the roadway.

Computer Analyses

Rockfall analysis was accomplished using the Colorado Rockfall Simulation Program (CRSP), Version 3.0a, 1995, developed by the Colorado Department of Transportation. The properties of the rolling rocks, surface roughness, tangential coefficient of friction, normal coefficient of restitution and slope geometry were all determined considering the results of the field tests.

For a smooth 1.5:1 slope to the shoulder, a maximum bounce height of approximately 10 feet (mid-point of a 1.5-foot diameter rock) was calculated at the guide-rail location. A maximum reasonable kinetic energy of just over 65,000 foot-pounds was calculated at this location.

Necessary modifications to the buttress toe had the effect of increasing the trajectory of the rock at the bottom of the slope. However, this well may be due to the simplifications in the original model.

Analysis of Barrier Options

Based on the results of the field test rolls and supported by the CRSP runs, a percentage of rocks initiating from the top of the soil cut slope will cross the existing guiderail location at heights up to ten feet. Because of this, several options were considered to reduce the likelihood of rockfalls reaching the roadway. These options included:

1. Excavate to re-establish the rockfall drop zone
2. Drape a fence on the slope
3. Create a raised drop zone and use a rockfall fence
4. Place a rockfall fence or barrier at the toe without a drop zone
5. Place wire mesh netting on the entire cut slope
6. Create a depressed drop zone with a retaining wall and use a rockfall fence

All options include installation of single face barrier in place of the existing guiderail at the edge of the roadway shoulder.

Technical Considerations

Option 1 is basically the extension of the rockfall zone which exists for the adjacent rock cut. It would involve removing the rock/soil buttress to establish a depressed rockfall drop zone. CRSP runs demonstrated that the standard-height single face barrier would stop rocks rolling through the drop zone, provided the 6-foot depression behind the barrier were created using a gabion wall instead of the 1.5:1 slope which exists ahead station. The precise extent to which the current buttress is built against a rock slope or it is supporting an over-steepened soil slope is unknown. However, excavation to the extent required by this option would probably cause immediate soil

slides at the head of the slopes, and leave marginally stable soil which would continue to periodically slide and accumulate in the drop zone. Therefore, this option was removed from further consideration.

Option 2 is to install fencing with vertical posts approximately halfway up the buttress. The posts would have an exposed height of eight feet and be spaced approximately every 30 feet along the slope. Cable would be run between the tops of the posts. Wire mesh would be draped loose off the tops of the posts and cables. This system catches and controls the rocks by restricting their bouncing and absorbing their energy. Installation of the posts into the buttress material would probably require field adjustments due to boulders, but precise placement is not critical. Maintenance would involve periodic inspection and occasional partial replacement of the mesh or a post. It would not be necessary to continually clean the buttress slope. However, any large rocks observed on the slope would have to be removed or broken up before impact with the fence. Because of worker safety issues associated with maintenance on the slope, this option was removed from further consideration.

Option 3 entails construction of a raised rockfall drop zone supported by a soldier pile and lagging wall. The soldier piles would be extended above the wall portion to allow for incorporation of a rockfall fence at the top of the wall. The soldier piles would support the fence fabric. Alternately, a proprietary-type rockfall fence (e.g., Brugg Cable) could be installed at the top of the raised drop zone. An eight-foot minimum bottom width would be maintained for catchment and clean out. Since rocks accumulating in this zone are anticipated to be in the 0.1 to 3.5 cubic foot size range (typically much less than 500 pounds), cleanout can reasonably be accomplished with the use of a small "Bobcat"-type vehicle. This option allows for construction of a rockfall drop zone and clean out area while simultaneously providing increased support for the toe of the existing buttress. CRSP runs indicate that a fence 8 feet high would need to be installed above a drop zone elevated 6 feet in order to stop the rocks from reaching the shoulder. This option is technically feasible.

Option 4 is construction of a rockfall fence (or barrier) in the area between the toe of the buttress and the single face barrier. In the absence of a drop zone, impacts of larger sliding rocks would be more likely to cause damage to the fence. Maintenance would be complicated, but could be performed from the front of the single face barrier around the soldier beams, with proper traffic control. If materials are allowed to accumulate behind the fence, the height should be increased to 14 feet. If a solid barrier (wall) is constructed to this height, accumulated materials could be graded behind the barrier to construct a drop zone (comparable to Option 3). This option is technically feasible.

Option 5 is placement of a wire mesh netting over the cut slope in the area of concern. This works well when the anticipated initiation of the rockfalls is determined to be within the cut slope itself. However, at this site, the rocks which pose the greatest potential hazard are those from the uppermost parts of the cut. Therefore, this is not a technically feasible option here.

Option 6 is creation of a drop zone at essentially the roadway grade, using a wall to retain the toe of the buttress. A rockfall fence comparable to that of option 4 would be installed. This option significantly decreases the potential for damage to the fence due to sliding of larger rock blocks down the slope by creating a catchment area. It allows maintenance to be performed from behind the fence with considerably more ease and less restrictive traffic control. Several types of walls were initially considered, including MSE and gabions. The extent of excavation needed for these walls created potentially unstable conditions comparable to those which made option 1 not feasible. However, a soldier pile and lagging wall can be constructed with a minimum of excavation behind it. With this wall type, this option is technically feasible.

Cost Comparisons

Since options 3, 4 and 6 were determined to be technically feasible, cost comparisons were made for several variations. For comparison purposes, all options were considered to be installed for the entire 200-foot length. Prices were developed from similar construction projects built over the last dozen years, with appropriate adjustments for inflation.

Option 3, the elevated drop zone and fence, was estimated to cost between $125,000 and $165,000. Option 4 was estimated to cost between $35,000 and $40,000, using a 12-foot high rockfall fence system. Two types of systems were available: the proprietary Brugg fence system, and an H-pile, cable and wire mesh system. If a solid barrier (wall) were constructed instead of a fence, it was estimated such a wall would cost roughly $130, 000. The incorporation of a drop zone at road level, as called for in Option 6, would add $64,000 to $98,000 to the cost of Option 4.

The Recommended Option

Option 4 was selected, and Baker was directed to develop a final design to allow construction at the earliest reasonable date. The option was to use a Brugg containment system with breakaway posts, without a drop zone. After major impacts, the system can be readily inspected and reset, with replacement parts as necessary (including breakaway posts). Maintenance of materials behind the fence could be scheduled as necessary based on actual accumulation of debris.

TASK 2 - ANALYSIS OF THE ROCK BUTTRESS

As soon as the buttress started to fail, the contractor was directed to regrade the area, lowering the crest elevation of the buttress and flattening the buttress face slope from roughly 1:1 to 1.5:1. The Department wanted assurance that the flattened slope would be stable.

Computer Analyses

The buttress was analyzed using the STABL program, for both the 1.5:1 and the original conditions. A unit weight of 148.2 pcf was used to account for both the presence of rock and water in the embankment. An internal friction angle of 34 degrees was assumed, based on the test results. With these parameters, even the lowest tested value of cohesion (485.2 psf) resulted in a safety factor greater than one for the original conditions. Stability of the original slope was then analyzed with a range of cohesion values. It was determined that in order to have failure of the original slope (safety factor less than one), the cohesion had to be equal to or less than 218 psf.

Conclusions and Recommendations

The results indicate that the 1.5:1 slope has a static safety factor between 1.0 and 1.6, and probably is within the range of 1.3 and 1.5. According to PennDOT's Design Manual 1, Chapter 5, Section 7.2: "The minimum factor of safety shall be 1.5 for long-term final design conditions. For short-term conditions during construction the minimum safety factor shall be 1.3." Thus, the 1.5:1 slope existing safety factor probably meets the PennDOT Design Manual requirement for short-term conditions. PennDOT often accepts a 1.3 safety factor for long-term conditions, provided strength testing has been performed on the site materials. Since this was done in this case, the slope probably meets long-term requirements also.

However, the stability analysis only considered internal stability of the buttress. The analysis did not consider any loading that might exist from the natural soil mantle materials behind the buttress. It is believed that the natural soils are only marginally stable, especially where they are thicker than a few feet. The buttress elevation was lowered during regrading, exposing most of the upper thick soils.

TASK 3 - INVESTIGATION OF STEEP SOIL SLOPE EROSION PROTECTION

Because of their marginal stability, the steep soil slopes at the top of the roadway cut are especially sensitive to erosion. With the rock buttress no longer providing protection, alternative measures were needed.

Field Observations

During the site visits, the upper soil slopes were inspected. Four observations made during the site visits are of particular note:

1. Concentrated water flows, decisive in creation of major erosional features, appear to travel below the vegetative mat on the upper soils, on the top-of-rock surface and through bedrock joints. These flows saturate local pockets of soil to create small debris flows on the soil portion of the cut.

2. Where the soil slopes thicken, they appear to present the potential for landslides or debris flows if water is allowed to saturate the soil.

3. There are no major discernable erosion gullies above the cut.

4. The steep soil slopes at the top of the cut have slid away in several places. Where these soil slopes have failed, they have left a series of arcuate scarps which appear to have a fairly consistent chord length.

Preliminary Considerations

Management of groundwater flows through a series of horizontal drains was briefly considered, but dismissed because of constructability problems. Practically any work on the slope would have to be done manually to minimize the potential for creating slope instabilities. Therefore, attempts to intercept the groundwater by mechanical ("inert") construction were set aside in favor of "live" construction (seeding and/or planting).

Since concentrations of water typically lead to erosion problems, it usually is best practice to strive for as much overland sheet flow as possible rather than channelization. Where soil is thick, the use of geosynthetic blankets or mats promotes this overland flow, especially prior to full vegetative cover.

The previously failed soil slopes now appear to be in a reasonably stable configuration and will not be remediated at this time.

Plant Alternatives

The standard Formula C mixture can be supplemented with two classes of plants to improve the erosion control on this slope.

Areas of thinner soil and the buttress surface will be prone to periodic droughty conditions. Under these conditions it takes longer for the crownvetch to establish itself, and the ryegrass may die off before full groundcover is achieved. To avoid this, a warm season grass can be added to the mix. Broom sedge, little bluestem, and deertongue are warm season grasses typically recommended for such conditions.

Numerous woody shrubs are available which will help to establish a strong root mat, use available groundwater, and even help to absorb energy during rockfalls. The key is to use a low rhizomatous shrub, which will grow extensive roots without a large or heavy trunk which could become a future tree-fall hazard or adversely affect the slope stability. Options here include varieties of sumac, bristly locust (not black locust), Virginia rose, bush honeysuckle, and redbud. These shrubs may be either seeded with the PennDOT mix or planted as bare-root seedlings.

Geosynthetic Alternatives

Unlike organic nets, mats, and blankets; geosynthetic mats are considered to be permanent and are resistant to deterioration and degradation after installation. Two types of geosynthetics are covered in the PennDOT Pub. 408 specifications:

1. Turf Reinforcement Mat (TRM) - Sec. 806.2(b), and
2. Erosion Control and Revegetation Mat (ECRM) - Sec. 806.2(c).

ECRM's are basically just synthetic erosion control blankets that replace mulch or other ground protection. On the other hand, TRM's are expected to function as an intergral part of the final vegetative mat. Typically, TRM's are used under more extreme conditions than ECRM's, including steep slope applications

The most obvious distinguishing material feature is that TRM's normally are black, while ECRM's are green. Beyond that, TRM's are thicker, but they are more porous and have a lighter weight per square yard than ECRM's. Major differences in installation are that ECRM's are usually placed after seeding and soil supplement is applied; while TRM's are usually placed before seed, supplement, and mulch are applied.

PennDOT's Bulletin 15 identifies four producers of geosynthetic erosion control materials. At least two of them manufacture TRM's and three of them manufacture ECRM's. Costs for TRM's range from roughly $5.00 to $5.25 per square yard, while costs for ECRM's are approximately $3.50 to $4.00 per sq. yd., plus installation cost. Installation costs for both types are comparable, and are more dependent on the site conditions than handling of the material.

Manufacturers recommend that the upper edge of mats be anchored, typically by burial in a slot trench. In steep slope conditions where trenching can not be performed, anchoring can be done successfully by pinning the mat down with 18-inch long metal pins.

Recommendations

Where mixed (both live and inert) construction is used for erosion control, it is commonly termed biotechnical slope protection. The biotechnical slope protection recommended for this area consisted of:

1. Supplementing the standard PennDOT Formula C mixture with broom sedge at the rate of three pounds per acre and bristly locust at the rate of ten pounds per acre. This supplemented mixture was to be applied with the standard soil supplements over the entire cut above the rockfall system.

2. Covering the entire cut above the wall with TRM from a producer listed in Bulletin 15.

3. Applying mulch hydraulically to the entire cut slope in this area at the standard rate, using only hay or straw.

CONSTRUCTION AND POST-CONSTRUCTION

Regrading of the buttress was accomplished by the contractor responsible for its original placement. It was decided to close that emergency contract and bid a separate contract for construction of the rockfall containment system and the biotechnical slope protection. Baker prepared the documents for this separate construction during the spring of 1998.

This separate project was bid in May and awarded in June, 1998. It attracted only two bidders, and the costs were much higher than anticipated - roughly $174,000 and $197,000. The contract was awarded to the low bidder, and construction was undertaken later that summer.

Adjustments were made during construction to both the rockfall system and the slope protection. Alternate lengths of single-face barrier sections were proposed and accepted. Minor modifications were made to the break-away at the concrete block - fence post connection. An ECRM was substituted for the TRM, and was only placed on the upper thick soil portion of the cut slope.

After two years, the rockfall system has yet to sustain any damage. Very little material has traveled down-slope to impact the system. However, the design has application in other areas because of its potential portability.

The inclusion of broom sedge and bristly locust seeds has resulted in more complete and hardier slope protection. It appears that the ECRM has actually hindered vegetation of the uppermost soil slopes, although it is slowly getting covered as the existing plants propagate.

The figures on the following page show the rockfall containment system roughly a year after installation.

Figure 1. View from the roadway. Note the smooth face on the concrete barrier.

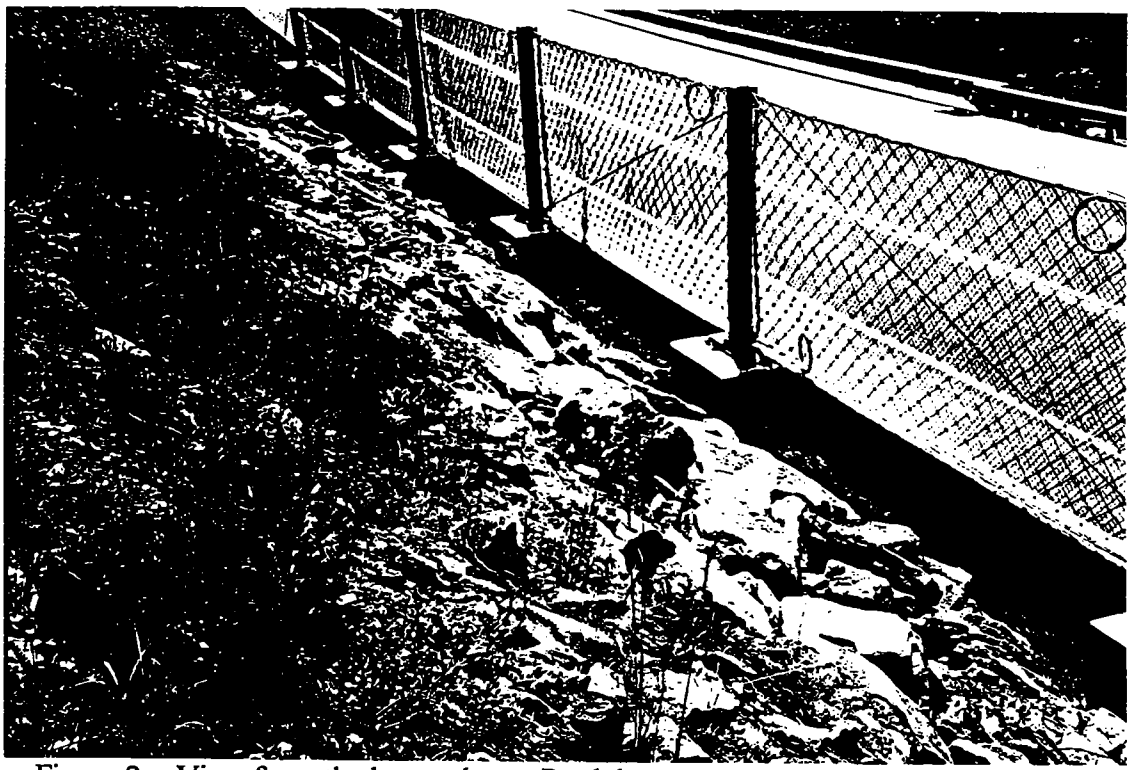
Figure 2. View from the lower slope. Rock buttress remnant at the base.

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Highway Landslide Stabilization Using Wick Drains

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ABSTRACT

One of the most effective options to stabilize landslides is to reduce the amount of water they contain by the installation of horizontal drains. While drains may not control rapid infiltration following heavy rainfall in fine-grained materials, they do provide long-term control of ground-water levels, thereby reducing driving forces and uplift pressures. The most common horizontal drains, which are slotted PVC pipes placed into drilled holes, have two significant drawbacks: they require periodic maintenance to remove soil that clogs the slots, and they are expensive, approximately $6 – 11 per lineal foot.

Horizontally installed wick drains may solve both of these problems. Wick drains are flat, geotextile-covered plastic channels which have been used since the 1970's in vertical installations. They are driven into the ground by a specially adapted crane, and are used to accelerate consolidation and settlement. The geotextile prevents clogging of the drain, and installation is very economical. We have installed drains at an instrumented test embankment at the University of Missouri-Rolla, and at seven highway landslides in Missouri, Colorado, and Indiana. Three different types of equipment, including bulldozers, backhoes, and standard wick drain driving cranes, have been used to install over 100 horizontal wick drains. An overview of equipment requirements, installation methods, and solutions to common drain installation problems is presented.

Simulated rainfall experiments on the test embankment verified the effectiveness of drainage and water table control with the wicks, and provided a controlled environment for computer modeling of slope stability and evaluation of wick layout.

The stabilized landslides are being monitored to track water levels, slope movement, and roadway movement to verify the effectiveness of the wick drains. Preliminary analysis shows that the wick drains have been successful in lowering water levels and enhancing stability, and that the drains are resistant to clogging and natural deterioration.

INTRODUCTION

A method has been developed to use soil wick drains for a novel application of landslide and slope stabilization. Wick drains are flat, fabric-coated plastic channels, which were initially developed to be vertically driven into the ground using a specially adapted crane. The wick drains accelerate consolidation and settlement by an order of magnitude by significantly shortening the flowpath for water to exit a soil layer. They were first developed in the 1930's and have found widespread use since the 1970's, when durable plastic became available, replacing the cardboard channels originally used (Santi, 1999). This study has developed equipment to install wick drains horizontally, so that they might be used to drain landslides. Drains have been installed in an instrumented test embankment to assist in drain layout design, and over 100 drains have been installed at seven sites in Missouri, Colorado, and Indiana to demonstrate the effectiveness of the procedure.

Horizontal wick drains are intended to address two significant drawbacks experienced by the drilled horizontal drains currently used. Drilled drains, which consist of slotted PVC pipes placed into drilled horizontal holes, tend to clog with fine material, since it is often impractical to place a filter pack over the slots, and they are expensive, approximately $6 – 11 per foot (Ruff, 1980; Trolinger, 1980). The geotextile fabric on the wick drains serves as a filter to reduce clogging, and the wick drains are economic to install.

INSTALLATION PROCEDURE

The method of installation is to use a bulldozer or hydraulic excavator to push a small-diameter steel pipe into the hillside. The pipe sections are preloaded with lengths of wick drain, and the first pipe is attached to a

disposable drive plate at the front. At the target depth, the pipe is withdrawn, leaving the wick drain in place. The installation procedure consists of the following steps:

1. Wick Preparation – The wick must be rolled and tied to fit inside the push pipe. Wick sections are cut to lengths one foot longer than the push pipe, rolled into tight cylinders, and tied at one foot intervals with five to eight inch long cable ties (Figure 1).

Figure 1. Rolled wick being loaded into drive pipes.

2. Pipe Loading – Pipes are preloaded with lengths of rolled and tied wick. Depending on the inner diameter of the pipe, wick may be pushed through the pipe, or it may need to be pulled through with a piece of rope. The first pipe to be driven should have a drive plate attached to the front end of the wick with cable ties (Figure 2).

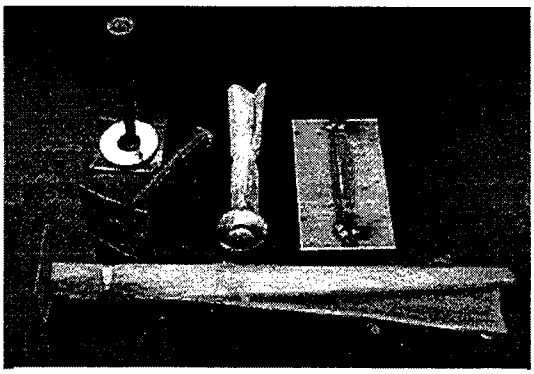

Figure 2. Drive plates with supporting washer (upper left), carriage bolt used for driving (center), drive plate attached to rolled wick (bottom), and standard drive plate for vertical wick drain installation (upper right).

3. Preparation of Drain Location – Because the first pipe has a tendency to slide up the slope during initial driving, a vertical face should be cut into the slope for a driving surface. This face only needs to be a few inches in height.

4. Driving of First Pipe – The drive head is slid over the end of the first pipe and the wick is folded out of the way (Figure 3). The first pipe is then aligned at the desired angle (usually horizontal or less than 5° upward) and pushed one or two feet into the slope. The angle and bearing may be measured again and adjusted slightly at this point, and the pipe is pushed for the remainder of its length.

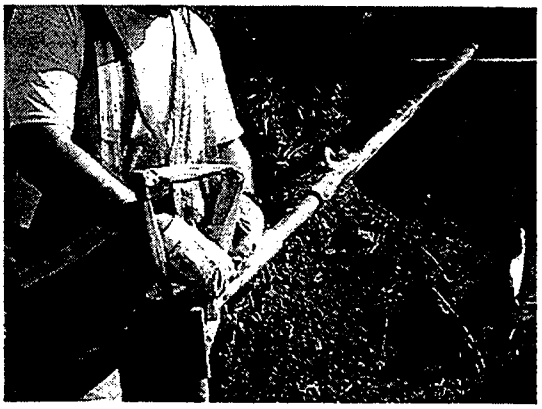

Figure 3. Drive head in place on drive pipe. Note larger diameter pipe used as a bracing sleeve around drive pipe.

5. Continued Driving – Additional pipes are driven by first splicing the end of the wick protruding from the pipe in the ground to the end of the wick in the next pipe section (Figure 4). Splicing is accomplished by stapling with a plier stapler. The spliced wick is rolled and tied, the pipes are threaded together, and the new pipe is pushed in the same manner as the first. Pipe sections may be added until the desired drain length is reached or until the driving resistance causes refusal.

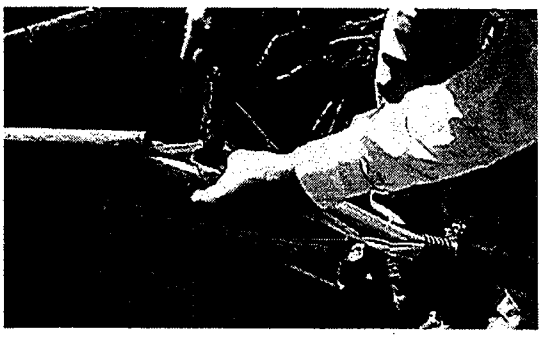

Figure 4. Splicing a new wick section before attaching the next pipe.

6. Pipe Withdrawal – Once the total length is driven, pipes are pulled from the ground by attaching a chain to the end of the protruding pipe and pulling each section smoothly out of the ground. The wick remains in the ground because the drive plate anchors the wick in place and resists withdrawal.

Based on our time records, an experienced crew (with 2-3 days of site work) can install 40-70 feet of wick drains in an hour. A crew consists of an equipment operator and three laborers. Prices for wick drain vary by volume, but typically range from $0.25 to 0.75/foot. The resulting cost for drain installation ranged from $3-6/foot.

INSTALLATION EQUIPMENT

The equipment required to install horizontal wick drains can be purchased from drill pipe vendors or easily constructed in a machine shop. Each item is discussed below:

1. Drive Plate – The drive plate shown in Figure 2 is modeled after the type of plate used in commercial vertical wick drain installation by the Nilex Corporation. The plate is three inches square and is cut from 18 gauge (0.05-inch thickness) sheet steel. Thicker steel (up to 12 gauge) will also work, but thinner steel may rip or puncture during driving. The steel is intended to fold around the pipe during driving, and then slip off and anchor itself in the soil during pipe withdrawal. A six inch piece of #4 reinforcing bar is welded onto the steel, and a one inch diameter washer is welded to the other end of the re-bar. The re-bar holds the plate in place during driving, and serves as an attachment point for the wick. The washer keeps the wick from sliding off the re-bar during withdrawal.

2. Drive Pipe – The drive pipe should have a minimum inner diameter of 1¼ inches to accommodate the rolled wick. The outer diameter is only limited by the pushing force available by the driving machinery. The work described in this study used 10-foot lengths of AQ wire-line drill rod, which is flush-threaded both inside and out, with an inner diameter of 1 3/8 inches and an outer diameter of 1¾ inches (wall thickness of 3/16 inch). This size pipe has been used to drive drains 100 feet in length, and it has been driven through materials with SPT stiffnesses up to 28 blows per foot. Larger diameter drill pipe will withstand higher driving pressures, will allow longer drains in harder geologic materials, and will provide for easier wick loading, but larger pipes are significantly more expensive.

3. Drive Head – A drive head receives and transmits the pushing load induced by the driving equipment, while protecting the female threads of the drive pipe and reducing the tendency of the pipe to slide off the equipment or to buckle. The drive head shown on Figure 3 consists of an 18-inch section of 2½-inch diameter pipe (2-inch inner diameter), on which is welded a thick flat steel plate. The steel plate has a slot cut into it so that the wick may be fed through and then folded back out of the way. Reinforcing plates and braces were added to make the drive head more robust.

4. Drive Equipment – The estimated pushing load required for the drive pipe used is less than 10,000 to 15,000 pounds. We estimate that bulldozers or trackhoe excavators in the 25,000 to 45,000 pound range are best suited for this task. Equipment substantially larger than this range may not provide the fine control needed during driving.

SOLUTIONS TO COMMON INSTALLATION PROBLEMS

The most significant problem encountered during drain installation was the tendency of the drive pipe to flex or buckle under the driving pressure. The problem was most significant when a new pipe was first being driven and the full 10-foot section was exposed and not confined by the soil of the hillside. The buckling generally subsided once several feet of the pipe had been driven into the hill. However, the pipe would also buckle when hard materials were encountered, such as sloping bedrock surfaces or boulder floaters in residual soil.

Larger diameter, thicker walled drill pipe will resist buckling better than the pipe used in this work. We have also used a larger diameter pipe as a sleeve around the drive pipe to prevent buckling. The sleeve may be pushed into the hillside along with the pipe, and then pulled out to be used with the next pipe section. Finally, buckling may also be controlled by supporting the drive pipe from below with timbers, and then forcing the flexure downward by controlling the attack angle on the bulldozer blade or trackhoe bucket (Figure 5).

Figure 5. Control of pipe buckling by supporting from underneath with timbers.

Another problem during driving is damage to the drive plate by the drive pipe. With flush-threaded pipe, the male threads present a narrow wall thickness which can tear through the drive plate steel. The solution is to attach a short two-foot section of pipe to the front end of the first pipe. The leading end of the short section is not threaded and the tail end is female threaded, to attach to the lead male end of the first pipe. The drive plate then rests against the leading end of the short pipe. This system will also protect the threads of the first pipe.

If it is anticipated that weathered rock, boulders, or other hard material may be encountered during drain driving, thicker steel should be used on the drive plate. We have experienced the drive plate shearing off when we have pushed through hard zones. Alternatively a 2½-inch steel washer may be slid onto the re-bar to rest between the drive plate and the front of the lead pipe to add strength. A 5-inch carriage bolt and 2-inch washer may be substituted for the drive plate for very hard or rocky zones.

The pipe-pulling process may also be streamlined. While a wrapped chain generally has enough friction to pull pipes out of the ground, a short piece of pipe with a hook welded onto it makes the attachment process easier. This pipe should have male threads, so that it can be attached to the exposed female end of each section of drill pipe still in the ground (Figure 6). Similarly, a hook welded onto the sleeve pipe used to prevent buckling simplifies the removal of the sleeve each time a pipe section is added.

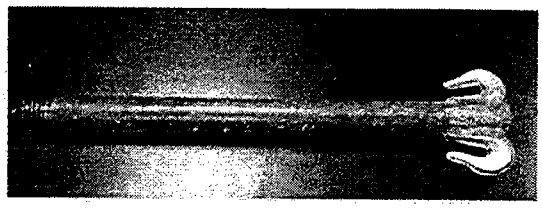

Figure 6. Pulling pipe which attaches to the drive pipes by threads at the left end, and attaches to the trackhoe or bulldozer by a chain looped through the hooks on the right end.

TEST EMBANKMENT WORK

Using a bulldozer, drains were initially installed and tested in an instrumented embankment constructed from sandy clay. The embankment is approximately 60 cubic yards in volume and has a 1:1 front slope face. It is instrumented with six piezometers, 16 nested soil moisture meters and 20 survey markers. One-half of the slope was stabilized with six wick drains, and the other half of the slope was not stabilized, so that it could be used as a control point in the experiments. Figure 7 shows the embankment and wick drain arrangement. A ground-water table was developed in the slope by induced infiltration through a trench at the back of the slope. The slope was then subjected to an increasingly intense simulated rainfall event, using sprinklers to produce a 100-year 24-hour rainfall (0.31 inches/hour, as shown on

Figure 7. Test embankment layout. The left side was stabilized with six wick horizontal wick drains (exiting the slope encased in the white PVC pipes attached to black hoses). The right side was not stabilized. Vertical white pipes are shallow piezometers.

Figure 8). During this simulation, water levels, wick drainage, and slope movement were measured and recorded.

The results of this initial testing are encouraging, with three lines of evidence supporting the effectiveness of the wick drains. First, the drains removed approximately 10 gallons of water per hour apiece. This is an equivalent of 0.11 inches of rainfall per hour, over the slope area affected by the wick drains (40% of the total slope area). This drainage rate is well below the estimated capacity of the wick drains, which is 96 gallons per hour (Bergado, et al., 1994). It is likely that the low permeability of the embankment material controls the drainage rate of the wick drains.

Second, the piezometers show lower ground-water levels in the vicinity of the wick drains (Figure 9). This effect was pronounced at the slope face and in the middle of the slope throughout the duration of the test. It was not apparent at the rear of the slope, where piezometers may have been in direct contact with a sand trench installed to enhance infiltration and creation of a water table (Liljegren, 2000).

The third line of evidence supporting the effectiveness of the drains is based on the movement of survey stakes: the stakes show

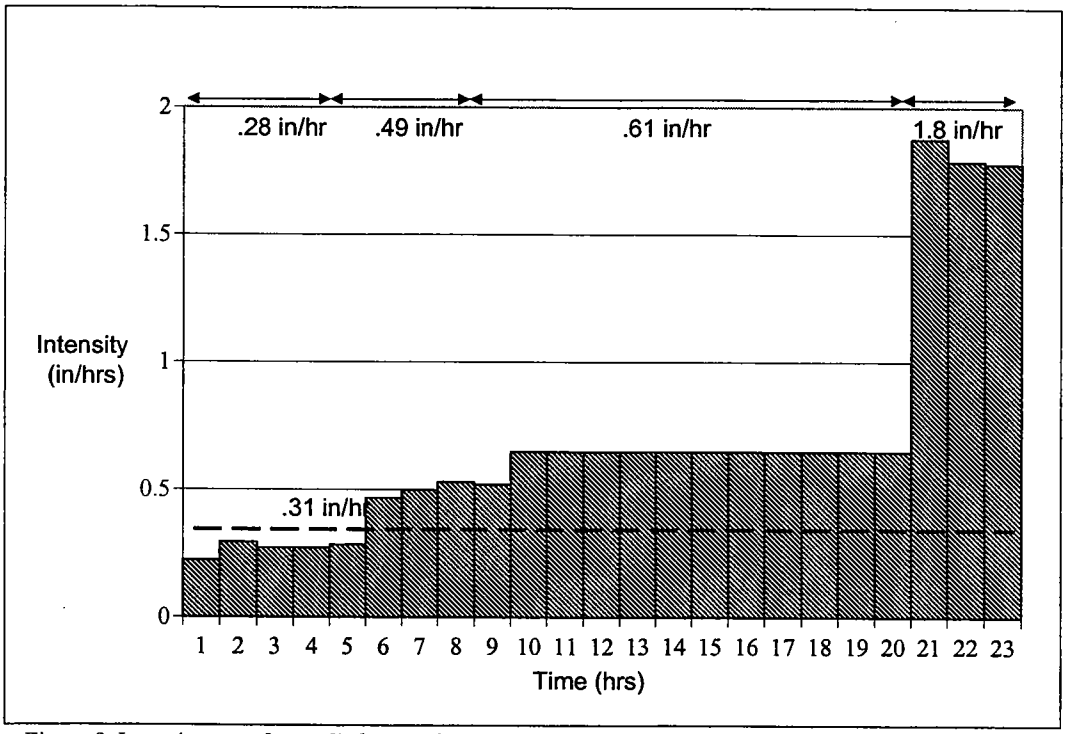

Figure 8. Intensity rates for applied water during simulated rainfall experiment (from Liljegren, 2000). The 100-year 24-hour rainfall is 0.31 inches/hour.

substantially less movement within the drained half of the slope. Total movement was measured and showed that the undrained half of the slope had moved downward or outward from 0.4 to 0.6 feet, and the drained half had moved less than 0.45 feet throughout, and less than 0.2 feet in close proximity to the drains (Liljegren, 2000).

Drains may be driven through rocky or hard zones several feet in width, although these zones often deflect the drain pipe toward the ground surface or completely halt the driving progress.

Wick drains have visibly reduced water levels at the Meeker South and Jasper landslides, and water drainage has been observed at the

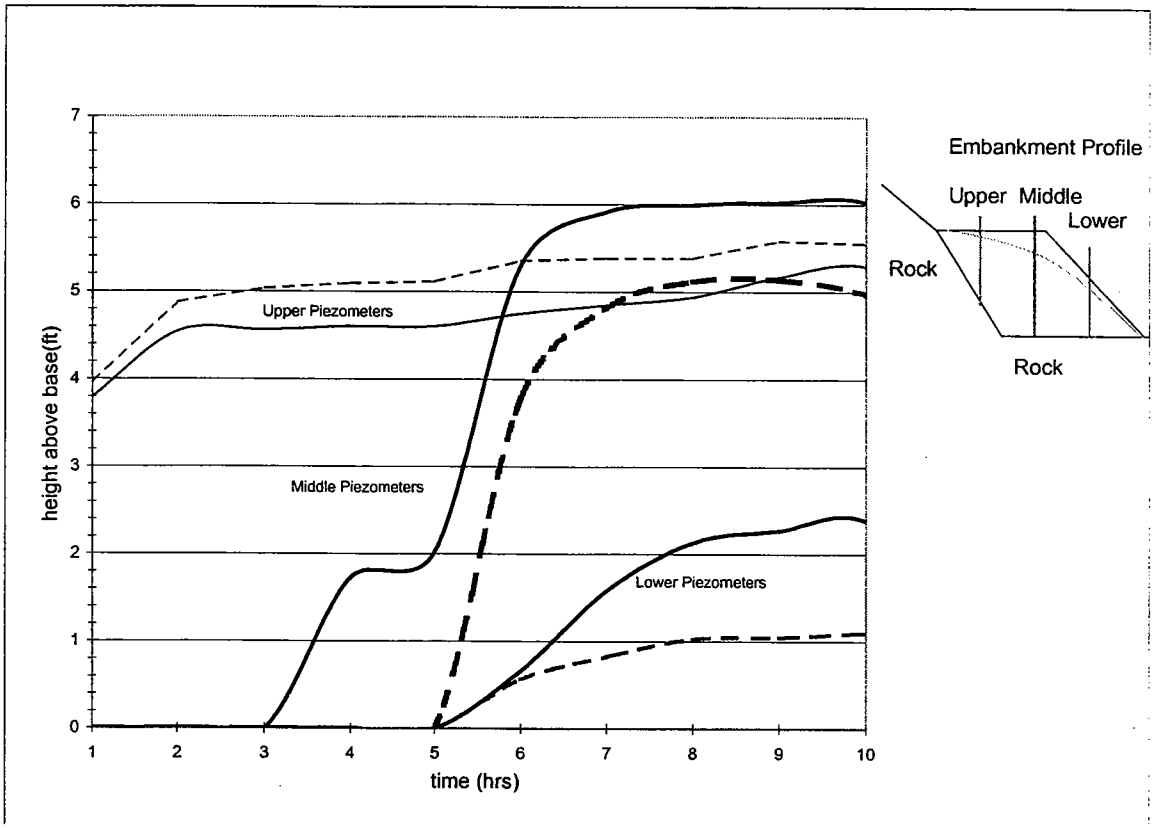

Figure 9. Water levels during the simulated rainfall experiment (from Liljegren, 2000). Dashed lines are piezometer readings from the stabilized side of the embankment and solid lines are readings from the unstabilized side.

HIGHWAY LANDSLIDE WORK

Following installation of wick drains at the test embankment, drains were emplaced at several locations with varying geology and using various types of driving equipment. Early efforts focussed on proving the feasibility of the wick drain driving method and on refining the installation technique. The latest installation work near Jasper, Indiana, was intended to be a complete landslide remediation project, with drain length and layout sufficient to impact the entire slide mass. A summary of each site is included in Table 1.

Drains were driven through a variety of natural and fill materials with SPT values as high as 28 blows per foot, although 20 blows per foot appears to be the realistic limit for longer drains. Boonville, St. Joseph, Meeker North and South, and Jasper landslides (Figure 10). Drainage rates have been as high as five gallons per minute at Meeker North and one gallon per minute at Jasper. Some minor drainage is identified as moisture on the wick which evaporates off the wick without ever falling to the ground.

Drains at the Rio Blanco landslide were installed too high to intercept ground water because of limited access points to the landslide. Drains at the Rye landslide were too short to intercept ground water, and later installation of an uphill cut-off trench lowered the ground-water table below wick levels (the maximum length of the Rye drains was limited to 40 feet because a standard wick drain driving crane was used to push the drains horizontally into the hillside).

Table 1. Summary of Wick Drain Installation Projects

Site	Location	Date	Geology	Driving Equipment	Number of Drains	Length (feet)
Rolla, MO (test embankment)	Fraternity Circle, N. of I-44 exit 185	8/98	Sandy clay fill over limestone	International Harvester (unknown model, approx. 15,000 lbs. operating weight)	6	20 (120' total)
Boonville, MO	S. side of Eastbound I-70, ¼ mile E. of exit 101 (S.R. 5)	12/98	Sandy clay and clay fill with 2'-7' thick cobble layer over limestone and shale	Case 550E bulldozer Case 9030B trackhoe	10	18-40 (249' total)
St. Joseph, MO	S. side of Eastbound I-229, ½ mile E. of exit 4 (E. Lake Blvd.)	5/99	Clayey silt loess overlying residual clay and weathered limestone and shale	Viatallis FX 130 trackhoe	7	40-67 (327' total)
Rio Blanco, CO	S.H 13, mile 15.7	6/99	Clay and clayey silt fill over weathered shale and siltstone	Caterpillar M318 wheeled excavator	6	40-50 (255' total)
Meeker S., CO	S.H. 13, mile 47.5	6/99	Clay and silty clay fill over claystone	Caterpillar D4C XL bulldozer	6	70-80 (430' total)
Meeker N., CO	S.H. 13, mile 47.7	6/99	Clay and silty clay fill over claystone	Caterpillar D4C XL bulldozer	5	48-70 (305' total)
Rye, CO	N. side of S.H. 165 1 mile N. of Rye	6/99	Clay fill	Caterpillar 215B LC trackhoe with vertical wick driving boom	21	18-40 (559' total)
Jasper, IN	E. Side of S.H. 545 3 miles S. of Dubois	6/00	Up to 21' silty clay and clay over weathered shale, limestone, and sandstone	Komatsu PC200LC	44	20-100 (2613' total)

Drains installed in 1998 and 1999 have showed no evidence of clogging with soil fines or algae, except where the drains lay directly on the ground surface and have been trampled. At locations where a short PVC pipe was used to encase the drain and was inserted a few feet into the soil, the drains were in excellent condition.

Continued monitoring of the drains will include periodic observations of drain conditions, water levels, and slope conditions for the Missouri and Colorado sites (installed in 1998 and 1999). The Indiana site (installed in 2000) will be more closely monitored through eight piezometers and two inclinometers.

SUMMARY

For agencies and companies interested in using horizontal wick drains for landslide stabilization, our work to date has shown that:

1. Equipment to install the drains is inexpensive and easily procured.
2. Drain installation is quick, inexpensive, and easily learned by untrained crews.
3. Pipe flexure upon encountering hard materials is the most significant installation problem. It may be controlled by increasing drive pipe diameter and wall thickness, using rigid pipe sleeves, and by bracing from underneath.
4. Drive plate damage may be reduced by using more robust materials when rocky or hard terrain is expected.
5. Test embankment and landslide stabilization work has shown that significant drainage is possible from driven horizontal wick drains.

ACKNOWLEDGEMENTS

Funding for this work was provided by the NCHRP-IDEA Program of the Transportation Research Board, Grant NCHRP-57, the

Figure 10. Water exiting from wick drains (Inset: Close-up of water drainage from a wick drain).

University of Missouri Research Board, and the Appleyard Fund of the UMR School of Mines and Metallurgy. Test sites were stabilized in partnership with the Nilex Corporation, the American Wick Drain Corporation, the Indiana, Missouri, and Colorado Departments of Transportation, and the Colorado Geological Survey.

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A GEOTECHNICAL INVESTIGATION OF STORM-INDUCED LANDSLIDES ALONG A SECTION OF INTERSTATE 77, SOUTHEASTERN OHIO

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ABSTRACT

On the night of June 27, 1998, a series of thunderstorms devastated Noble and Washington counties in southeastern Ohio. The storms dropped 6.5 inches of rain in a 72-hour period, causing extensive flooding, six deaths, and 179 million dollars in property damage. The total amount of rain in this region for the month of June was recorded to be 11.8 inches, the second highest monthly total recorded during the last 100 years. In addition to loss of life and property, the storms triggered more than 60 shallow (8-14 inches in depth) landslides along a 33.5 mile stretch of Interstate 77 between Belle Valley and Marietta, Ohio. The landslides occurred in colluvial soils that mantle the road cuts along the Interstate. Almost all of the landslides were translational in nature, occurring along the contact between the colluvial soil and the underlying bedrock. The bedrock in the region consists predominantly of shales, claystones, and mudstones, with alternating layers of limestones, sandstones, and siltstones. The purpose of this study was to establish the threshold values of precipitation required to trigger shallow landslides in the colluvial soils along Interstate 77.

Six sites in the affected area were selected for a detailed study of the shallow landslides. At each site, a record was made of the geomorphic setting, stratigraphy, slope geometry, number of landslides, dimensions of landslides, hydrologic conditions, and nature of vegetation. Both colluvial soils and the underlying bedrock were sampled for laboratory tests which included determination of natural water content, grain size distribution, slake durability, permeability, and shear strength parameters. All tests were performed according to ASTM specifications.

Data from laboratory tests were used to perform stability analyses with respect to varying thicknesses of saturated colluvial soils. Relationships between slope angle, thickness of saturated zone, and factor of safety were plotted as graphs. Due to the unique sequence of meteorological events during the month of June, 1998, the antecedent rainfall requirement for the colluvial soils was fulfilled. The storm events that produced 6.5 inches of rain over a 72-hour period and 4 inches in a 24-hour period were sufficient in duration and intensity to induce failures along Interstate 77. The results of the stability analyses show that the shallow landslides in the study area occurred when the colluvial soils reached 65% to 100% saturation.

INTRODUCTION

A line of severe thunderstorms, associated with a cold front, devastated Noble and Washington counties in southeastern Ohio on the night of June 27, 1998. The headlines in the local newspapers read as follows: "It's a weekend residents of southeastern Ohio hesitate to recall, but can not ever forget." "Measured in dollars and human lives, the storm that slammed into the Mid-Ohio Valley the weekend of June 27th was the costliest storm of this decade and one of the worst this century." (Hrach, *Marietta Times,* 06-27-99); "In a hellish weekend of wind and rain, the entire region was washed with nearly 10 inches of rain, which downed trees, electrical lines and caused extensive flooding." (Taylor, *Marietta AM,* 06-27-99). By June 29, Washington and Noble counties were operating under a state of emergency. The storms caused six deaths and a total damage of 179 million dollars for the 23 counties in southeast Ohio. A rain gauge station in Marietta, Ohio, recorded four inches of rain in a 24-hour period between June 27 and 28, 1998 (U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1998). This quantity of rain has a recurrence interval of twenty-five years (Lessig et al., 1977). Moreover, the total rainfall for the month of June, 1998, was 11.81 inches, which is the second highest monthly total recorded in the last 100 years (Schuck, *Marietta Times,* 06-27-99).

In addition to loss of life and property, the storms triggered more than 60 shallow landslides along both northbound and southbound lanes of Interstate 77 in Noble and Washington counties. Figures 1 and 2 show typical examples of these landslides. A preliminary survey revealed the slides occurred

Figure 1. An example of storm-induced shallow landslide along Interstate 77. Notice the shallow, translational nature of the landslide and the underlying bedrock serving as the failure surface.

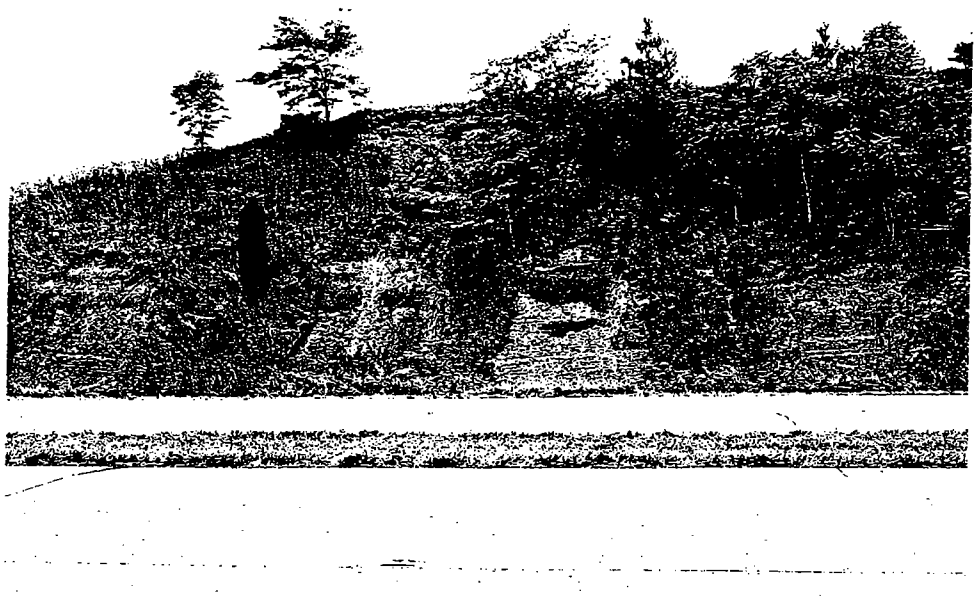


Figure 2. An example of multiple storm-induced shallow landslides along Interstate 77, showing well-defined boundaries and the influence of vegetation on their distribution.

along the contact between the colluvial soil, a poorly sorted mixture of rock fragments and fine-grained material, and the underlying bedrock. Although the slides did not threaten life or contribute to any structural damage to bridges or roadways, they did require considerable attention. The slide debris had to be removed from the catchment ditches and shoulders of the affected portions of the interstate and the landslide scars had to be reseeded to provide stability, prevent further erosion, and minimize pore pressure development. According to the Ohio Department of Transportation (ODOT), District-10 Office, the total cost for clean up and remediation of landslides along Interstate-77 was $39,014. The cost for removing debris and cleaning catchment basins totaled $24,564, and for reseeding and mulching the landslide scars was $14,449 (written communication, ODOT district-10, Pedigo, 2000).

A number of studies on slope instability have been conducted in the study area by previous researchers (Fisher et al., 1968; Hanson, 1986; Pomeroy, 1987; Mast, 1980, Weber and Shakoor 1985), but not in direct relation to storm-induced landslides. The primary focus of the previous studies has been on mapping of landslide-susceptible stratigraphic units. As storm-induced landslides have not previously occurred along Interstate 77, the study presented herein was undertaken to investigate these landslides in detail with an emphasis on the threshold of rainfall required to trigger such landslides.

Washington and Noble counties are located in southeastern Ohio with Washington County bordering the Ohio River and Noble County bordering Washington County directly to the north (Figure 3a). Interstate 77, which runs through both Washington and Noble counties, is a major north-south

highway originating in Cleveland, Ohio, and terminating in Charlotte, North Carolina. The two counties are located within the unglaciated portion of the Appalachian Plateau province, which covers much of western Pennsylvania, West Virginia, and eastern Kentucky. This area of Ohio is dissected by the tributaries of the Ohio River drainage system, with an average relief of approximately 350 feet (Fisher et al., 1968). The rocks of the area range from Pennsylvanian to Permian in age (Figure 3b). The general strike is northeast southwest with an average dip of less than 1 degree (Lessig, et al., 1977).

The Pennsylvanian age rocks exposed within the two counties consist of Conemaugh and Monongahela Groups which include interbedded sandstones, limestones, siltstones, shales, calcareous mudstones, calcareous claystones, and coal. The Permian age rocks, overlying the Monongahela Group, belong to the Dunkard Group which consists of varying proportions of interbedded sandstones, siltstone, claystones, shales, and coal layers (Shultz, 1999). The Dunkard Group outcrops more extensively in Washington County.

FIELD INVESTIGATIONS

Detailed field investigations were undertaken to determine the distribution and nature of the shallow landslides present along Interstate 77. Six sites, located between Belle Valley, Ohio, and Marietta, Ohio, were chosen for detailed study (Figure 4). Four of the six sites were selected along the south-bound lane (east-facing cuts) and the other two sites along the north-bound lane (west-facing cuts). Preliminary investigations indicated more than 60 landslides along this 33.5 mile stretch of the highway. However, most of the slope failures were very localized, with as many as 10 slides present at site 1. Selection of the six sites was based on the areas of landslide concentration. Field investigations for each site included determination of slope height, slope angle, number of slides present, slide dimensions, thickness of colluvial soil, and density of in situ colluvial soil. Representative samples of colluvial soil and unweathered bedrock were collected for laboratory testing.

The results of field investigations are summarized in Table 1. Most of the landslides are translational in nature, having occurred along the colluvium-bedrock contact, and are characterized by their shallow depths (8-14 inches) and well-defined boundaries (Figures 1 and 2). In some cases, the distribution of landslides on the slope is significantly influenced by the vegetation present, with the slides diverting around trees and shrubs (Figure 5).

LABORATORY INVESTIGATIONS

A series of laboratory tests were conducted to determine the engineering properties of the landslide-prone colluvial soil and the underlying bedrock. All laboratory tests were conducted according

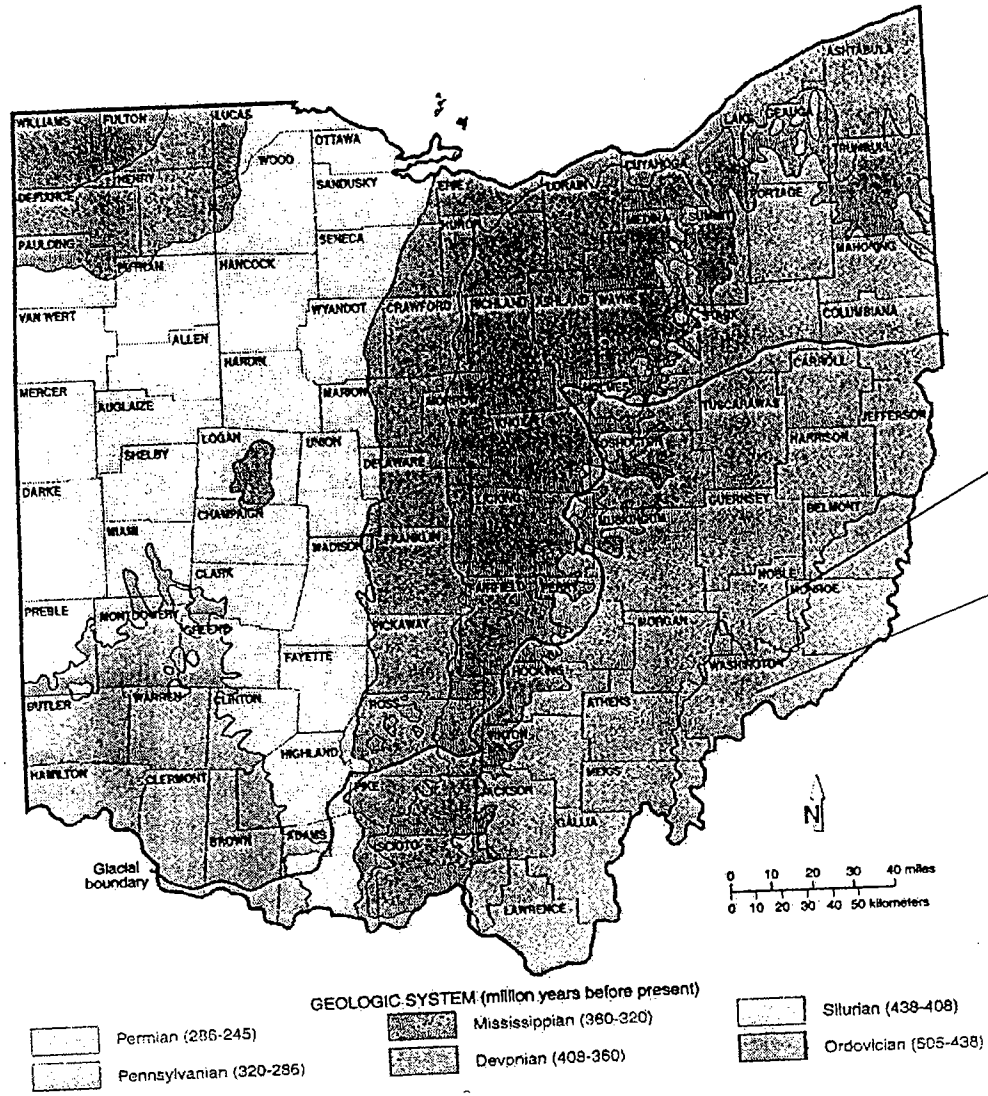


Figure 3a: Generalized geologic map of Ohio showing Washington and Noble Counties.

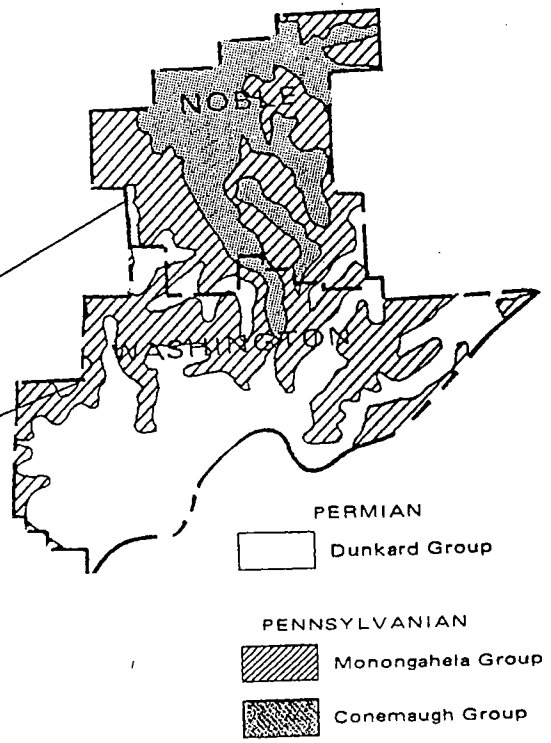


Figure 3b: Geologic map of Washington and Noble Counties.

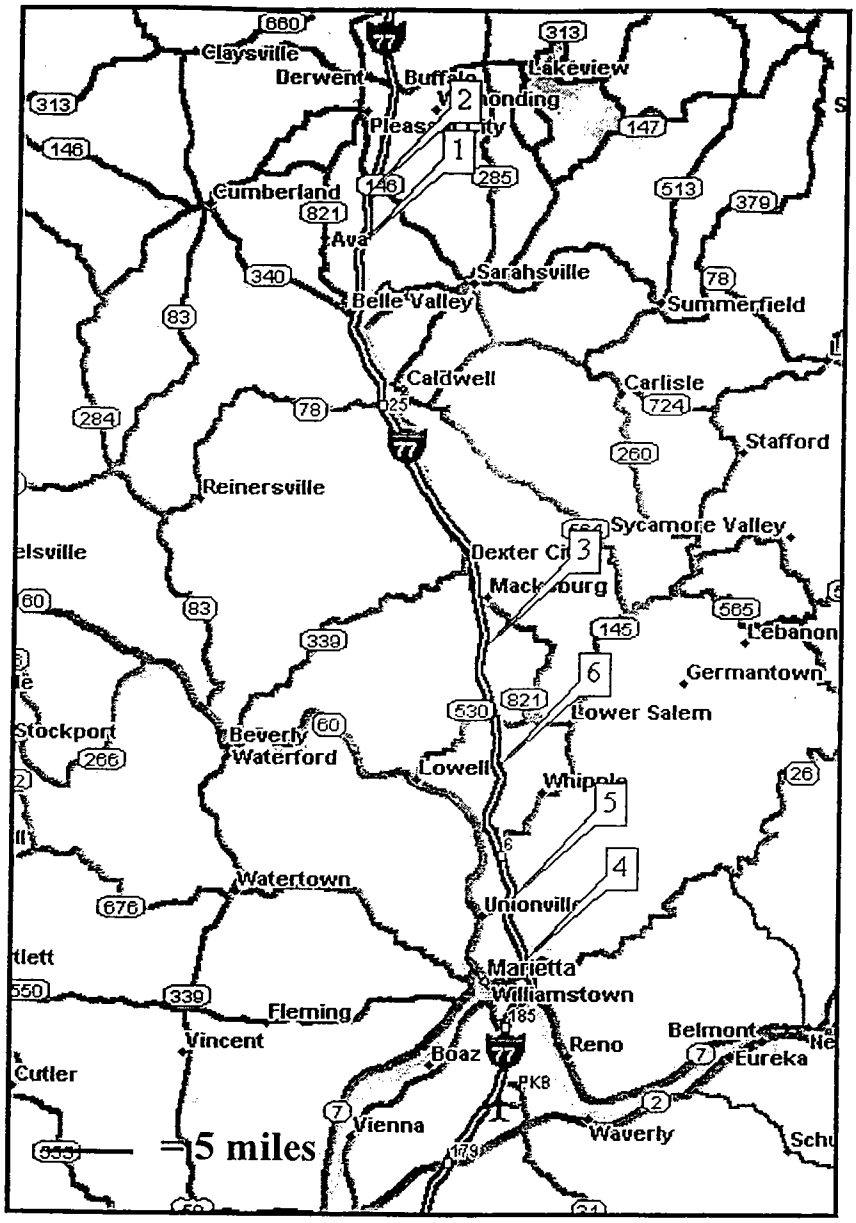


Figure 4. Map showing the location of the six study sites.

to the American Society for Testing and Materials (ASTM) specifications (ASTM, 1996). The properties determined included natural water content, Atterberg limits, grain size distribution, permeability, and shear strength parameters.

Soil Properties

The average engineering properties of the colluvial soils for the six sites are listed in Table 2. Natural water contents range from 12.6% to 4.3%. The hydrometer analyses, performed on material passing the #200 sieve, indicate an average silt content of 68% and a clay content of 32%. Permeability test results

Table 1: Summary of the results of field investigations at the six sites along Interstate 77.

Site No. Mile Post Direction	Site Length (feet)	Slope Height (feet)	Slope Angle (degrees)	No. of Landslides present	Dimensions of Landslides			Underlying Bedrock Group Name
					Length* (feet)	Width** (feet)	Depth (feet)	
1 #33.5 N-bound	650	50	45	10	20-55	20-30	2/3-1	Conemaugh
2 #34 S-bound	700	80	39	7	8-45	10-40	1	Conemaugh
3 #10 S-bound	215	60	43	8	10-40	10-30	<1	Monongahela
4 #2 S-bound	200	60	40	3	25-30	15-30	1	Dunkard
5 #5.5 N-bound	300	70	41	8	15-40	7-40	>1	Dunkard
6 #15 S-bound	200	60	43	3	15-25	20-25	<1	Monongahela

* Dimension parallel to dip direction of slope (perpendicular to road direction)
** Dimension perpendicular to dip direction of slope (parallel to road direction).

Figure 4. An example of the influence of vegetation on location and distribution of storm-induced landslides.

show a range from $4.30x10^{-4}$ cm/sec to $1.69x10^{-4}$ cm/sec. Based on Atterberg limits, the fines present in the residual soils can be classified as clays of low plasticity (CL) and silts of low plasticity (ML). Dry densities, as determined by the sand cone method, range from 89 pcf to 102 pcf.

Table 2. Engineering properties of colluvial soils at the six sites.

Property	Site #1	Site #2	Site #3	Site #4	Site #5	Site #6
Natural Water Content (w_n), %	12.6	10.7	6.3	4.3	6.3	8.0
Liquid Limit (LL)	35.6	37.3	30.3	30.3	29.3	32.0
Plastic Limit (PL)	26.0	30.3	23.6	26.6	26.3	26.3
Silt Content, %	69	57	68	62	69	80
Clay Content, %	31	43	32	38	31	20
Permeability (k), cm/sec	$3.69x10^{-4}$	$1.69x10^{-4}$	$4.30x10^{-4}$	$3.05x10^{-4}$	$3.50x10^{-4}$	$2.50x10^{-4}$
Dry Density (γ), pcf	96	102	89	101	90	97
Friction Angle (Ø), degrees	19.7	28.3	26.2	32.1	30.1	32.4
Cohesion (c)*, psf	67.6	52	60.3	49.1	51.7	55.7

* Indicates back calculated values of cohesion

Bedrock Properties

As previously stated, the shallow landslides occurred along the contact between the colluvial soil and the underlying bedrock. With this in mind, bedrock samples for direct shear testing were cut, prepared, and placed in the shear box of the direct shear test apparatus in such a manner that a planar surface served as the plane of shearing. Colluvial soil was placed on top of the rock sample and compacted to its calculated field density. It is believed that this methodology simulated the field conditions at the time of failure. Results of these tests are shown in Table 2. The friction angle values range from 19.7° to 32.4°. These values compare well with previous research (Mills et al., 1987). Values of cohesion generated from these tests vary from 55.7 psf to 427 psf. Due to the large range and relatively high values of cohesion, the cohesion values were considered questionable. Additionally this range of values for colluvial soils was not in agreement with previously published data (Mills et al., 1987).

Since cohesion values obtained from laboratory tests appeared to be questionable, back calculations were performed to obtain cohesion values for the corresponding laboratory values of friction angle. This was done taking into account the worst-case scenario where the soil density was assumed to be saturated at the time of failure. The equation for the infinite slope analysis, discussed later under stability analysis, was used for back calculations. Table 2 shows the results of the back calculations.

STABILITY ANALYSIS

A stability analysis was conducted for one translational landslide at each site to evaluate the relationship between slope angle and degree of saturation required to cause failure. Since the failure surfaces for these slides are parallel to the slope faces, the infinite slope analysis was used for this purpose. The method quantitatively analyzes the influence of soil saturation on the stability of a slope. It assumes the slip surface to be much longer compared to its depth, and ignores the driving and resisting forces at the upper and lower ends of the sliding mass, respectively. As the resisting force is ordinarily greater than the driving force, the analysis is considered to be slightly conservative (Turner and Schuster, 1996). The factor of safety against sliding is calculated by the following equation:

$$F.S. = (A)(\tan\emptyset'/\tan ß) + (B)(c'/\gamma H)$$

where: c′ and Ø′ = effective strength parameters for the failure surface

ß = slope angle

A and B = dimensionless stability coefficients determined from graphs shown in Figure 6.

γ = soil density

H = soil depth (measured vertically from slope face to failure surface

With the dimensions of the landslides recorded and the in situ densities determined, the weight of the soil mass involved in sliding was readily calculated. The thickness of the soil mantle varied at different locations within the slope so an average thickness of 1 foot was used for the purpose of analysis. The slope angles for the study area varied between 39-43 degrees. Therefore, β values of 39 to 43 degrees were used in the above equation.

The results of stability analyses are shown in Figure 7, which indicates an inverse relationship between slope angle and saturated fraction of the colluvial soil thickness. Figure 7 shows that sites 3 and 6 exhibit the highest level of stability where a saturated fraction of 0.9 (90% saturation) is needed to induce failure for a slope of 45°. Sites 2 and 4 display a lower level of stability with a saturated fraction of approximately 0.65 (65% saturation) needed to induce failure for a 45° slope. Figure 8 provides similar plots for a factor of safety of 1.3.

RAINFALL INTENSITY - DURATION THESHOLDS FOR INTERSTATE 77 LANDSLIDES

The rainfall threshold graph developed by Caine (1980) was used to evaluate the rainfall intensity-duration relationship that triggered the shallow landslides in the Marietta-Belle Valley area.

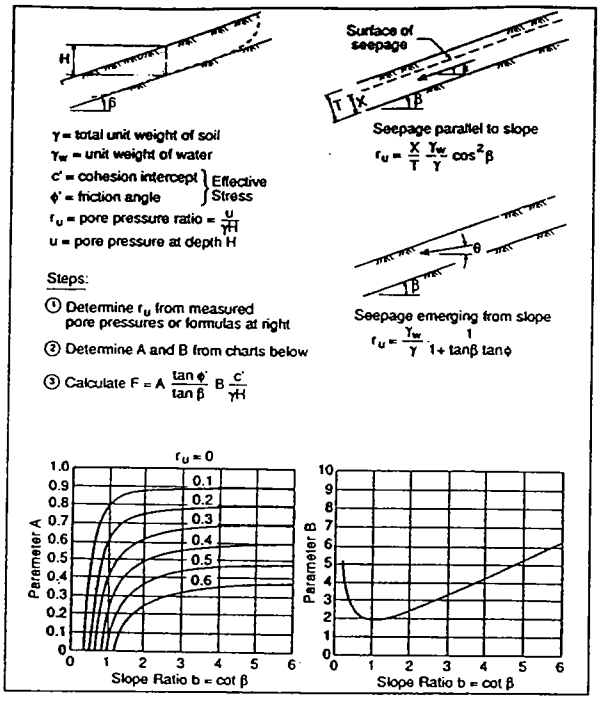
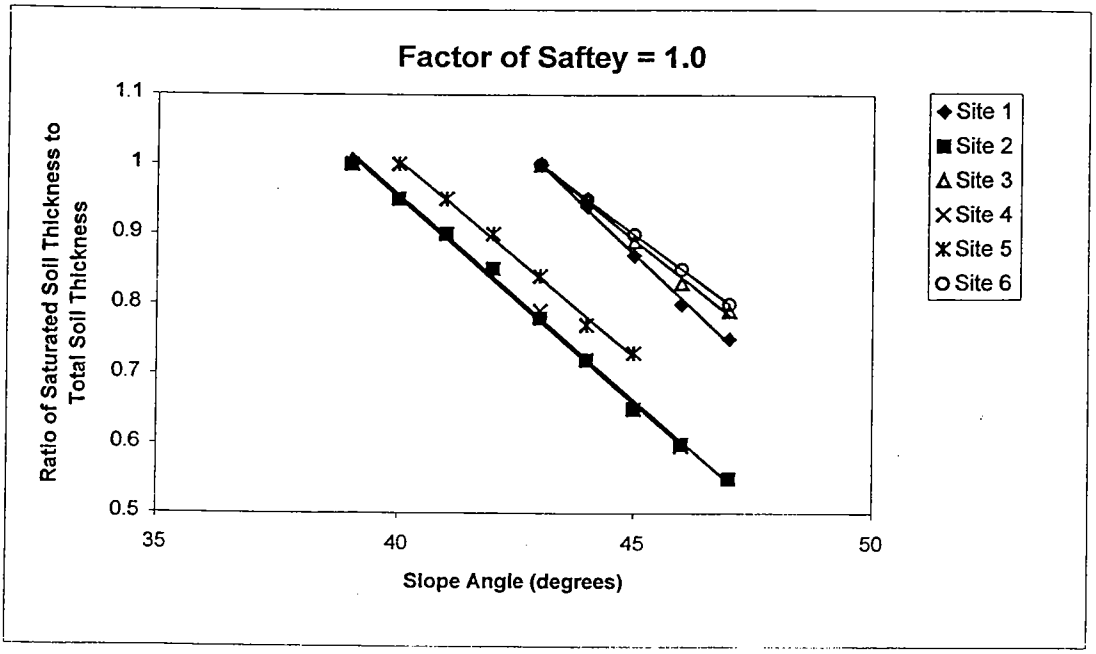


Figure 6. Graphs required for determination of parameters A and B (Turner and Schuster, 1996; modified from Duncan et al., 1987).



Figures 7. Relationship between saturated soil fraction, slope angle, and factor of safety for failure conditions.

Caine's graph is based upon 73 case histories of landslides worldwide, representing a wide variety of geologic and physiographic conditions. The rainfall intensities corresponding to 24, 48, and 72 hour durations were used for this comparison. Rainfall data collected at the Marietta waste water treatment

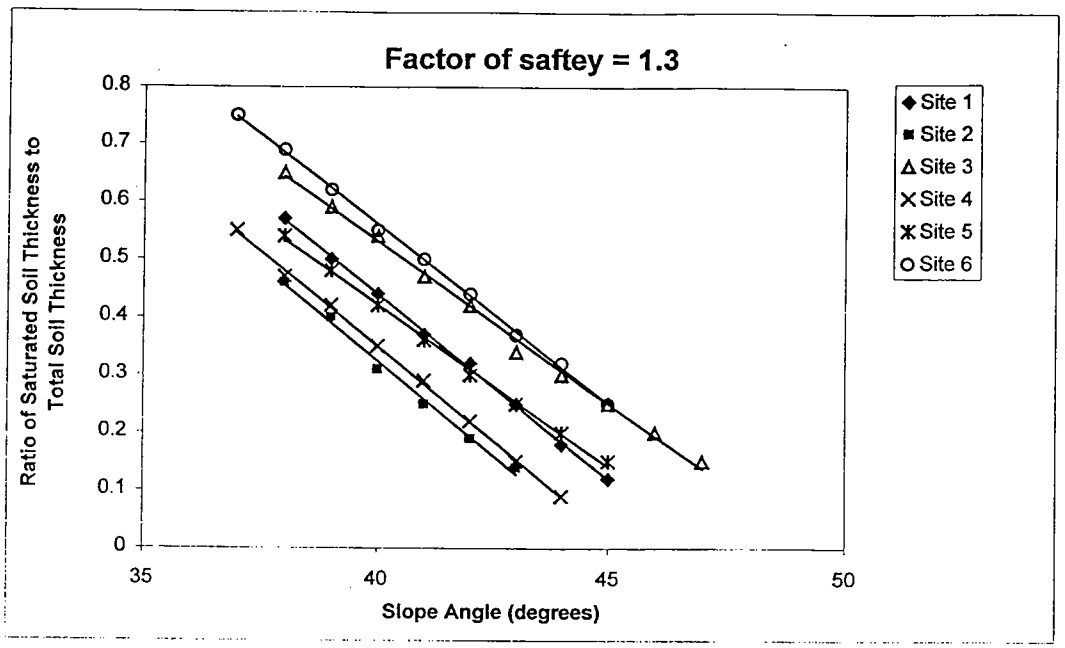


Figure 8. Relationship between saturated soil fraction and slope angle for a factor of safety of 1.3.

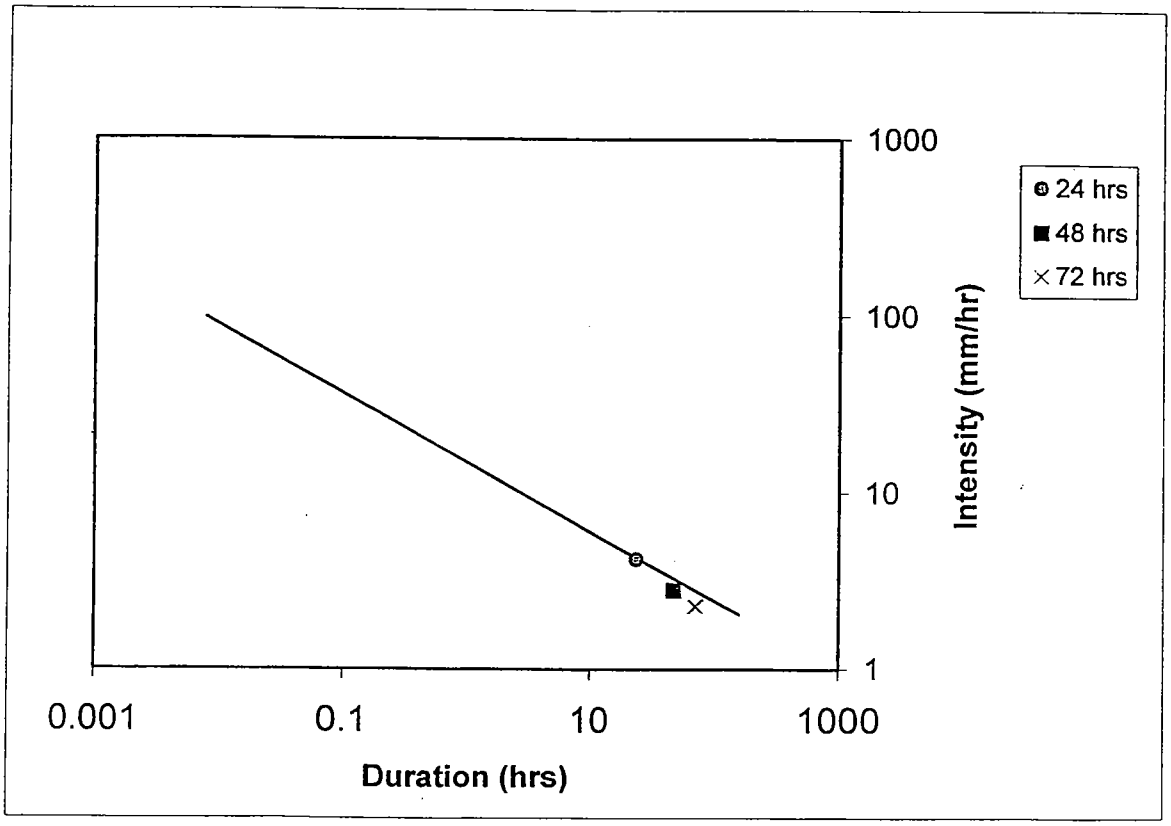


Figure 9. Relationship between storm intensity and duration (straight line) as established by Caine (1980). Notice the close agreement between Caine's graph and the rainfall data for 24-, 48-, and 72-hour periods from the Marietta area, Washington County, Ohio.

plant for the three-day storm event (U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1998) were plotted on the threshold graph by Caine (1980) (Figure 9). The equation of the graph takes the form:

$$I = 14.82D^{-0.39}$$

where: I = rainfall intensity in mm/hr and
D= rainfall duration in hours

Figure 9 shows that rainfall data from the Marietta area closely agrees with the threshold graph. The recorded intensity of rain for a 24-hour period was 4.2 mm/hr whereas the threshold value required is 4.3 mm/hr. The 48- and 72-hour durations, with intensity values of 2.8 mm/hr and 2.3 mm/hr, respectively, show a slightly weaker agreement with Caine's minimal thresholds of 3.2 mm/hr and 2.7 mm/hr. Caution should be exercised when using Caine's graph, as the rainfall intensity can be highly variable over a given area. Also, hourly precipitation totals, which are more accurate when establishing threshold values, were not available for the study area. Because a 24-hour time period could contain significant segments with little or no precipitation, the developed pore pressures have the opportunity to dissipate. Therefore, a continual recording of the downburst is essential.

CONCLUSIONS

The conclusions of this study can be summarized as follows:

1. The storm-induced landslides along Interstate-77, through Washington and Noble counties in Ohio, can be categorized as shallow translational failures that occur along the contact between the colluvial soil and the underlying bedrock.
2. Storm-induced translational slides require a continuous mantle of colluvial soil and a slope angle range of 35 to 55 degrees to occur. Slopes steeper than 55 degrees normally do not support a continuous colluvial mantle.
3. The landslides in the study area were initiated when the soils were 65% to 90% saturated depending upon their slope angles.
4. The vegetation present on the slopes plays an important role in the distribution of failures. Shallow landslides are frequently diverted around trees and shrubs. Generally, a root network contributes to soil strength by providing an additional cohesion component.
5. The red mudstones and claystones, belonging to the Conemaugh, Monongahela, and Dunkard Groups, that comprise a considerable portion of the stratigraphy of the area, are solely responsible for the formation of landslide prone colluvial soils.

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A TIMELY SOLUTION FOR THE NOJOQUI GRADE LANDSLIDE REPAIR US 101 SOUTH OF BUELLTON, CALIFORNIA

Sara von Schwind Connor, P.E.[1] and Glen A, Gorski, P.E.[2]

ABSTRACT

On February 23rd, 1998, the northbound lanes of Route 101, in an area known as Nojoqui Grade in Santa Barbara County, California, were shut down due to a landslide. With all other north/south routes in the area already closed due to mudslides, rockfall, bridge scour and landslides precipitated by significant El Nino rains, urgent slide repair and re-opening of Route 101 was critical. After considering and rejecting several alternatives for slide repair, the California Department of Transportation (CalTrans) accepted a proposal to stabilize the slope by the installation of Vibro-Replacement stone columns. Construction began on March 5th, 1998 and the northbound lanes were re-opened to traffic by March 24th, 1998. This paper discusses site history, site investigation, alternatives considered, analysis and design of the stone column program, and the construction process.

INTRODUCTION

On February 23, 1998, a slide occurred in the northbound lanes of Route 101 in Santa Barbara County, California near the Nojoqui Grade (Figure 1). Maintenance forces re-routed the northbound traffic to the southbound roadbed, creating one northbound and two southbound lanes. Due to the significant rains during the "El Nino" winter of 1997/1998, all other

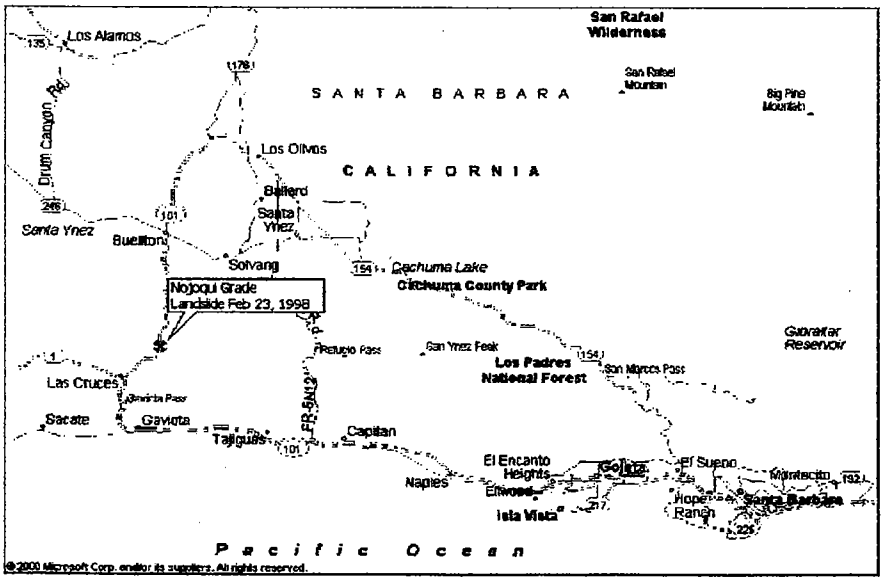

Figure 1. Site Location

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north/south routes in the area had already been closed because of mudslides, rockfall, bridge scour, or landslides. Since Route 101 was the only north/south route open at the time of the slide failure, repairing the slide and reopening the route was vital. Emergency funds were allocated, with emphasis for the repair solution placed on timeliness of the design and construction.

SITE DESCRIPTION

The alignment of Route 101 in the area of the slide was constructed by cutting into the existing hillside. The southbound lanes are founded on a cut surface while the northbound lanes were founded on embankment fill material, some 15 feet (4.6 m) deep, placed on the existing slope. The resulting roadway subgrade was approximately 40 feet (12.2 m) above the toe of the slope. The embankment slope beneath the roadway ends on a slight shelf above a small creek which approximately parallels the roadway in the slide area. Several horizontal drains had been previously installed to reduce water pressure in the fill material north of the slide. A 36-inch (0.9 m) culvert carrying storm water from the uphill side of the roadway ran under the roadway in the area of the slide and emptied into the creek.

SITE INVESTIGATION

On February 23, 1998, the Geotechnical Branch of CalTrans conducted a preliminary site investigation of the reported slide mass. The headscarp of the slide was located in the no. 1 northbound lane (Figure 2) and was about 6 inches (152 mm) high at the time. A circular pattern of cracks in the roadbed surface was also visible. The slide mass extended from the

Figure 2. View of Headscarp (Northbound Lanes) Looking South

road surface to the creek located below the roadway embankment (Figure 3). Water was observed flowing from the slope approximately 6 feet (1.8 m) above the creek flow line. The slope rising up from the opposite side of the creek also showed evidence of instability and water seepage. The right of way fence within the slide mass had been pushed over. Since the culvert inflow appeared equal to the outflow, it was assumed that the 36-inch (0.9 m) culvert running through the slide was not separated or broken. A cross section through the center of the slide mass is shown in Figure 4.

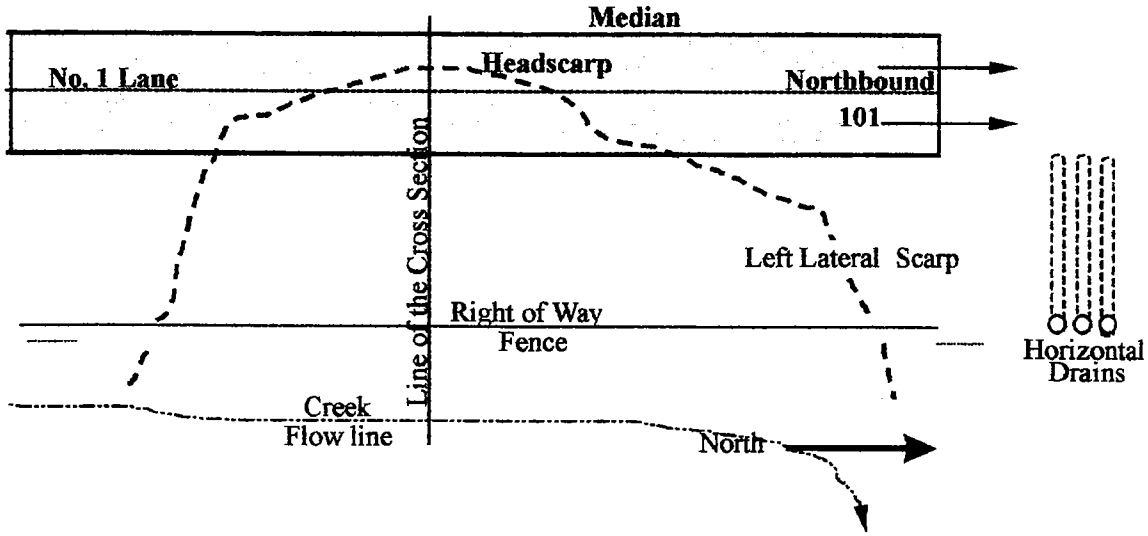

Figure 3: Plan View of Slide Limits

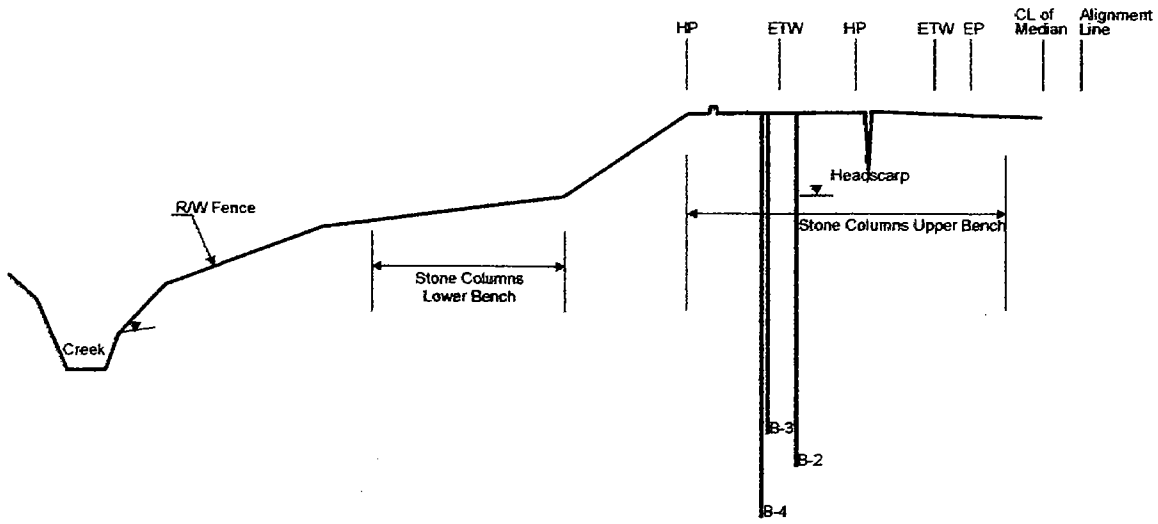

Figure 4. Cross Section

On February 24, 1998, a surface monitoring system was set up on-site to determine the rate of movement. This system consisted of five pairs of monitoring points installed on either side of the visible cracks in the pavement at the top of the slide (Figure 5). The vertical and horizontal difference between each set of nails was initially recorded and then checked each day. Graphs of the observed movements are shown in Figure 6.

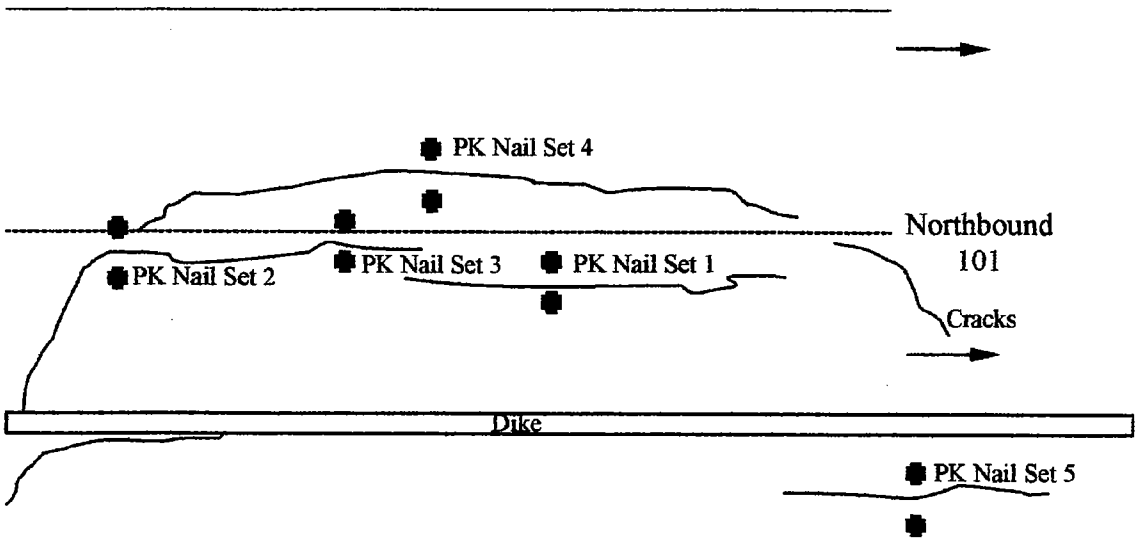

Figure 5. Monitoring Point Locations

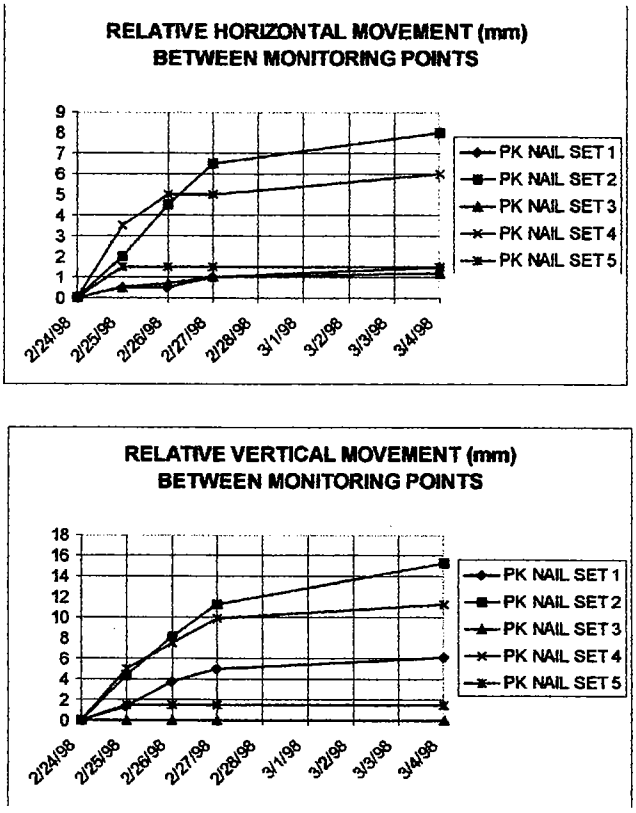

Figure 6. Graphs of Observed Movement

From the graphs, it can be seen that the slide mass was continuing to move. When construction began, the pavement surface was removed and the monitoring points were lost although the slide mass continued to move during the initial work at the top of the slope.

A subsurface investigation was conducted on February 25, 1998. Three borings were drilled using 8-inch (200 mm) diameter augers to depths of 50 and 55 feet (15.2 and 16.7 m). Piezometers were placed in borings B-1 & B-2. The piezometers in B- 1 indicated a ground water depth of 23 feet (7 m) below the ground surface outside the slide area while piezometer B-2 indicated a ground water table that was only 13 feet below the road surface in the actual slide itself. A time domain reflectometry (TDR) cable was placed in boring B-3 to monitor slope movement. Unfortunately, the cable was grouted in place using two grouts with substantially different strengths, and the TDR results were found to be unreliable.

REMEDIATION ALTERNATIVES

Based on the subsurface investigation it was determined that the slide mass was very soft to a depth of 35 to 40 feet (10.7 to 12.2 m), with a few lenses of stiffer material. No distinct slide plane was apparent. The mass appeared to be acting more as a soil flow. Several remediation alternatives were considered by CalTrans:

Soldier Pile Tie-Back Retaining Wall: This would require a fairly lengthy design and construction process due to the tie-backs that would be required. Tie-backs are necessary due to the proposed wall height of approximately 40 feet (12.2 m)and the anticipated loading from the slide mass.

Horizontal Drains: For drains to be effective they would need to be located just above the creek flow line. Placing the drains within this portion of the slope would require that the drill rig have access within the riparian habitat at the toe of the slide. However, the CalTrans Environmental Department stated that the riparian habitat located at the toe of the slide is considered a valuable resource and should not be disturbed.

Construction of a Buttress at the Toe of the Slope: The construction of a buttress has an even greater impact on the riparian habitat at the toe of the slope than the horizontal drains therefore this option was not viable.

Excavate the Slide Mass and Replace it with Quality Material in conjunction with Drainage: Due to the depth of the necessary excavation, there was a high potential for the failure to extend into the southbound lanes during excavation of the slide mass. Shoring was considered unreasonable and uneconomical due to the estimated height of at least 30 feet (9.1 m).

Since none of these options met the project requirements, they were rejected, and CalTrans contacted the local office of a national specialty geotechnical contractor experienced in ground improvement. The contractor proposed using the bottom-feed vibro replacement technique to install stone columns through the weakened slide mass to the underlying soils beneath the

slide. It was anticipated that the stone columns would reinforce and strengthen the entire mass of soil and provide lateral shearing resistance within the soil mass.

VIBRO-REPLACEMENT TECHNOLOGY

Vibro stone columns are designed to increase bearing capacity, reduce settlement, aid densification, reduce liquefaction potential and improve shear resistance. In the Vibro Replacement process, a vibrator penetrates the ground to design depth and crushed stone backfill is then introduced in discrete lifts. Repenetration of each lift, and the horizontal forces generated by the vibrator, laterally compacts the stone against the surrounding soil. The process is repeated up to ground level, forming a well-compacted, tightly interlocked stone column surrounded by soil of enhanced density.

The bottom-feed method uses the same vibrator as standard Vibro-Replacement, but with the addition of a hopper and supply tube to feed the stone directly to the tip of the vibrator. Bottom-feed vibro replacement eliminates the need for flushing water, in turn reducing the generation of spoil and extending the range of soils that can be treated.

Vibro-Replacement For Embankment Subgrade Improvement

Where embankments are to be constructed, Vibro stone columns installed through the calculated failure surface can increase the slope stability safety factor by introducing material with higher shear strength and concentrating sufficient loading to increase the overall shearing resistance.

The shear strength of treated soil depends on the shear strength of the untreated soil, the transverse shear strength of the columns, the area replacement ratio and the load conditions. Design methods typically use some average shear strength based on these parameters for analysis. Stability calculations are then analyzed by Bishop's, Janbu or other numerical analysis methods.

Of particular interest is the case of a planned embankment over soft soils. As the embankment is constructed over the Vibro stone columns, stress concentration occurs in the columns due to their modulus value being much greater than the soft, surrounding in situ soil. This stress concentration results in a high vertical stress in the column, thus greatly increasing the columns' shear resistance. Stress concentration factors of three or greater are common.

VIBRO STONE COLUMN PROPOSAL

The proposed solution consisted of two separate treatment areas within the project limits (Figure 7). The stone columns were not constructed on the slope between the two treatment areas to help preserve some of the existing slope vegetation and to avoid having to completely reconstruct the slope. The probe would be vibrated into the ground to refusal, which was estimated to be 35 to 40 feet (10.7 to 12.2 m), and then crushed angular gravel would be bottom fed through the probe and compacted in place.

The upper treatment area, at the roadbed elevation, extended from 50 feet (15.2 m) south of the right lateral scarp to 50 feet (15.2 m) north of the left lateral scarp. The lower treatment area, on the slope of the embankment above the creek, was planned to be the same length as the upper treatment area. The geotechnical contractor recommended the installation of a grid of 3 foot (0.9 m) diameter stone columns, 6 feet (1.8 m) on center, within both treatment zones. This layout provided for approximately 20 percent area replacement.

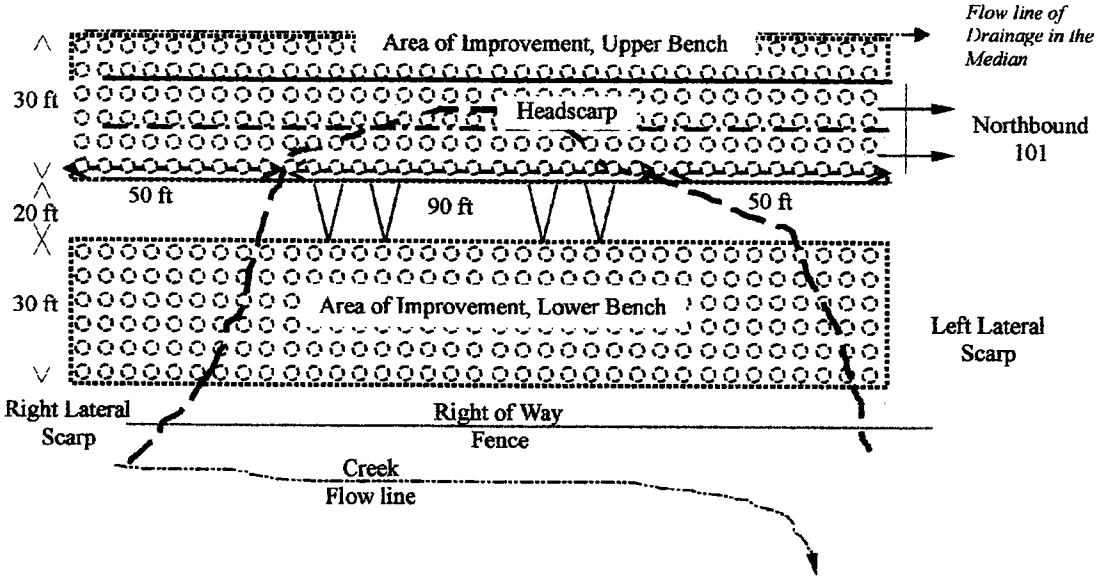

Figure 7. Proposed Limits of Stone Columns

DESIGN ANALYSIS

Using the cross section and soils information derived from the field exploration, a reasonable set of subsurface soil parameters were back-calculated, assuming a factor of safety of one for the existing slope. The soil parameters were estimated to be:

$$\phi = 19°$$
$$c = 50 \text{ psf.}$$

Two methods were utilized to determine the equivalent friction angle and cohesion of the soil mass with the proposed column layout.(Priebe, 1993; Mitchell, 1981) Using the Priebe method, resulted in the following equivalent soil parameters:

$$\bar{\phi} = 39°$$
$$\bar{c} = 13 \text{ psf.}$$

Using the method presented by Mitchell resulted in the following equivalent soil parameters:

$$\bar{\phi} = 39°$$
$$\bar{c} = 42 \text{ psf.}$$

Calculations using the soil parameter results from the Mitchell method show that the factor of safety was increased from approximately 0.95 to 1.46 after construction of the stone columns. This predicted factor of safety is most likely slightly greater than the actual factor of safety achieved because some of the stone columns did not reach the anticipated depth of 35 to 40 feet (10.7 to 12.2 m), as discussed below.

CONSTRUCTION

Installation of the stone columns began on Thursday, March 5, 1998. The stone columns were installed during two, 10 hour shifts per day, seven days per week. Stone columns were constructed along the upper bench first (Figure 8) so that the contractor could use the resulting reinforced mass as a staging area for working on the lower bench.

The upper bench (roadbed) continued to move horizontally and vertically towards the creek during the stone column construction. Due to a concern that the 36-inch (0.9 m) culvert, which ran beneath the traveled roadway and through the slide, could be feeding water into the slide mass, field personnel inspected the culvert. This inspection revealed that the pipe had separated at a joint with 6 to 8 inches (152 to 203 mm) of vertical displacement between the segments. The pipe was structurally sound and showed no signs of corrosion, which appeared to confirm that the movement had caused the break as opposed to the possibility that a broken culvert had led to the slide. Within a day, additional movement and possibly soil infiltration had plugged the culvert and started to back up water in the culvert. The prime contractor was directed to pump the water out and plug the catch basin to prevent additional water from flowing into the slide mass.

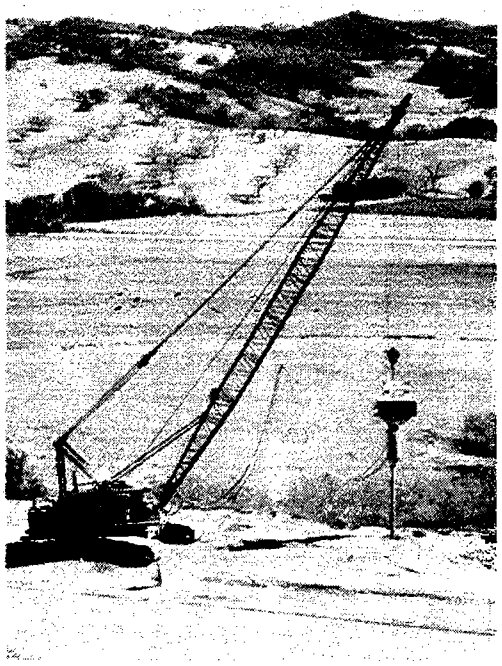

Figure 8. Stone Column Installation

Movement of the upper roadbed also raised concerns regarding setting up on these improved soils for stone column work on the lower bench.. A TDR cable was therefore installed in the middle of the slide at the roadbed level (see B-4 in Figure 4). CalTrans stayed on-site at the beginning of the lower bench stone column construction and determined that no significant movement was occurring according to the TDR readings. Construction of the lower bench columns was completed on the afternoon of March 15, 1998.

The columns did not extend laterally the full 50 feet (15.2 m) to either side of the failed area because dense material and refusal was encountered near surface in these locations, indicating competent material that did not need reinforcement. The final column locations are indicated on the stone column as-built (Figure 9).

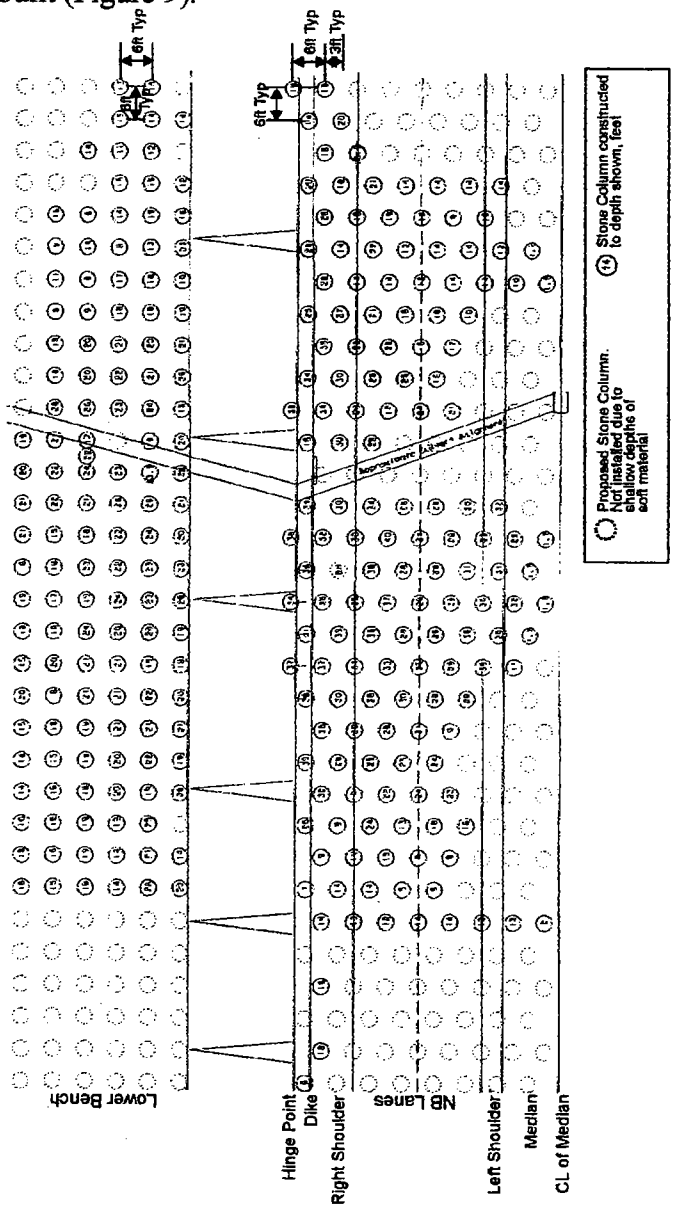

Figure 9. As-Built Stone Column Locations

LONG TERM MONITORING

To continue to monitor potential movement of the slope a permanent 60 foot (18.3 m) TDR cable was placed on May 4, 1999. One crack in the pavement has developed in the outside shoulder in the area where stone columns were not installed to avoid damaging the existing culvert. The site will continue to be monitored for signs of visible distress and slope movement with the TDR, but as of September of 1999, the slope is performing well.

SUMMARY

While introducing vibrations and heavy equipment to an already unstable slope is a risky proposition, the relatively small vibratory probe allowed the quick installation of granular material that improved the overall slope stability. The improved soil strength and drainage characteristics of the reconstructed slope allowed restoration to pre-slide configurations in a limited access area. Vibro-replacement provided an in situ ground transportation corridor.

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ACKKNOWLEDGEMENTS

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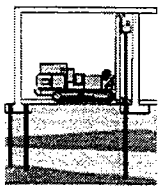
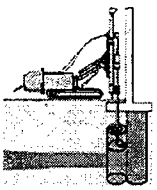
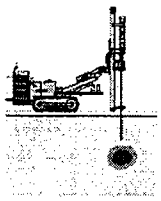
Assessing and Repairing Deep Foundations for Transportation Projects

by Glen Gorski, P.E., C.P.G. and Dennis Boehm

The repair of deep foundations has always been a challenge for both structural and geotechnical engineers. Until recently, repair methods consisted of some destructive means such as removal and replacement of the affected deep foundation. Other factors such as scheduling or maintenance of traffic flow may also drive the repair method of the affected deep foundation, making it costly to stop or halt the ongoing construction to allow removal and/or replacement of the affected foundation element. Although not as typical as some of the other replacement issues would be the removal of some of the in-place construction in a nearly completed structure to allow the removal of the affected foundation element. Several remedial techniques are available for the repair of deep foundation systems. These include minipiles, compaction grouting and jet grouting.







Engineers have turned to these alternative techniques to repair and/or replace the deficient deep foundation. The load that the foundation element carries, and whether the deficiency is related to the ground or the structural element, will determine the repair method.

Five case histories, from Georgia, Michigan, Massachusetts (2), and Nevada, are cited where these methods have been used to repair deep foundations associated with transportation related projects. A discussion on cross-hole sonic logging to assess both the deficiencies as well as the quality control of the repair is also included.

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Mechanically Stabilized Earth (MSE) Wall Repair Using Soil Nails: A Case History

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Key Terms: ***Mechanically Stabilized Earth (MSE) Wall, Soil Nails, Proof Testing.***

ABSTRACT

Mechanically Stabilized Earth (MSE) retaining wall systems were widely used in the construction of the East Street Valley Expressway (I-279 North) connecting the City of Pittsburgh, Pennsylvania to Interstate I-79 and northern suburbs. One of these walls exhibited movement shortly after construction and was regularly monitored thereafter by the Pennsylvania Department of Transportation (PennDOT) Engineering District 11-0. Gannett Fleming, Inc. was engaged by PennDOT to continue the monitoring effort, investigate the site conditions, identify the mode of failure, and recommend a remedial treatment. Through subsurface investigation and instrument monitoring, the investigators determined that the wall movement was attributed to an internal creep of the reinforcement strips through the reinforced soil.

Remedial alternative analyses concluded that passive soil nails would offer the most appropriate technical and cost effective solution. This system would act to provide additional internal reinforcement to the soil mass in a manner similar to the reinforcement strips. In essence, the design would effectively double the pullout resistance at each MSE wall panel.

The intent of this paper is to discuss the case history involving the evaluation of a failing MSE wall and the design and construction of soil nails for its remediation. Specifically, the topics discussed herein include: monitoring and geotechnical investigation; identification of the failure mode; remedial alternative evaluation; soil nail design; soil nail installation and soil nail proof testing.

INTRODUCTION

In an effort to increase accessibility between the City of Pittsburgh, Pennsylvania and northern suburbs, the Pennsylvania Department of Transportation (PennDOT) embarked on the design and construction of Interstate I-279, locally known as the East Street Valley Expressway. The three mile long expressway was constructed in Allegheny County through cut slopes in colluvial soil and weak bedrock units requiring extensive use of retaining walls to maintain the roadway geometry. Physiographically, bedrock consists of the Birmingham shale unit within the Casselman formation of the Conemaugh group. Regionally, the site is within the Kanawha section of the Appalachian Plateaus Province (see Figure 1).

The designers of the highway, constructed in the mid to late 1980's, made use of the emerging retaining wall concept of Mechanically

Stabilized Earth (MSE). The MSE technology, patented in France in 1966, was widely used in Europe prior to the first application in the United States for the California Department of Transportation. The concept was not adopted by PennDOT until the 1980's.

The Reinforced Earth® MSE technology consists of the placement of 2 inch wide by 0.2 inch thick galvanized reinforcement strips spaced approximately 2.5 feet in the horizontal and vertical directions. The strips are attached to concrete facing panels measuring approximately 5 foot wide by 5 foot high and 5½ inches thick. The reinforcement strips are backfilled with high quality coarse aggregate to form a gravity type structure relying on the pullout resistance of the reinforcement strips as the result of normal load and friction. See Figure 3 for a generic MSE wall cross section.

One of the MSE walls which runs parallel to the outbound lanes of the expressway was constructed adjacent to a pier for the Jacks Run Road bridge overpass. The wall is separated into two sections, one on either side of the pier, and are designated as Wall A and Wall B (see Figure 2). Wall A measures approximately 235 feet in length and approximately 20 feet in height at the bridge pier. Wall B is approximately 369 feet in

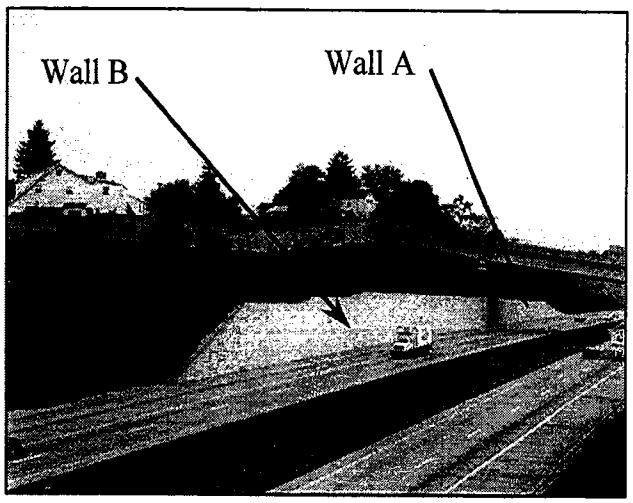

FIGURE 2
VIEW OF MSE WALLS AND JACKS RUN ROAD BRIDGE

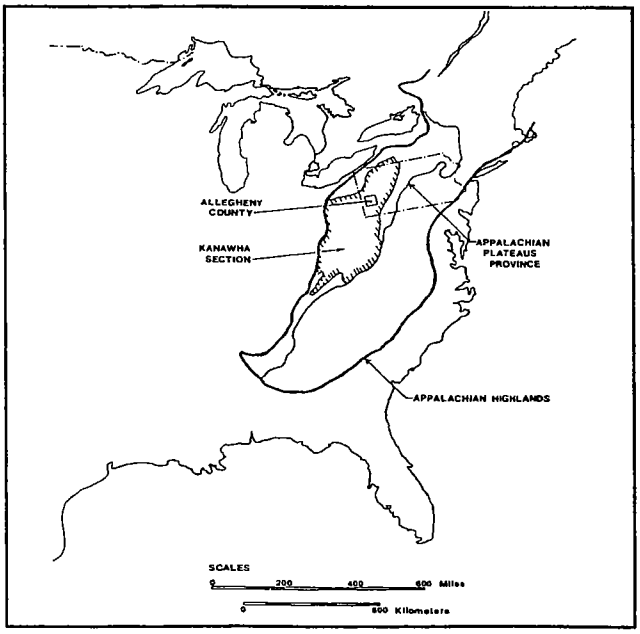

FIGURE 1
REGIONAL PHYSIOGRAPHIC SETTING

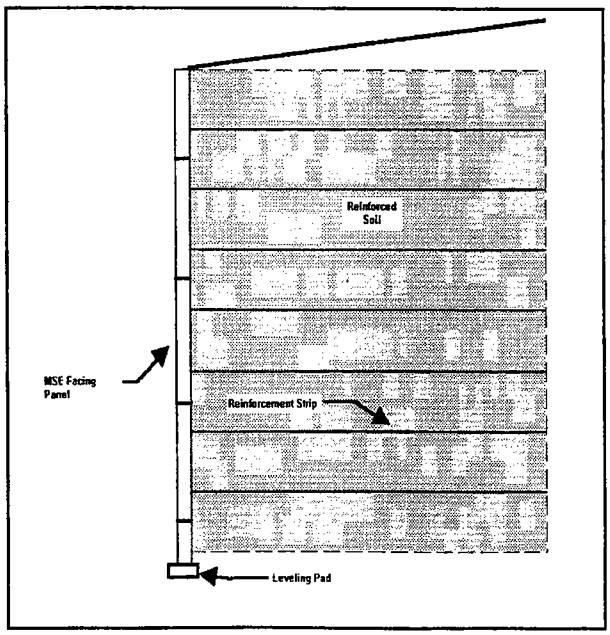

FIGURE 3
GENERIC MSE WALL CROSS SECTION

length and approximately 22 feet in height at the bridge pier. Shortly after its construction in 1987, a displacement at the pier to wall interface at Wall B was noticed. A monitoring effort was initiated at that time by PennDOT and continued for several years. Once the displacement reached a magnitude of concern, PennDOT officials decided to investigate remedial alternatives to stabilize the wall.

MONITORING AND GEOTECHNICAL INVESTIGATION

Several investigations were conducted at the site subsequent to observation of MSE Wall B movements in July, 1988. These investigations included;

- ❑ survey monitoring of the wall face,
- ❑ installation and monitoring of tiltmeters,
- ❑ installation and monitoring of slope inclinometers, and
- ❑ subsurface investigation and laboratory testing by means of vertical and horizontal borings.

Survey Monitoring

PennDOT affixed sixteen amber reflectors to Wall B and the bridge pier to serve as reference points of measurement for traditional survey monitoring. A transit was set up at a fixed reference point along the right lane concrete shoulder of the southbound lanes, and horizontal distances were measured from that fixed point to the amber reflectors. Surveys were conducted by PennDOT on 12 occasions between April 27, 1989 and March 22, 1995. Although the data obtained show some scatter, the general trend indicates the majority of the movement occurred prior to November 1992, more specifically between July 2, 1991 and November 5, 1992. Subsequent movements of the wall were considered to be within the accuracy limits of the measuring method and, therefore, could not be quantified. The survey data indicated a maximum wall displacement of approximately 3½ inches from the time of construction completion through March 22, 1995 (see Figure 4.)

FIGURE 4
DISPLACEMENT AT WALL B - PIER INTERFACE

Tiltmeters

PennDOT secured nine brass tilt plates to Wall B using epoxy cement, and a tiltmeter readout device was used to obtain rotation data at the wall. The tiltmeter is a portable device that is temporarily attached to a tilt plate and uses a force-balanced servo-accelerometer to measure inclination. Tiltmeter readings were taken by PennDOT on 13 occasions between July 16, 1990 and December 17, 1991. Initially the readings indicated a trend in wall movement, however, later readings were inconclusive and monitoring was discontinued by PennDOT. It has been determined that given the flexible nature of the individual MSE panels, readings show that the wall is rotating outward at some locations, but inward at others. This is indicative of the "bulge" in the wall (see Figure 4), but cannot be used to determine the magnitude of movement. This method was determined to be

more appropriate for measuring movements of a rigid type wall.

Slope Inclinometers

A total of four slope inclinometers were installed by PennDOT behind the MSE walls in November and December of 1991. The initial data set from the inclinometers was obtained on April 8, 1992. Two additional readings were obtained on June 29, 1992 and September 1, 1992. The results of the data collected through September 1992 indicated no movement. Gannett Fleming conducted three additional sets of inclinometer readings and compared these to the previous data set. The inclinometer readings were taken on June 16, July 12 and September 6, 1995. None of these data indicate movement since the initial set of inclinometer data obtained on April 8, 1992. It was concluded that the wall displacements could not be attributed to a global slope failure.

Subsurface Investigation and Laboratory Testing

Horizontal Borings. Two horizontal borings were cored through the second and third MSE Wall panels from the top of Wall B adjacent to the bridge pier. For each boring a 6-inch diameter hole was cored through the center of the concrete panel, and the reinforced soil was drilled using a 4-inch diameter solid stem auger. Cuttings from the auger were collected in a plastic lined sample bag for laboratory testing. Samples were taken in 2½ foot increments for the first 5 feet of augering and 5 foot intervals thereafter, to a maximum horizontal distance of 23 feet. A total of 12 samples were taken; 6 from each boring. Upon completion of augering, drainage pipes were installed. The pipes measured 4 inches in diameter, 2 feet long, and the inlet end of the pipes were covered with a geotextile fabric to prevent piping of the reinforced soil. Neither evidence of water flowing from the drainage pipes nor staining on the face of the wall have been observed. This indicates that free standing water does not collect immediately behind the wall face at the drain pipe locations.

Vertical Borings. A subsurface investigation was conducted by Gannett Fleming in May, 1995. Eight borings were advanced in the soil behind the MSE walls.

Six piezometers were installed and read daily during the field exploration program, with monthly readings thereafter. Groundwater was measured near the top of bedrock in three of four adjacent borings behind the bulging portion of Wall B. It should be noted that the piezometers typically took upwards of two weeks to stabilize, indicating a permeability lower than would be expected in a quality coarse grained soil required for MSE construction.

As a result of the subsurface investigation, the site was characterized to consist of three layers: reddish brown clayey silt with varying amounts of rock fragments (random backfill material), underlain by gray to brown clayey gravely sand to sandy gravel (the reinforced soil mass), and gray, soft, thinly bedded silty claystone. The uppermost layer, the reddish brown clayey silt, represents the sloping backfill material above the reinforced soil mass and the gray silty claystone represents the bearing material for the MSE wall. For the purpose of these discussions the focus of the subsurface conditions will concentrate on the reinforced soil mass only.

The reinforced soil mass consists of gray to brown clayey sandy gravel/gravelly sand with varying amounts coal fragments (AASHTO Classification A-1-a to A-6; USCS Classification GC to SM). The gravel sized portions of the reinforced soil consists of sandstone, siltstone, shale, and claystone fragments. The soil index

tests and strength tests indicate a relatively uniform soil condition. The natural moisture contents ranged between 6.0% and 19.9%. The in-situ unit weights ranged from 118.1 pcf to 135.8 pcf and the angle of internal friction ranged from 23.0 to 46.0 degrees, with an average of 32.9 degrees. The percent fines ranged between 8.0% and 46.3%, with an average of 19.4%. Project specifications requires a minimum angle of internal friction greater than 34°, and a fine content (material passing the #200 sieve) less than 15%.

Although the reinforced soil conditions throughout the length of the wall were found to be similar, some differences were noted. These differences included the visual description and natural moisture contents of the soils. The visual descriptions indicated that the gravel sized portions of the soil behind the bulging portion of Wall B consisted primarily of slaking siltstones, shales and claystone, while sandstone was the major constituent of the samples obtained at other locations of the wall. In an effort to quantify the quality of the coarse grained fraction of the reinforced soil material, durability testing was conducted on portions of the sample retained on the 1 inch and 2 inch sieves. Testing of 13 samples indicated that 9 had a wet/dry durability of 0%, and 4 samples had wet/dry durabilities of 11.6%, 23.1%, 38.7%, and 11.8%. Only one of the thirteen tests indicated a material exceeding the specification requirement of 30.0%.

The average natural moisture content was 9.5% at the bulge location, 1.4% higher as compared to 8.1% for other areas. In addition, the soil classifications in the bulging area ranged from A-1 type soils to A-6 type, but were generally A-2 through A-6. The classification tests for soil samples obtained from other portions of the wall were generally A-1 or A-2 type materials. The soil moisture retention was determined to be associated with the high fine content.

The laboratory corrosivity testing indicated that the reinforced soil properties are potentially conducive to a corrosive environment, as compared to the contract specifications. Resistivity results indicated that all samples were outside the specification requirements of >3000 Ω·cm, with an average resistivity of 1200 Ω·cm for Wall B. Sulphate content results indicated that only 7 of 24 samples were within the specification requirements of <1000 ppm, with an average 1100 Ω·cm for Wall B. Chloride content results were within the specification requirements of <200 ppm, and pH results were within the specification range of 5 to 10.

Summary

In summary, the monitoring and geotechnical investigation yielded several conclusions regarding the current state of the MSE wall.

- The majority of the wall movement occurred between July, 1991 and November 1992.
- The movement appears as a "bulge" near the mid-height of the wall at the interface with the bridge pier and measures approximately 3½ inches.
- Slope inclinometers did not reveal movement, therefore the wall displacements are not attributed to a global type slope failure.
- Laboratory testing indicates that the reinforced soil does not meet specifications for: shear strength; fine content; durability; and, corrosivity.

IDENTIFICATION OF FAILURE MODE

The design of MSE retaining walls, and specifically Reinforced Earth® structures, is based on several variables consisting of:

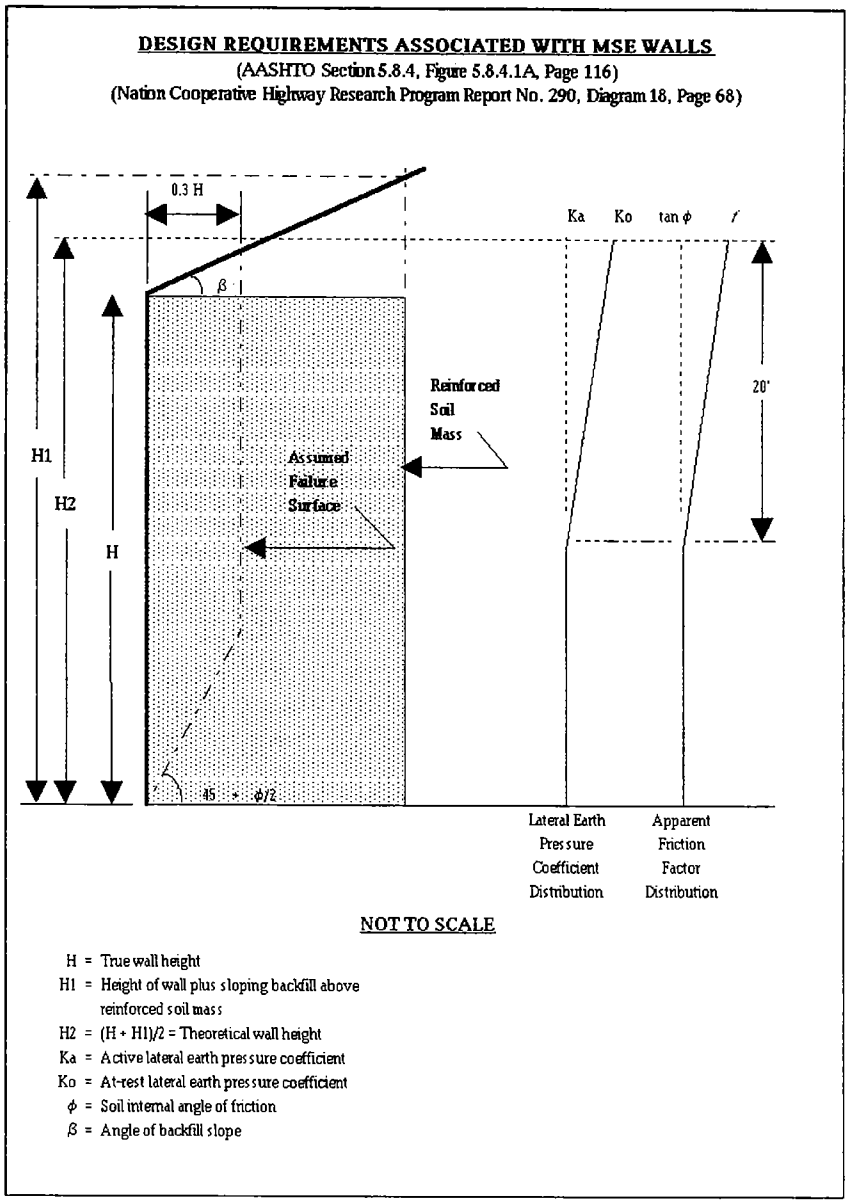

FIGURE 5
DESIGN REQUIREMENTS

- lateral earth pressure coefficients of the reinforced soil,
- the apparent friction factor between the reinforcement strip and the reinforced soil,
- wall height, and
- contact area of the reinforcement strip.

A graphical presentation of the design requirements is shown in Figure 5. Since the backfill material for this project is sloping, a theoretical height was assumed for the wall, and was defined to be the height of the wall plus the average height of the sloping backfill material above the reinforced soil mass. The average height of the sloping backfill material above the reinforced soil mass was, in essence, treated as a surcharge load. The sloping backfill increased the estimated magnitude of the lateral earth pressures exerted on the wall. A theoretical failure plane was assumed to occur in the reinforced soil mass behind the wall at a distance equal to 0.3 times the true height of the wall. The failure plane near the base of the wall was drawn at an angle of $45 + \phi/2$ from the horizontal axis to form the complete theoretical failure surface.

Lateral earth pressure estimations were determined by varying the lateral earth pressure coefficient with depth. The lateral earth pressure coefficient at the theoretical height of the wall was assumed to be equal to the at-rest earth pressure coefficient, as determined by:

$$K_o = (1 - \sin\phi)$$

The lateral earth pressure coefficient decreased linearly to a depth of 20 feet, in accordance with AASHTO, below the theoretical height where the

value was equal to the active earth pressure coefficient, K_a, as determined by the Rankine method:

$$K_a = \tan^2(45 - \phi/2)$$

The value of K_a remained constant from a depth of 20 feet, in accordance with AASHTO, to the bottom of the wall.

Values for the apparent friction factor between the reinforcement strip and the reinforced soil were determined in much the same manner. A maximum value of the apparent friction factor, an empirically derived value f equal to 1.5, was assigned to the theoretical height line and decreased linearly to a value of $f = \tan\phi$ at a depth of 20 feet, in accordance with AASHTO, below the theoretical height line and remained constant from this depth to the base of the wall.

In determining the frictional resistance provided by the reinforcement strip, the friction factor for that strip was multiplied by the overburden pressure and the surface area of the top and bottom sides of the reinforcement strip extending beyond the theoretical failure plane (described above). The length of the strip beyond the failure surface was defined as the effective strip length. Once the value of resisting force was obtained, it was divided by the lateral earth force to determine a factor of safety. A factor of safety of 1.50 was specified as the minimum for each reinforcement strip in the retaining wall.

It can be seen from the short description of the design assumptions and procedures listed above that the system is heavily dependant on the lateral earth pressure coefficients (a function of ϕ) and the empirical friction factor values (a function of backfill quality, i.e. fines and moisture). If the frictional resistance of the reinforcement strips is jeopardized, the structure may experience an internal instability. The frictional resistance of the reinforcement strips may diminish if the backfill contains fines in excess of the specification requirements and a higher moisture content. Once the frictional resistance is decreased:

- the Reinforced Earth® system provides less resistance to lateral earth pressures,
- the factor of safety against pullout is reduced, and
- the structure experiences creep.

As reported by the national Cooperative Highway Research Program Report 290:

> "The major factors governing creep behavior of the reinforced soil are the nature and quantity of the fine grained portion of the backfill material and the excess moisture content above the optimum moisture content." [1]

The construction specifications for this project required a minimum 34° internal angle of friction, a plasticity index of not less than 6, and fines less than or equal to 15% passing the #200 sieve for the reinforced soil. These specifications were developed to ensure a proper frictional resistance interface between the reinforcement strips and the reinforced soil, thus protecting against internal instability leading to creep. The initial wall design completed in 1986 assumed these soil parameters for all design calculations.

Laboratory testing performed on the reinforced soil in the area of the bulging portion of Wall B indicate:

- the presence of excess fines,

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[1]National Cooperative Highway Research Program, Report 290, Reinforcement of Earth Slopes and Embankments, James K. Mitchell and Willem C.B. Villet, Transportation Research Board, June 1987, page 101.

- a higher moisture content, and
- a reduced internal friction angle of the soil.

The consequence of these indications is non-conformance to the specification requirements and a diminished pullout resistance of the reinforcement strips. These conditions have resulted in a reduced friction factor between the reinforcement strips and the reinforced soil, and an increase of lateral earth pressure coefficients leading to lower factors of safety at all of the reinforcement strips. As the factors of safety decreased below static equilibrium, creep of the reinforcement strips developed in the reinforced soil mass and became apparent in deflections of the wall face. Given the non-linear nature of the empirically derived internal stability calculations for MSE wall systems, the minimum factor of safety occurs just above the mid-height of the wall. This theoretical assumption manifested itself in the actual displacement of the wall (see Figure 4).

In order to determine the current stability of the walls, external and internal stability calculations were performed in accordance with AASHTO (American Association of State Highway Transportation Officials) Design Manual Section 5.8, and the design procedures presented in the AASHTO sponsored National Cooperative Highway Research Program Report No. 290, Reinforcement of Earth Slopes and Embankments. Evaluations were conducted along critical sections of both Wall A and Wall B.

Wall A. The internal and external stability calculations utilizing reinforced soil parameters from laboratory tests for Wall A indicated that the bearing pressure, as well as factors of safety for overturning, sliding, and reinforcement strip pullout are above the minimum AASHTO requirements.

Wall B. The external stability calculations utilizing reinforced soil parameters from laboratory tests for the bulging portion of Wall B indicate that the factors of safety for overturning and sliding are above the minimum AASHTO requirements. However, the internal stability calculations indicate that the factor of safety for reinforcement strip pullout is near 1.00 at several elevations when the reinforced soil parameters from laboratory tests are utilized.

The minimum factor of safety for the bulging portion of Wall B, calculated by Gannett Fleming using the original design assumptions, was 1.51 and slightly above the minimum AASHTO requirement of 1.50. It can be concluded that any combination of factors which reduced both the frictional resistance between the reinforcement strips and reinforced soil, and increased the lateral earth pressures would also reduce the factor of safety. Because the original factor of safety was only slightly above the specification requirements, only a small tolerance was allowed for deviation of reinforced soil parameters.

The external stability calculations utilizing reinforced soil parameters from laboratory tests for other portions of Wall B are above the minimum AASHTO requirements. However, the internal stability calculations indicate that the minimum factor of safety for reinforcement strip pullout is 1.20 when the reinforced soil parameters from laboratory tests are utilized.

Comparison of Wall A and Wall B. The most significant difference between Walls A and B is the as-built reinforcement strip length. At the bridge pier, Wall B is two feet higher than Wall A. This minor difference did not noticeably affect the calculated lateral earth pressures. However, the designed length of reinforcement strips were not equal. Wall B strip lengths were originally designed and constructed 18 feet from the back face of the wall whereas Wall A strip lengths were 26 feet. As a result, the design factor of safety for Wall B (F.S. = 1.51) was significantly less than for Wall A (F.S. = 2.52), using original design assumptions. As a result,

Wall A is over-designed and capable of resisting the additional stresses induced by deviation of soil parameters from the specifications and the resulting changes in loading. However, Wall B was determined to be extremely sensitive to these changed conditions as the result of shorter reinforcement strip lengths. Therefore, it was concluded that remedial alternatives should be pursued for the Wall B location only.

REMEDIAL ALTERNATIVE EVALUATION

Gannett Fleming identified two general remedial alternative solutions; external rehabilitation or internal rehabilitation.

External Structural Alternatives

Three external structural remediation alternatives were evaluated, and included:

- an anchored soldier beam wall
- a cantilevered soldier beam wall, and
- an anchored wale alternative.

To return the structure to the minimum required factor of safety of 1.50, both the cantilevered soldier beam wall and anchored wale alternatives were designed to resist 50% of the loads on the wall as determined by the soil properties identified through laboratory testing, thus restoring a factor of safety of 1.50.

The anchored soldier beam wall was designed to carry 100% of the in-situ load, therefore essentially replacing the MSE wall.

Anchored Soldier Beam Wall. The design of the anchored soldier beam wall alternative included the placement of anchored soldier beams, spaced approximately 9'-10" on center, approximately 2'-10" in front of the existing MSE wall. A W21 x 101, A572 GR50, steel beam grouted in a 2'- 6" diameter drilled socket 12' - 9" deep was determined appropriate for this application. Each of the six anchors required for this alternative consisted of a 5 strand, 7-wire ASTM A416 GR270, cable anchor. The anchors were designed to be installed 7 feet from the top of the soldier pile and to extend 30 feet through the reinforced soil and backfill material and 35 feet into bedrock on a 20° declined angle from the horizontal plane. The anchor bond zone has been determined to require a 5½ inch diameter, 30 foot long, bond length. This alternative was determined to be very costly, and would require encroachment onto the expressway shoulder.

Cantilever Soldier Beam Wall. The design of the cantilever soldier beam wall alternative included the placement of cantilevered soldier beams spaced approximately 4'-11" on center (one MSE panel width). A cast-in-place reinforced concrete facing was proposed to bridge between the piles, and the annular space between the new and existing wall was proposed be filled with No. 57 coarse aggregate. A W21 x 147, A572 GR50, steel soldier beam grouted in a 2'- 6" diameter drilled socket 24' - 0" deep was determined appropriate for this application. A cast-in-place reinforced concrete support was proposed to be placed between the soldier beam and the existing wall to provide transfer of load from the existing wall to the new wall. This alternative was also determined to be costly and would encroach onto the shoulder.

Anchored Wale. The layout of the anchored wale system provided resistance for every exposed panel. The design of the anchored wale system consisted of three 54 foot long horizontal wales, spaced 5 feet vertically. Six anchors, on approximate 9'-10" horizontal centers, were required for each wale. A wale consists of two C10 x 20 steel channels placed back to back, with a spacer, forming an I-beam section through which the anchors pass. The design of the wale sections was determined by

obtaining the lateral earth pressure exerted on the lowest wale layer and sizing each wale section for that maximum load.

The anchors were designed uniquely for each wale. Due to the different lateral earth pressures exerted at each wale level, it was unnecessary to design each anchor to the highest load. The top wales each required six, 25 foot long, 2 strand, 7-wire ASTM A416 GR270, cable anchor tendons grouted in a 6" diameter horizontal hole. The bottom wale required six, 45 foot long, cable anchors, extending approximately 10 feet into rock. In addition to a relatively high construction cost, aesthetics were a decisive factor in elimination of this alternative.

Internal Structural Alternative - Soil Nails

The soil nailing alternative provided additional reinforcement to each individual MSE wall panel. This approach is based on the same theory used during the original design of the MSE system; namely, frictional resistance. The addition of one soil nail of proper length and diameter to each MSE panel would provide the necessary resistance to return the MSE system to a minimum required internal factor of safety. This is the only alternative to provide a truly internal structural remediation for the creep type failure of the existing MSE Wall B.

Selection of Alternative

In order to increase the internal wall stability factors of safety to the required PennDOT and AASHTO standards, a soil nail remedial design for Wall B was recommended. Soil nails are considered to be the most effective remedial alternative for the following reasons:

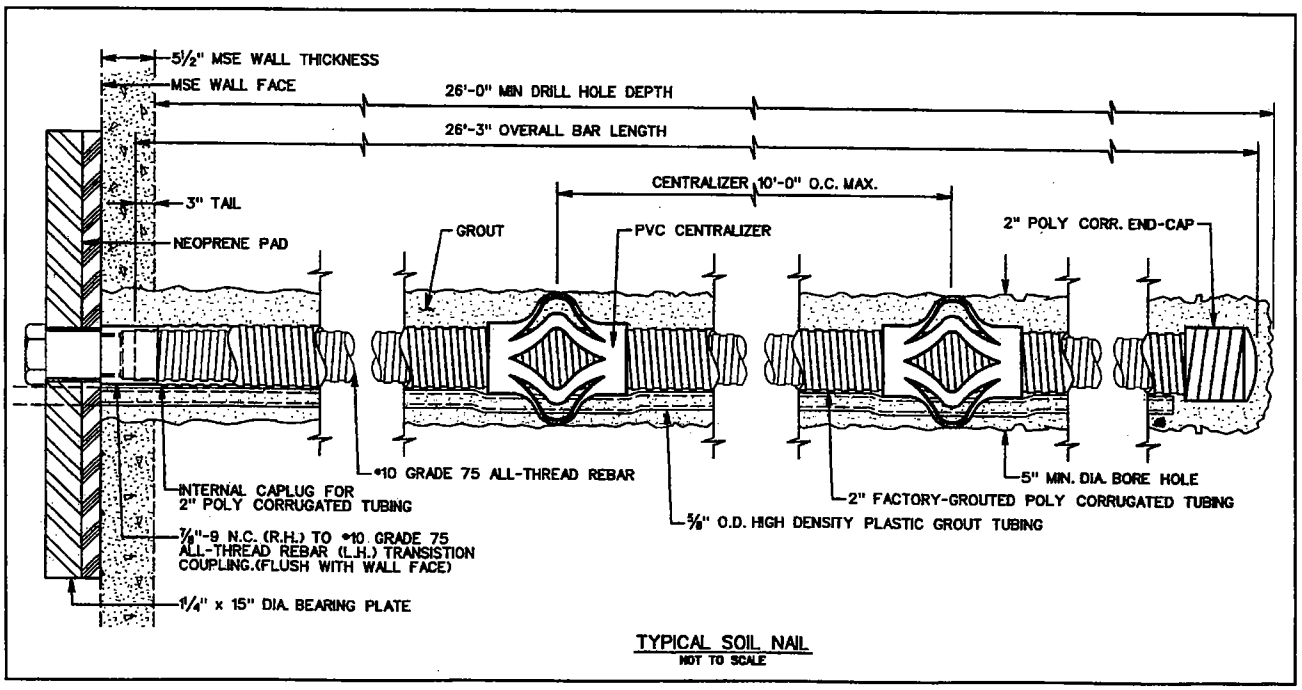

FIGURE 6
SOIL NAIL CONFIGURATION

- The MSE Wall B failure has occurred due to internal instability, i.e., a creep-type pullout of the reinforcement strips. The soil nail concept provides additional internal reinforcement which will supplement the existing reinforcement, and increase the internal factor of safety to an acceptable level.
- The soil nail alternative does not require encroachment into the roadway shoulder.
- Unit costs for soil nailing are the lowest of the considered alternatives, and construction time is minimized.

SOIL NAIL DESIGN

The soil nails were designed in accordance with the Federal Highway Administration Publication No. FHWA-SA-96-069 manual for Design and Construction Monitoring of Soil Nail Walls. Soil nails are passive reinforcement elements that mobilize shear strength resistance within the bond zone through movement. Therefore, the soil nail elements are not post tensioned after installation and concerns regarding the cracking of the thin MSE facing panels were mitigated. The design approach anticipates that the soil nails will act passively at each MSE facing panel until the MSE wall continues to creep and the shear strength along the soil nail elements is mobilized.

For purposes of design, the maximum lateral earth pressure was determined at the base of the wall and was used as the loading criteria for all of the soil nail elements. This loading translated into a 37.3 kip load per soil nail. A typical soil nail design was applied to all of the MSE facing panels to ease material procurement and installation. The typical soil nail consists of a 26 foot long, #10 Grade 75, epoxy coated reinforcement bar, encapsulated in corrugated PVC and grouted in a 5 inch diameter horizontal bore hole (see Figure 6). The PVC encapsulation was specified to resist the corrosive environment identified by the laboratory testing program. Figure 7 illustrates an assembled soil nail; however, this particular picture reveals a seven foot long PVC casing that was used to develop the un-bonded zone for the proof test nails as discussed in the Soil Nail Testing section. The soil to grout interface allowable shear strength was determined to be 1,500 psf based on the laboratory testing and published data.

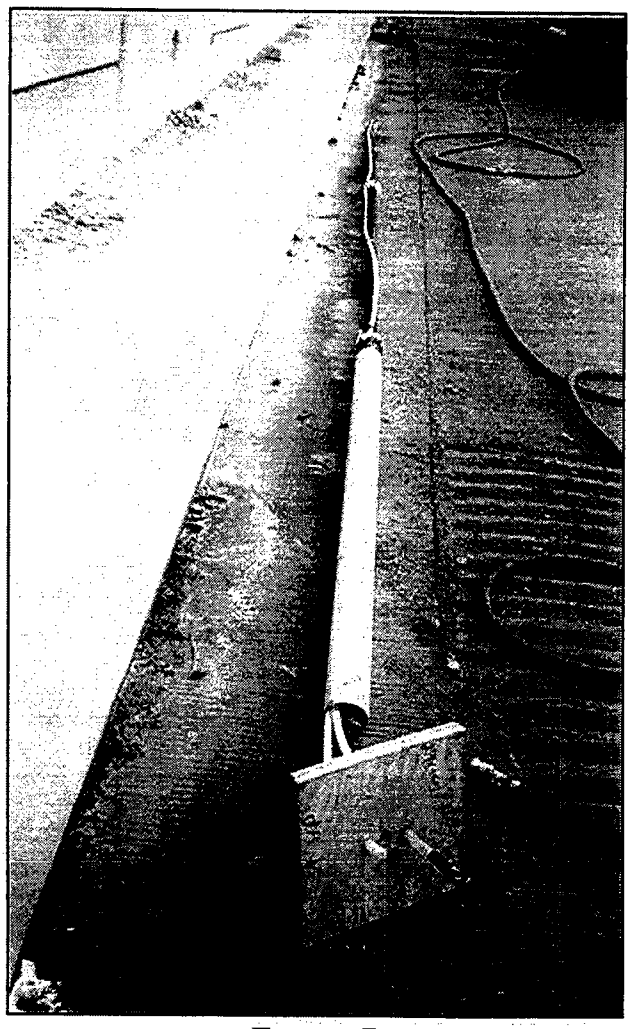

FIGURE 7
ASSEMBLED SOIL NAIL
(NOTE WHITE PVC CASING FOR TEST NAIL)

FIGURE 8
GROUTING OF A SOIL NAIL

Each soil nail element was designed to resist 100% of the lateral earth pressure, or in essence, replace all of the four reinforcement strips per MSE panel. The minimum factor of safety was determined to be 2.0 at the wall location near static equilibrium when considering pullout resistance from both the existing reinforcement strips and the soil nail.

The design required a horizontal installation of the soil nails which was determined necessary to prevent the imposition of the vertical component of the load induced by a declined installation. Horizontal installation of soil nails is not a typical application; however, given the data obtained from the subsurface investigation, the investigators were confident that the backfill material would not cave in during the drilling of the bore holes given the high fine content and cohesive nature of the backfill. Temporary steel casing was not specified for the installation of the soil nails. The investigators do not recommend un-cased drilling for soil nails unless the material is well characterized.

SOIL NAIL INSTALLATION

Installation of the soil nails at Wall B was initiated in March, 1998. Given that this project was located on a major arterial roadway to the City of Pittsburgh, and a temporary lane closure would be required, the construction schedule was aggressive. The sequence of work consisted of concrete coring all of the MSE panels prior to soil nail installation. Once this operation was completed, one soil nail, the first test nail, was installed. The contractor was required to wait 3 days prior to testing of the soil nail to allow adequate time for grout cure and a minimum 1,500 psi unconfined compressive strength. Once the test nail was accepted, production installation was approved. The specifications for the soil nail installation were written as "performance specifications" in which the contractor was responsible for the installation techniques and grout mix design. Submittals for the construction sequence, techniques, and grout design were subject to the engineer's approval.

A total of 255 soil nails were installed using a Cassegrande C7 drill rig and a special platform was fabricated by the contractor to reach the upper soil nail locations. At maximum productivity, 7 borings were drilled and 7 soil nail elements were installed per day. Drill holes were not permitted to remain open overnight.

The soil nails were grouted by attaching a temporary wooden bearing plate (see Figures 7 and 8). The grout tube and de-air tube extended through the temporary bearing plate as shown. A grout pump was attached to the grout tube using a special fitting and the grout traveled the length of the nail to the back of the bore hole. The grout was placed from the rear of the soil nail to the surface to ensure a completely grouted hole with the air being forced to the front of the wall through the de-air tube. The hole was completely grouted when grout discontinued spitting out of the de-air tube and flowed continuously (see Figure 8). The de-air tube was then sealed and the soil nail was allowed to cure

for several days. The de-air and grout tubes were trimmed, the temporary bearing plate was removed and the neoprene pad and the 15 inch diameter permanent steel bearing plate was installed using a high strength bolt. Upon completion, all bolts were tack welded to the bearing plates, and the bearing plates were painted to match the concrete tone of the MSE facing panels.

Soil nail installation, both test and production elements, utilizing tremie tube grouting techniques in an open horizontal bore hole proved to be relatively simple. The presence of the moist, fine grained material minimized bore hole collapse and simplified construction. All except for one of the 255 soil nail drill holes remained open. The single collapsed hole required re-drilling and minor additional grout take. No casing was required during production nail installation.

SOIL NAIL TESTING

In order to properly test the soil nail design assumptions, seven test nail locations were randomly selected within the installation pattern.

The contractor installed the initial test nail on March 24, 1998 and developed the required 7 foot un-bonded length by attaching a 7-foot long, 4-inch nominal diameter PVC casing over the soil nail behind the wall face (see Figure 7). However, since the nominal drill hole diameter was approximately 5½-inches in diameter, an annular space of ¾ of an inch was created around the outside of the PVC casing. Upon filling the bonded zone with grout, it was apparent that the 4-inch diameter casing was not sufficient to develop the bonded to un-bonded zone, and grout migrated around the PVC casing. The loss of grout around this casing resulted in a diminished volume of grout in the bonded zone and loss of bond surface area. The initial test nail exhibited soil mobilization movement that was greater than the predicted ¼ inch (See Table 1). The contractor was directed to provide a better seal at the bonded to un-bonded zone interface. On subsequent test nail installations, the 4-inch diameter PVC casing was used; however, a 6-inch long by 5½-inch nominal diameter casing was installed at the bonded to un-bonded zone interface and centered using wooden shims. This method was sufficient to retain the grout in the bonded zone and the soil mobilization movements were reduced dramatically (63% on average).

Testing

Designated test nails were loaded using a compressed air driven hydraulic pump and a hydraulic ram. The applied load was measured using a load cell (primary) and two pressure gauges (secondary.) The typical soil nail testing jack set-up is shown in Figures 10 and 11. The

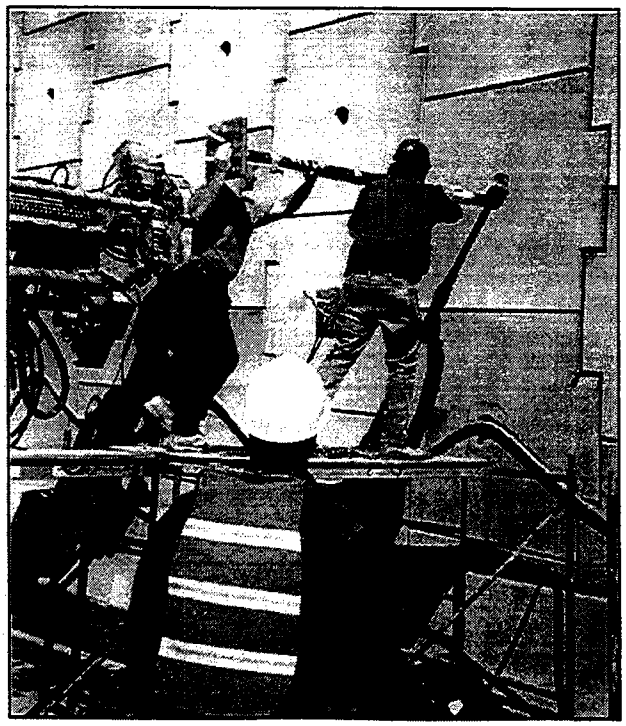

FIGURE 9
INSTALLING A SOIL NAIL

testing procedure involved loading the test nail to an alignment load, approximately 5% of design load, and incrementally stressing the nail to 100% design load in intervals of 15%. The test load was not taken above 100% of the 37.3 kip design load in order to minimize the stress on the relatively thin MSE facing panels. Additionally, design assumptions included that the MSE wall is currently at limit equilibrium and the soil nails are a supplemental system capable of achieving 100% of the lateral earth pressures; therefore, unlike new wall construction, test loading above the design assumption was not required. Upon achieving 100% design load, the nail was tested for creep movement with the initial criteria being 0.04 inches of movement between 1 and 10 minutes, and the subsequent criteria being 0.08 inches between 6 and 60 minutes. The 6 to 60 minute test was required only if the nail failed, or was marginal, within the initial creep period. These criteria are based on FHWA guidelines

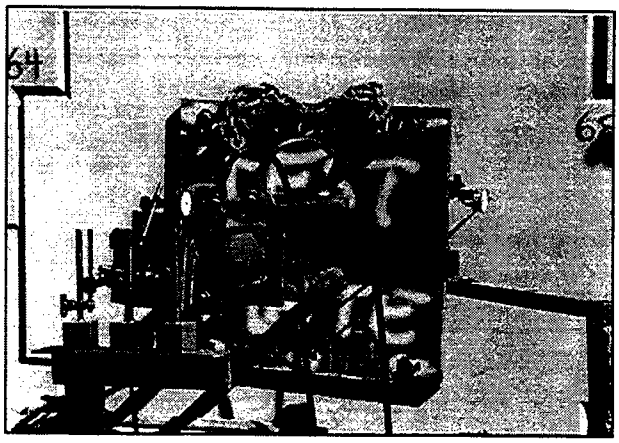

FIGURE 10
SOIL NAIL PROOF TESTING

(Publication FHWA-SA-96-069). All test nails passed the initial creep test criteria. Six of the seven test nails were extended to 60 minutes for data collection purposes only. The one nail that was not extended to 60 minutes was terminated due to adverse weather conditions.

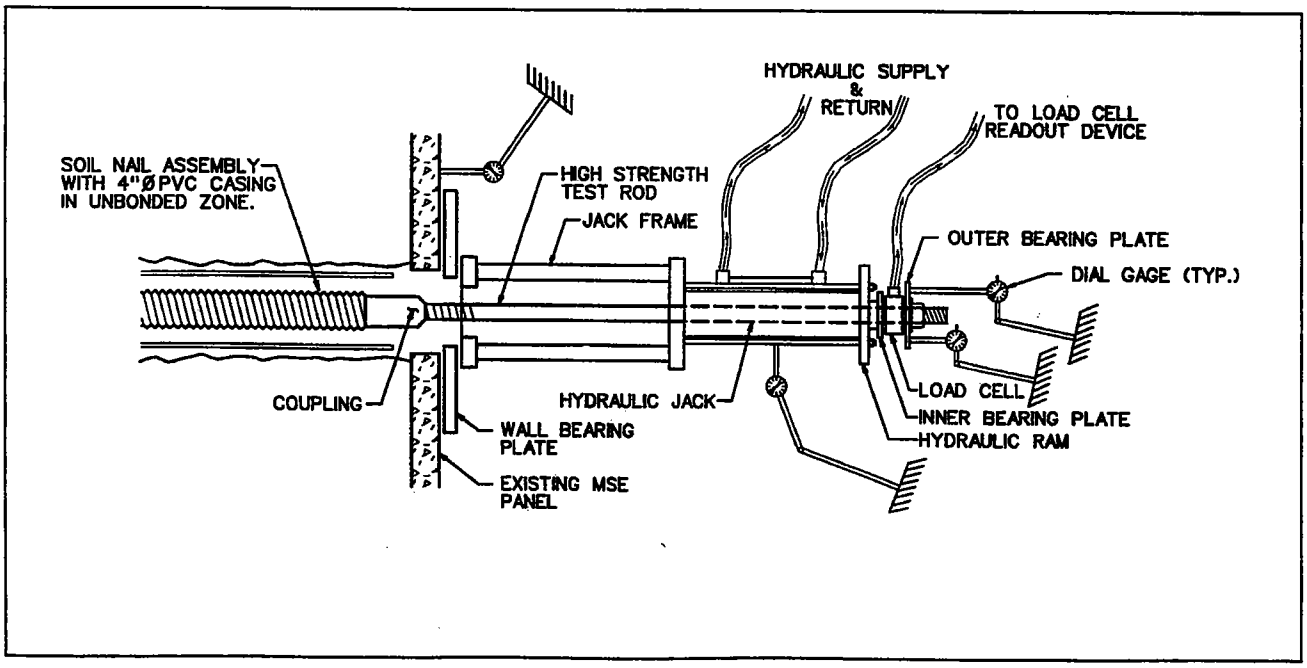

FIGURE 11
SOIL NAIL TESTING APPARATUS

Test Nail	Date	Creep Movement 1 - 10 min. (inches) Criteria = 0.04	Creep Movement 6 - 60 min. (inches) Criteria = 0.08	Total Movement at 100% Design Load (inches)	Soil Mobilization (Unloaded Reading Minus Total) (inches)	Bar Elongation (Total Minus Soil Mobilization) (inches)
1	3/27/98	0.037	0.051	0.583	0.482	0.101
2	4/6/98	0.005	0.006	0.283	0.029	0.255
3	4/13/98	0.020	0.018	0.414	0.179	0.235
4	4/2098	0.023	0.016	0.275	0.073	0.203
5	5/5/98	0.030	0.037	0.372	0.144	0.228
6	5/11/98	0.011	N/A	0.394	0.149	0.245
7	5/15/98	0.012	0.030	0.340	0.129	0.211
Avg. w/o #1		0.017	0.021	0.346	0.117	0.230

TABLE 1
RESULTS OF SOIL NAIL PROOF TESTS

Interpretation of Test Data

All seven test nails passed the FHWA guidelines for proof testing. Specifically, these nails exhibited sufficient strength to achieve the desired test load over a sustained duration of time upon achieving shear strength through soil mobilization within the bonded zone. Additionally, the creep data plotted linearly on a log-time scale, as required by FHWA, with a statistical fit analysis (R^2) ranging between 0.8835 and 0.9959. The data that did not reveal an R^2 near 1.0000 was due to fluctuations in the load associated with re-pumping of the hydraulic ram through the creep period due to hydraulic losses, temperature fluctuations, etc.

CONCLUSIONS

Through the monitoring, geotechnical investigation, analysis of failure mechanism, soil nail design, installation, and testing, several conclusions have been reached:

- MSE walls are heavily reliant on the physical properties of the material used for the reinforced soil. Deviations from these assumed parameters in design can have grave consequences after construction.
- Proper inspection during construction is essential to ensure the quality of materials being installed and conformance to specifications.
- Soil nails are an efficient, internal, remedial alternative solution for MSE walls exhibiting displacement, if:
  - The failure mechanism is not a global

failure extending behind the reinforced soil,
- The material is of a fine grained nature such that horizontal installation is simplified, and grout takes are not excessive.

- ❑ Soil nails are an attractive remedial alternative for MSE walls constructed adjacent to highways in terms of cost and construction schedule, as well as not requiring additional shoulder space for the finished installation.
- ❑ A proper seal is required to assure the un-bonded length of the proof test nail to mitigate loss of grout and reducing the strength of the test nail.

This project was the first of its kind in PennDOT District 11-0. A similar approach was used for a failing MSE wall in an adjacent engineering district, however, the failure mode was not the same. Evaluations for that wall concluded that the MSE panels were failing due to inadequate connection of the reinforcement strip to the MSE panels. High quality backfill, meeting the specifications discussed above, was used and the soil nail installation required an excessive grout take as it migrated though the coarse grained backfill. Therefore, soil nails for repair of failing MSE walls cannot be considered a universal approach, and extensive site investigations should be conducted to determine the mode of failure and appropriateness of the remedial repair.

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Rock Foundations for Route 33 Bridge Over the Lehigh River

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ABSTRACT

The Pennsylvania Department of Transportation is proceeding with a project that involves the southward extension of State Route 33 and the construction of a new interchange between Route 33 and Interstate 78 near Bethlehem, Pennsylvania. Immediately to the north of the interchange, a new 1,870' long steel truss bridge will be built to carry the four lane highway over the Lehigh River Valley.

Extensive investigations, involving a team of engineers and geologists, were undertaken in order to characterize the bedrock stratigraphy and geologic structure and to assess the rock mass strength properties across the valley. The bedrock at the proposed bridge site is typically shallow, but the geology is very complex. The Cambrian-aged Leithsville and Allentown Formations that underlie the area consist of rocks that grade from relatively pure limestone and dolomite to argillaceous limestone and calcareous shale. Highly variable bedrock conditions at the bridge site have resulted from intense structural deformation, including overturned bedding, and severe solution weathering. Therefore, the field and laboratory test programs were planned to clearly define the geologic conditions and to obtain the necessary data to predict the rock mass behavior at each of the bridge substructure foundation locations. Specific techniques used in the investigations included detailed geologic mapping, large scale test pits, and state-of-the-art angled drill hole core reorienting.

Large foundation design loads resulted for the bridge piers since they vary up to 135' in height and support bridge spans of up to 594' in length. Consideration of the difficult geologic conditions, the large structure loads, and the stability of bridge piers perched on rock slopes with adverse joint orientations, dictated that several different foundation types were required. The appropriate foundation for each bridge support was selected from the standpoint of reliability, constructability and economy. This project is illustrative of the intricate process that is essential in order to design and construct a major highway structure in a region with complex subsurface conditions. Construction of this bridge has just recently begun and the results of the detailed static load testing and non-destructive test programs of the various foundation components will provide data to verify the design assumptions and ensure the quality of the construction.

Artist Rendering of Lehigh River Bridge

1.0 INTRODUCTION

State Route 33 (S.R. 0033) is a limited access north-south highway in northeastern Pennsylvania. The Pennsylvania Department of Transportation (PENNDOT) Engineering District 5-0 has initiated a project to extend S.R. 0033 from its present southern terminus at S.R. 0022 in order to connect directly with Interstate 78 (I-78), a major east-west highway. The project area is located east of Bethlehem in Northampton County, as shown by Figure 1.

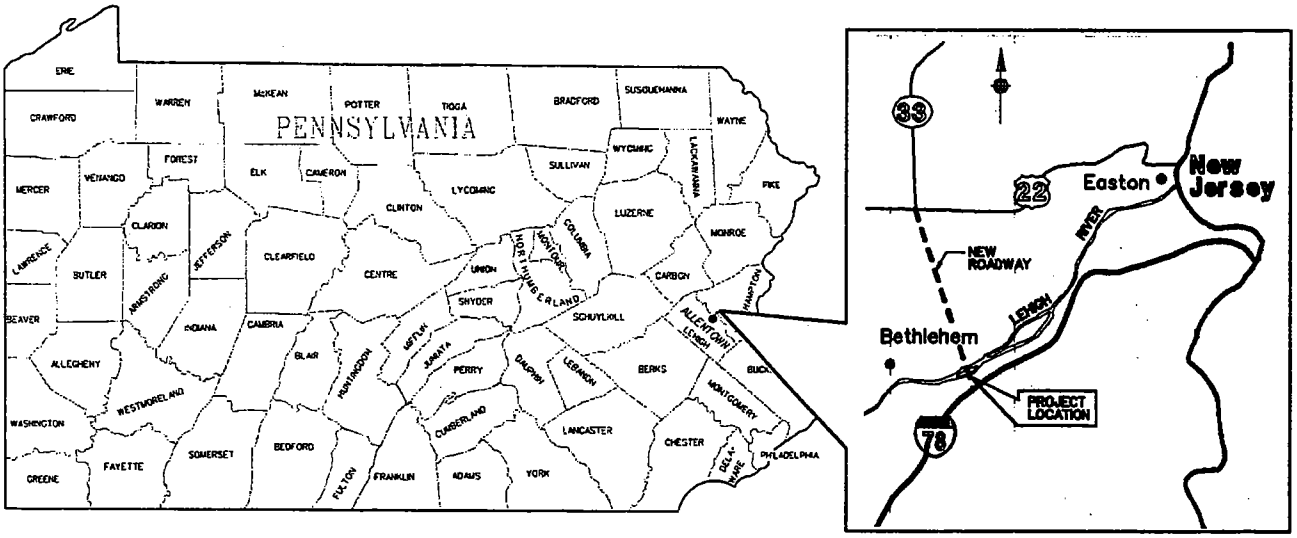
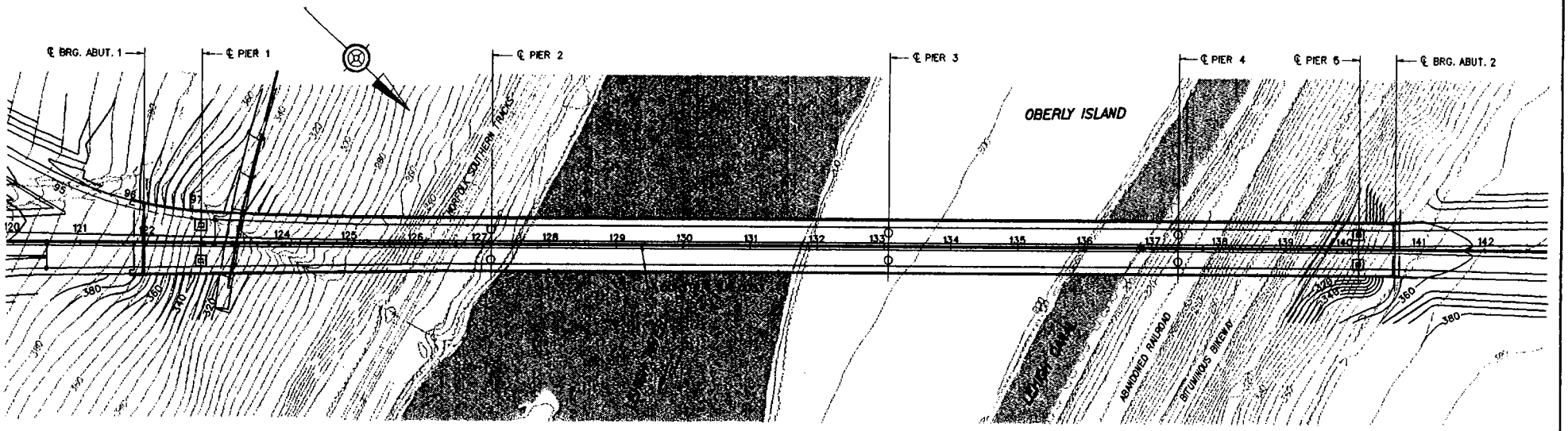


Figure 1 - Project Location

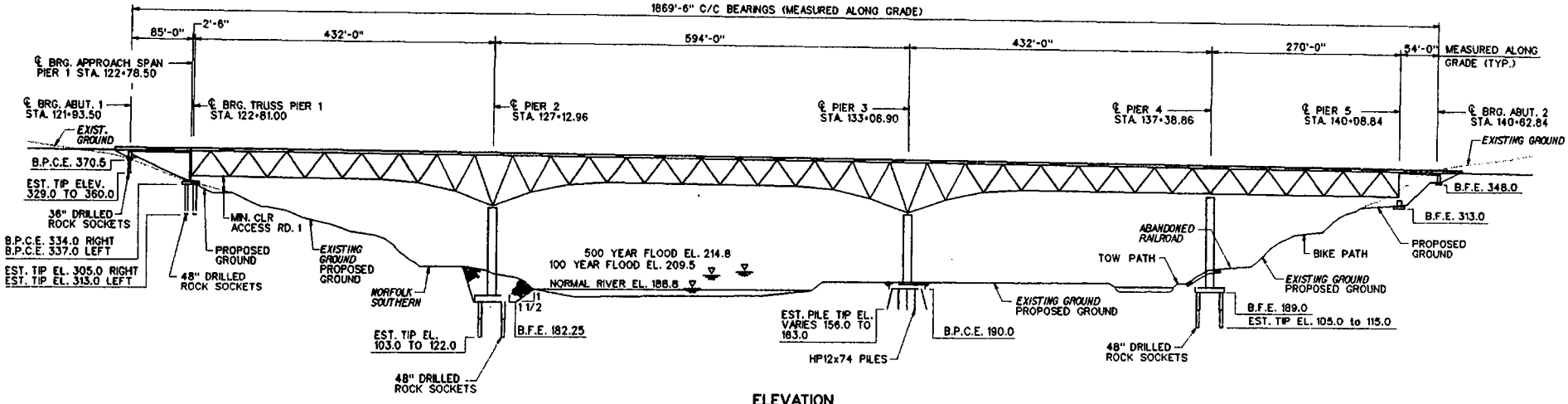
A 1,870' long bridge is required to carry the new section of S.R. 0033 across the Lehigh River Valley. Construction of the bridge was initiated in April, 2000 and is scheduled to be completed in August, 2002. URS Corporation provided engineering services for the design of this project. This paper presents a summary of the geologic studies, foundation designs, and construction observations (to date) for the foundations of the Lehigh River Bridge.

Figures 2 and 3 present a plan view and elevation of the proposed bridge, respectively. These figures illustrate the substructure locations and the other prominent features located in the river valley. The Lehigh River flows roughly west to east (and nearly parallel to I-78) through the project area, and the proposed bridge alignment crosses the valley with a skew of about 35 degrees to the northwest. In addition to crossing over the Lehigh River, the new bridge will cross over active Norfolk Southern railroad tracks, the historic Lehigh Canal and its tow path, an abandoned railroad bed, and a pedestrian and bicycle path that is owned by the local townships. An island (Oberly Island) separates the river from the canal in this area. The bridge is to be constructed as a four span steel deck truss and it will have short multi-girder approach spans at each end. The lengths of the four spans of the steel truss are to be 432', 594', 432' and 270', from south to north. The southern and northern approach spans are to have simple steel multi-girder superstructures of 85' and 54' length, respectively.



PLAN
80 40 0 80

FIGURE 2 - LEHIGH RIVER BRIDGE PLAN VIEW



ELEVATION
80 40 0 80

FIGURE 3 - LEHIGH RIVER BRIDGE ELEVATION VIEW

As shown on Figures 2 and 3, the south abutment (Abutment 1) and the southernmost pier (Pier 1) are located on the existing slope which rises up directly from the south side of the river. The second pier from the south (Pier 2) is located between the river to the north and the active railroad tracks to the south. The middle pier (Pier 3) is situated on Oberly Island near the north side of the river. The fourth pier (Pier 4) is located between the canal tow path and the abandoned railroad bed. And finally, the northernmost pier (Pier 5) and the north abutment (Abutment 2) will be constructed near the top of the steep slope that rises directly up from the north side of the river.

The potential locations for the bridge piers were strictly limited due to environmental and right-of-way concerns. Firstly, since the river is heavily used for outdoor recreation, including boating and fishing, no piers were allowed to be constructed within the river. Secondly, the bridge has to span across the active railroad on the south side of the river and across the canal, tow path, and bike path on the north side of the river. Thus, working within the framework of these limitations and the structural requirements to optimize the span lengths, the four span configuration for the bridge, as described above, was developed.

2.0 SUBSURFACE INVESTIGATIONS AND GEOLOGIC STUDIES

The project site is located within the Great Valley section of the Ridge and Valley Physiographic Province of the Appalachian Mountains. The topography of the region is dominated by very broad, parallel ridges and valleys of moderate relief and with a northeasterly trend. The geologic structure of the area consists of thrust sheets, nappes, overturned folds, and steep faults, as characterized by "thin-skinned tectonics". In the vicinity of the proposed bridge, the Lehigh River Valley is underlain by carbonate rocks of the Cambrian-aged Leithsville and Allentown Formations. The surface expression of the contact between these two formations is interpreted as being near the north side on the river.

2.1 Drilling

In early 1994 an initial boring program for the Lehigh River Bridge, consisting of 53 borings, was conducted in order to perform an initial site assessment. These borings were drilled by the Pennsylvania Drilling Company of McKees Rocks, Pennsylvania. The areal distribution of these initial borings reflected several alternative abutment and pier locations that were under consideration at this early stage of the project, and additional borings were spotted at intermediate positions along the centerline of the bridge. The geologic profile which was constructed using this initial boring information revealed that the river valley is underlain by carbonate rock formations with highly variable bedding and fracture orientations, material properties, and degrees of solution weathering. The depth of the bedrock surface was found to vary between only a few feet on the slopes rising up from river to a maximum of about 40 feet on Oberly Island.

Following the initial investigations, the final structure type and span configuration was decided upon, thus allowing subsequent investigations to focus on the exact foundation locations. URS retained the services of Golder Associates (Golder), King Geosciences Inc. (KGI) and EWD Geologic/Computer Consulting to assist in performing in-depth geologic investigations of the project site and to aid in the preparation of the bridge substructure foundation recommendations, including the performance of stability analyses of the river valley slopes.

In July of 1996, Golder and KGI undertook a study of the available subsurface data and a preliminary analysis of the foundation conditions. As a part of their efforts, Golder and KGI performed an in-depth review of the core samples from the initial boring program and conducted geologic reconnaissance at the project site.

During this period, additional test borings were drilled at the locations of Piers 2 and 3 by L.G. Hetager, Inc. and at the south abutment by AWK Consulting Engineers, Inc. In addition, large-scale test pits were excavated in order to expose the top of rock surface at the locations of Pier 1, Pier 4, and Abutment 2.

In April of 1997, Golder and KGI submitted a report of their initial findings. This report discussed various alternative foundation design options at each substructure location, and it presented preliminary rock slope stability analyses for the valley slopes with bridge loads considered. Within this report, it was recommended that oriented core borings be drilled to better identify the attitude of the bedding and fractures within the bedrock.

Following the above recommendation, a series of vertical and angled oriented borings were drilled at the locations of Piers 1, 4, and 5 and Abutment 2. Owing to the difficult access conditions at the boring locations and the requirement to use NQ triple-tube with sophisticated core orienting techniques (clay imprint method), Salisbury and Associates, Inc. of Spokane, Washington was contracted for this work. A photograph of the portable drill rig used by Salisbury is shown in Figure 4. In selected borings at Pier 1, Pier 4 and Abutment 2, HX-sized vertical holes were drilled and water pump-in tests were conducted at various depths to ascertain the permeability of the rock mass and evaluate the possibility of grouting to improve the bedrock.

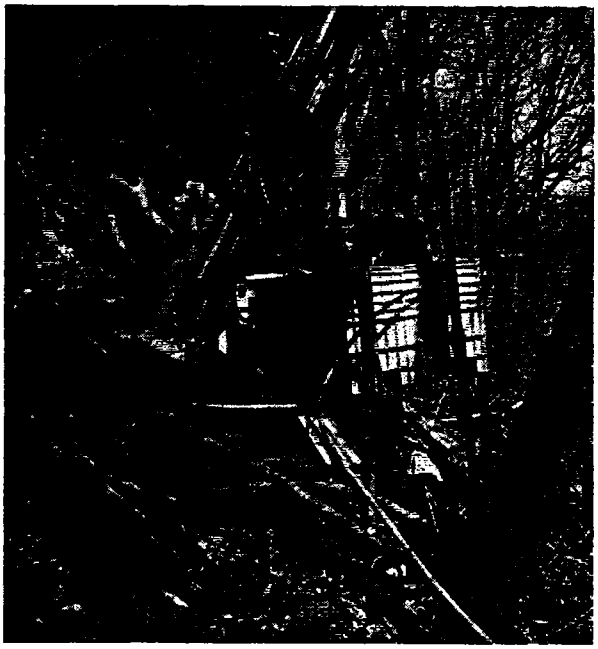


Figure 4 - Portable Drill Rig Used for Inclined Borings

Subsequent to completion of the all of the field investigations and laboratory testing, the final foundation recommendations were prepared by the project team and presented to PennDOT in March of 1998.

2.2 Geologic Interpretation

A geologic cross-section oriented along the bridge alignment was prepared by Golder and KGI based on their interpretations of the site geology, and this is presented in Figure 5. As shown, the rock bedding on the south side of the river is fairly uniform, striking nearly parallel to the river and dipping at roughly 10 to 15 degrees to the north (in the downslope direction). On the north side of the river, the bedding has been completely overturned, with a dip angle of between 65 to 75 degrees to the south (again in the downslope direction) and a strike similar to that on the south side. Beneath the river, the attitude of the rock bedding is interpreted as being transitional. Thus, as illustrated by Figure 5, the Lehigh River is apparently coincident with the axis of a large

antiformal fold at the project location. As shown, the northern fold hinge is apparently overturned, with resultant shortening and shear offset of the beds. The assymetric antiform structure plunges in a westerly direction.

The borings drilled to the south of and directly within the river encountered generally hard dolomite and shaley dolomite with thin to medium thick bedding and very close to medium close fracturing. Thin to medium thick beds of soft dolomitic shale and occassional partings of weak sandstone were found interbedded within the harder rock.

Moving northwards, the borings drilled at the southern half of Oberly Island, (near the location of Pier 3), found the rock to change to soft, intensely bedded, extremely closely fractured and highly weathered dolomitic shale. This bedrock has been eroded to a significant depth by the river, and is overlain by alluvial sand, gravel, and cobble deposits up to 40' thick.

At the location of Pier 4, just to the north of Oberly Island and the canal, very thick beds of soft dolomitic shale were found interbedded with harder dolomite and shaley dolomite. These dolomitic shale beds are typically highly to completely weathered and, as indicated by rod drops through voids and soil-filled cavities, contain numerous solution features. Furthermore, joints in the shale were commonly observed to be smooth and slickensided with sericite coatings, indicative of shear movement within the rock mass. A zone of more massive and hard dolomite was finally encountered at depths of greater than 70' beneath Pier 4.

To the north of Pier 4, the rock abruptly changes to consist primarily of relatively hard and unweathered dolomite. The dolomite bedrock at the locations of Pier 5 and Abutment 2, at the northern extent of the bridge, is generally thinly to medium bedded with close to medium spaced orthogonal joint sets.

2.3 Laboratory Testing

The laboratory testing of rock core samples lifted from throughout the project site yielded uniaxial compressive strengths of between roughly 10,000 psi to 20,000 psi for the dolomite and between only 1,000 psi and 4,000 psi for the impure shaley dolomite and dolomitic shale.

2.4 Groundwater

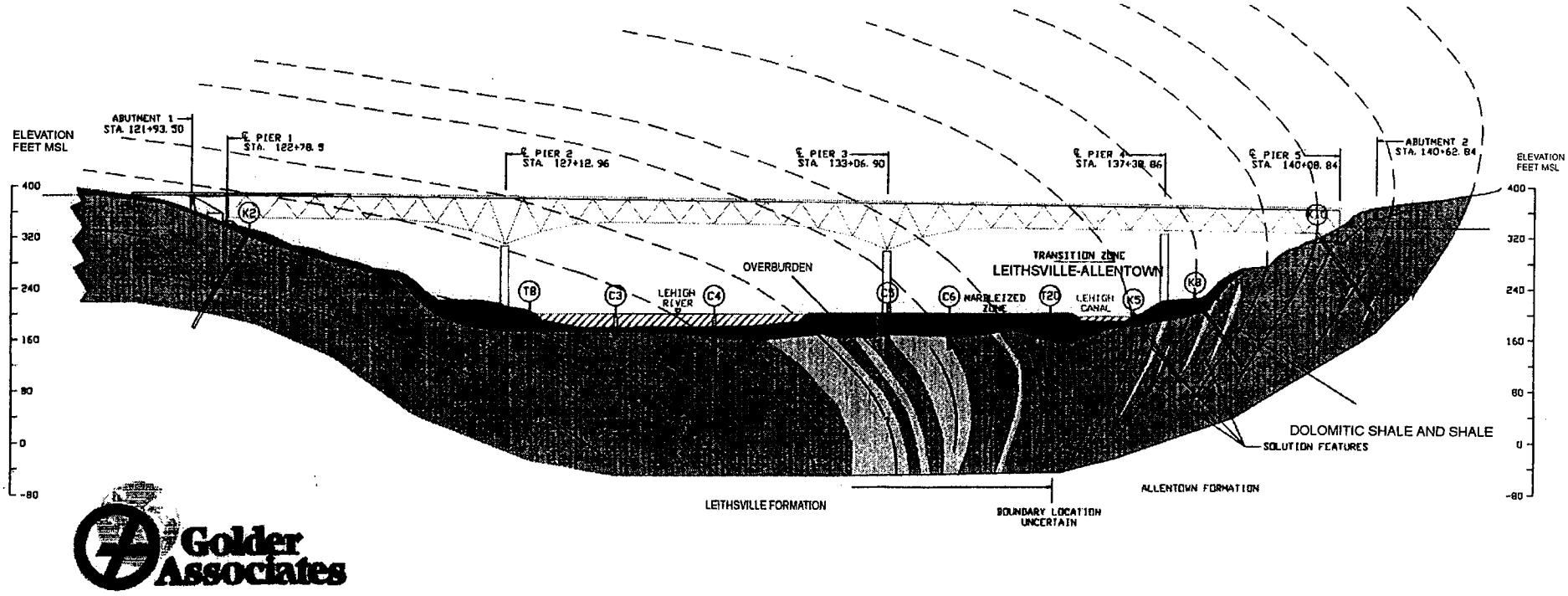
The subsurface investigations found groundwater to be consistently below the top of rock surface except directly adjacent to the river. The significant depth to the groundwater surface is likely due to the high secondary porosity of the carbonate bedrock resulting from fracturing and numerous solution channels.

3.0 FOUNDATION DESIGNS

The following is a summary of the decision process undertaken to determine the appropriate foundation type for each bridge support.

3.1 Abutment 1 and Pier 1

Due to their close proximity to each other, the bedrock conditions at Abutment 1 and Pier 1 were found to be very similar. The borings and test pit in this area indicated that the bedrock surface is



Golder Associates

0 400

SCALE (FT)

FIGURE 5 - GEOLOGIC SECTION ALONG BRIDGE ALIGNMENT

LEGEND

T8 - TEST BORING LOCATION

shallow. However, the rock surface beneath Abutment 1 was found to be pinnacled, with a maximum elevation change of about 18'. Thin to medium thick beds of soft, weathered dolomitic shale and sandstone were observed to be interbedded within the prevailing thickly bedded, hard dolomite. Furthermore, an extensive zone of highly weathered rock was found at the south end of the test pit, and this was interpreted as being related to either a flexure fold in the bedding or an epi-karst feature. Water pump-in tests performed at Pier 1 showed the rock mass to have a moderate secondary permeability.

The allowable design bearing pressure for spread footings on rock at these two locations was found to be low because of the presence of the continuous shale bands directly below the footings and the adverse attitude of the rock bedding. Using the large structure loads at these locations, the required dimensions for spread footing and drilled caisson foundations were developed. Based upon a cost comparison, it was determined that drilled caisson foundations would be less expensive to construct than spread footing foundations at Abutment 1 and Pier 1.

Large diameter drilled caissons were selected in order to allow for the performance of hands-on inspections of the completed rock socket walls. Since the rock sockets will not be excessively deep, and will be above the groundwater table, the rock sockets are easily accessible for inspection. We recommended that the construction of the large diameter drilled caissons with a hands-on inspection of the rock sockets would preclude the need for costly and time consuming load tests in order to verify the caisson capacities. Using this same reasoning, smaller diameter caissons and pin piles were eliminated from consideration because they would have required a load testing program. At Abutment 1, 36" diameter, 10' long rock sockets were used with a design load capacity of 450 kips. At Pier 1, 48" diameter rock sockets with lengths of both 10' and 17' were used, providing design capacities of 650 kips and 1000 kips, respectively. The total lengths of the caissons were expected to range between about 11' and 47' at Abutment 1 and between about 17' and 27' at Pier 1.

An analysis was performed of the overall stability of the sloping bedrock mass south of the Lehigh River, taking into account the bridge loads imposed at Abutment 1 and Pier 1. This analysis utilized the 3-dimensional stability analysis program CLARA. The computed minimum factor of safety against slope failure is 4.9 for service load conditions and 3.1 for seismic conditions.

3.2 Pier 2

The bedrock surface at this pier location was found to vary between 17' and 25' depth, overlain by both residual soils and very soft to medium stiff alluvial soils. The rock is characterized as primarily hard dolomite, but with two distinct interbeds of soft, highly weathered dolomitic shale ranging in thickness between 1' and 9' in the upper portion of the rock profile. Similar to Abutment 1 and Pier 1, the rock bedding at this location generally dips at 10 to 15 degrees to the north.

A spread footing on rock foundation was eliminated as a design alternative for this pier because the vuggy, highly fractured rock mass (with open joints) at the bearing elevation was determined to require improvement by grouting in order to achieve adequate bearing capacity. Since this pier is located adjacent to the Lehigh River and there is a high groundwater pressure gradient from the slope above, flowing water was suspected within the fractures and solution features, and thus grouting was deemed to be impractical and excessively costly. Pin piles were similarly

eliminated due to this same concern with flowing water. The use of 18" diameter drilled shafts was also considered, but the very large axial load at this substructure necessitated the higher capacity afforded by larger shafts.

Pier 2 has been designed utilizing 40 reinforced concrete caissons with 48" diameter, 17' length rock sockets. The concrete is to be placed by tremie below the water table. The sockets extend down into the relatively high quality rock identified beneath the upper shale interbeds, providing an axial capacity of 1,000 kips. The total lengths of the drilled caissons are expected to range from about 55' to 77'.

Out of concern for contamination of the concrete or loss of concrete from the caisson shafts above the rock sockets due to flowing water in the rock mass, each caisson is to be cased with steel casing down to the top of the rock sockets. The casings are to be left in place so that they protect the concrete until it sets.

The construction of Pier 2 will require temporary excavation support, including rock anchors, in order to protect the adjacent active railroad tracks above the pier excavation.

3.3 Pier 3

Since the rock surface is about 40' deep at the location of Pier 3, only deep type foundations were considered for use at this substructure. The bedrock was found to consist of very poor quality, weak dolomitic shale extending to depths of at least 125' below the top of rock surface. The rock is consistently soft to medium hard, highly weathered, and intensely laminated, with rock quality designation (RQD) values near zero. Golder and KGI concluded that the location of Pier 3 coincides approximately with the axis of the antiformal fold that has been identified by the geologic investigations, and that the condition of the bedrock can likely be attributed to tectonic stresses associated with this folding.

A drilled type of foundation was ruled out due to the very poor bedrock quality and due to concerns with possible degradation of the soft shaley rock while drilling the sockets under water. A driven pile foundation was therefore recommended for Pier 3. A relatively large pile section (HP12x74) was chosen in order to reduce the potential for pile damage while driving through the alluvial gravel and cobble overburden. Since the gravel and cobbles are generally well rounded, it is expected that the piles can displace this material, but hard driving conditions are still likely to result.

The design of Pier 3 calls for 276 piles with anticipated lengths of between 27' and 34'. A design allowable pile capacity of 130 kips was used, which is based on an allowable tip pressure of 6.0 ksi in accordance with PennDOT DM-4 requirements for piles founded on carbonate rock. Battered piles have been utilized to resist lateral loads. It has been specified in the construction contract that pile driving is to begin at the center of the pile cap and progress outwards in order to preclude densification of the soils by confinement within the tight pile pattern.

3.4 Pier 4

Of all the substructure locations, Pier 4 was the most challenging from a design standpoint because of the complex and overall very poor rock conditions. As such, Pier 4 was the focus of

intensive field investigations, including the excavation of a 20' deep test pit and the drilling of five angled borings with core reorienting, in addition to numerous vertical test borings. These investigations revealed that, although the bedrock is shallow, there is an abundance of soft, highly weathered dolomitic shale layers as well as numerous solution features (voids and soil seams). The shale layers were observed to be as much as 15' thick (measured perpendicular to bedding), and the bedding dips at about 65 degrees to the south. The hard dolomite that was sporadically encountered is very highly fractured with open joints to a significant depth. However, a steeply-dipping bed of consistently fair quality, hard dolomite was finally encountered at depths of up to 70' beneath the proposed bottom of foundation elevation.

Similar to Pier 2, large groundwater flow rates were suspected to exist beneath the foundation area because the pump-in test inferred a high secondary permeability of the rock mass and because the foundation is essentially at the river level. Therefore, it was not practical to rely on subsurface grouting to improve the rock mass in order to allow for a spread footing foundation. Furthermore, pin piles were eliminated from consideration since they also depend on conditions that are conducive to successful grouting. A driven pile foundation was not considered for this pier because of the shallow and steeply sloping bedrock surface.

Thus, Pier 4 was designed with 40 reinforced concrete drilled caissons with 48" diameter rock sockets, similar to Pier 2. Pier 4, however, utilized 12' length rock sockets compared to the 17' sockets used at Pier 2. The concrete is to be placed by tremie below the water table.

The caissons are to extend into the consistent bed of hard dolomite identified in the borings, with anticipated lengths of between about 73' and 83'. Like Pier 2, 48" diameter steel casing is to be left in place down to the top of the rock socket in order to protect the wet concrete from groundwater flow. A design axial capacity of 800 kips per shaft has been used in designing this foundation.

3.5 Pier 5 and Abutment 2

Pier 5 and Abutment 2 will be perched on the existing 1.5 to 1 slope that rises directly northwards from the Lehigh canal. The excavations for Pier 5 and Abutment 2 will be cut back into the slope somewhat, and then a finished cut rock slope with an inclination of about 55 degrees will be left between the two substructures.

The borings and test pit performed in the area of Pier 5 and Abutment 2 encountered very shallow bedrock consisting of fair quality, hard dolomite bedrock interbedded with thin dolomitic shale seams which are relatively soft and weathered compared to the surrounding dolomite. The rock core recoveries were generally excellent, but the core RQD values were only poor to fair.

The test borings showed that the rock quality does not improve significantly with depth. The rock bedding dips at about 65 degrees toward the south, suggesting that the beds at this location are conformable with those further downslope at Pier 4. Figure 6 is a photograph of the west face of the test pit which was excavated at Abutment 2.

Discontinuities within the rock mass were observed to be tight, both in the cores and in surface exposures. This observation is supported by the water pump-in test performed at Pier 5, which indicates only low to moderate secondary permeability of the rock mass.

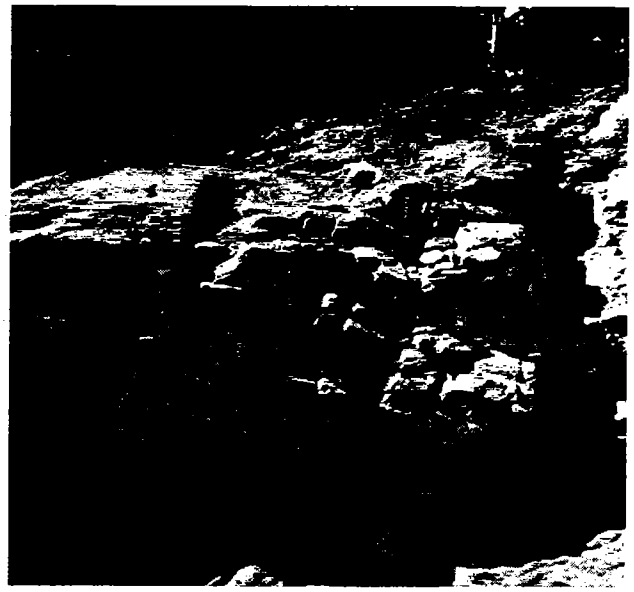


Figure 6 - Abutment 2 Test Pit

Since the loads on these foundations will be relatively light and the rock quality at this location does not improve appreciably with depth, spread footing on rock foundations were determined to be the most practical and cost effective alternative for both Pier 5 and Abutment 2. Drilled caissons, however, were also given serious consideration.

Pier 5 was designed with two separate truss bearing pedestals, each on its own spread footing. These footings were designed for a maximum bearing pressure of 4.0 tons per square foot (tsf). Abutment 2 was designed with a maximum footing pressure of only 3.5 tsf. The low allowable bearing pressures for these spread footing foundations were largely governed by the close proximity of steep rock slopes with an adversely dipping fracture set.

A stability analysis using the program CLARA was performed to evaluate the imposed bridge loads at Pier 5 and Abutment 2 onto the sloping rock mass on the north side of the valley. The computed minimum factor of safety against slope failure was determined to be 3.2 for service load conditions and 3.1 for seismic load conditions.

4.0 CONSTRUCTION CONTROLS

Due to the variable and sometimes poor rock mass conditions at the Lehigh River Bridge, emphasis was placed on preparing construction documents that allowed for a high degree of observation and testing during construction of the foundations. The results of the field monitoring will either confirm the design assumptions or provide the necessary data to adjust the designs where the actual conditions do not match the design assumptions.

All of the caissons were designed using side friction only and not accounting for any contribution from end bearing. During the design, presumptive values for unit side friction were determined by evaluation of the test data and from prior experience. The computed design values of ultimate side shear resistance for the rock socket concrete cast against hard dolomite varied between 90 and 100 psi at the various bridge foundations locations.

For all of the drilled caisson foundations, the anticipated rock socket tip elevations were provided on the contract drawings. However, it was specified that the exact tip elevation of each caisson would be determined during construction based upon the results of additional field testing and inspection.

4.1 Abutment 1 and Pier 1

For the caisson foundations at Abutment 1 and Pier 1, the specifications called for hands-on inspection of the rock sockets by an engineer who would be lowered into the shafts. This is possible because the caissons are well above the groundwater table and because the caissons are expected to have total depths of between about 16' and 47'. The specifications set limits on the maximum cumulative thickness of soft shaley seams and voids that would be allowed within the rock socket interval. If this amount was exceeded based on the inspection, then the socket length is to be increased or the socket interval lowered.

Additionally, at Abutment 1 and Pier 1 test holes were specified to be drilled to a depth of two times the rock socket diameter below each caisson tip to ensure the consistency of the rock quality to that depth. The specifications set minimum rock quality requirements of 80% recovery (of hard dolomite) and 40% RQD for these test holes. It was specified that the test holes be drilled using an NQ-sized triple tube (NQTT) core barrel in order to maximize core recovery.

Since we had a relatively high degree of confidence in the quality and consistency of the rock at Abutment 1 and Pier 1 and since a hands-on inspection of each caisson is possible, load testing was not deemed necessary for these two substructure units.

4.2 Piers 2 and 4

At Piers 2 and 4, hands-on inspection of the rock sockets was considered to be impractical because of the shallow groundwater table and the much greater anticipated lengths of the caissons. Furthermore, our confidence in the quality and consistency of the bedrock was not high, particularly at Pier 4. Therefore, it was recommended that an Osterberg load test be performed on a production shaft at each of these two pier foundations. The tested caissons will be loaded to two times the required design capacity.

The final caisson rock socket tip elevations at Piers 2 and 4 will be determined on the basis of NQTT test core holes that are drilled at each caisson location prior to the drilling of the caissons. The test holes were specified to be drilled to a depth where they obtained minimum 90% recovery (of hard dolomite) with 40% RQD for the design socket length plus two times the socket diameter.

As a contingency, provisions have been made in the construction contract to provide a downhole camera for inspection of the drilled NQTT proof hole or the sidewalls of the caissons if deemed necessary.

In addition to the specification requirements for ensuring the load capacity of the caissons, as described above, vertical steel tubes are being installed in each caisson to allow pile integrity testing by cross-hole sonic logging in order to detect defects in the casting of the caisson concrete.

4.3 Pier 3

For the pile driving at Pier 3, it has been specified that the driving of a percentage of the HP12x74 piles is to be monitored using a pile driving analyzer (PDA). This will allow for the detection of pile damage that could occur while driving through the alluvial gravel and cobble deposits. Furthermore, the PDA data is to be used to perform CAPWAP analyses to confirm the pile capacities.

4.4 Pier 5 and Abutment 2

For the spread footing on rock foundations at Pier 5 and Abutment 2, the completed footing excavations are to be inspected by an engineer for the presence of soft, weathered shale seams. The specification requires the contractor to gouge out all such material to a depth of three times the seam width and backfill with concrete as the footing is cast.

5.0 CONSTRUCTION OBSERVATIONS TO DATE

Construction of the Lehigh River bridge began in April, 2000. The general contractor for this project is the Dick Corporation, and the caisson drilling subcontractor is McKinney Drilling Company. In May, 2000, the drilling of NQTT test core holes was initiated at Pier 1 by McKinney's subcontractor E2SI, Inc. In order to save time, the contractor has elected to drill the test core holes at Abutment 1 and Pier 1 all the way from the bottom of pile cap elevation rather than starting them from the caisson tips after the rock sockets have been drilled. As of June, 2000, the test holes had been completed at Pier 1 and Abutment 1, and were in progress at both Piers 2 and 4. Figure 7 is a photo showing two drill rigs performing NQTT test cores at the Pier 4 foundation.

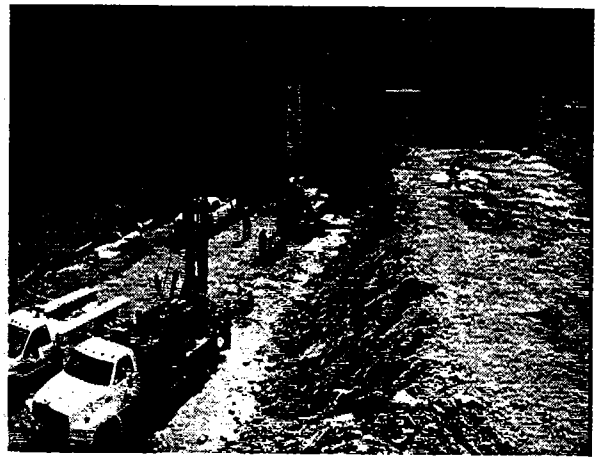


Figure 7 - Rigs Drilling NQTT Borings at Pier 4

Based on the results of the completed NQTT borings, the rock socket tip elevations have been set for both Abutment 1 and Pier 1, pending final approval of the hands-on inspections. At Abutment 1, the actual tip elevations are very close to the estimated tip elevations, however at Pier 1, the actual tip elevations are consistently much deeper than originally estimated. The borings completed to date at Pier 4 have confirmed the presence of very poor and erratic subsurface conditions, highlighting the importance of conducting an evaluation at each individual caisson location.

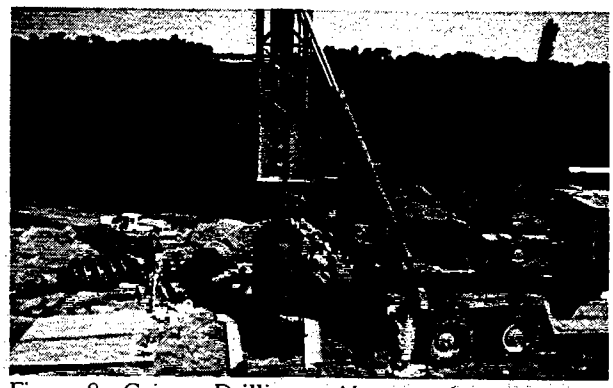


Figure 8 - Caisson Drilling at Abutment 1

Caisson drilling has been started at both Abutment 1 and Pier 1 (see photo in Figure 8). To date, URS has performed hands-on inspections for about half of the caissons at Abutment 1 and Pier 1. In all cases, only minor seams of soft, weathered shale and minor voids have been observed along the walls of the rock sockets, and therefore each of these caissons has been approved for the socket tip elevations determined by the NQTT borings.

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Impacts of a Buried Paleochannel on Foundation Designs for the Box Canyon Wash Bridge, U.S. 93, Wickenburg – Kingman Section, in Northwestern Arizona

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ABSTRACT

The geotechnical investigation for a new three-span, 380-foot long bridge over Box Canyon Wash (Milepost 131.8) on U.S. Highway 93 in Mohave County, Arizona, revealed an old, deep and very narrow paleochannel which required special design consideration relative to foundation support of the new bridge. The bridge site is located near the south end of the Big Sandy Basin, on a gently sloping surface extending from the Aquarius Mountains to the Big Sandy River in the valley floor. The geologic units underlying this area primarily consist of two sedimentary basin fill deposits, Tertiary or later in age, with different geotechnical characteristics.

The lower basin-fill deposit is the oldest unit and consists of a thick sequence of soft sedimentary rocks varying from conglomerate along the edge of the valley, to mudstone in a lacustrine environment in the valley floor. These materials were deposited partially in a closed basin. The upper basin-fill deposit is the younger unit and is the remnant of an old alluvial fan that was deposited on a highly irregular and eroded surface of the lower basin fill after lowering of the basin floor and reopening of the closed basin. The upper unit is less consolidated and consists of dense, locally lime-cemented silt, sand, gravel and cobbles with some boulders. Remnants of the upper basin-fill alluvial fans dominantly occur as old filled paleochannels cut into the lower unit or as caps on ridges which subsequently were dissected by the current drainage system.

The bridge across Box Canyon Wash will consist of a three-span bridge with abutments situated on ridges which bracket Box Canyon Wash and piers within the wash bottom. Lower basin fill (conglomerate and sandstone) are exposed in most of the canyon walls, and were encountered at shallow depths in the piers and north abutment borings. However, a boring completed at the south abutment encountered an unanticipated paleochannel filled with the less consolidated lower basin fill. The depth of the paleochannel is at least 90 feet below the ground surface at the abutment location, which exceeds the depth of the adjacent, current box canyon which is walled with lower basin fill rocks.

Standard penetration tests in borings completed in soft rock at the center piers and north abutment consistently refused within the first few inches, compared to more variable SPT N-values of about 50 to 70 to a depth of 87 feet in the upper basin fill in the south abutment boring. Higher N-values in the upper basin fill typically reflected individual gravel or cobbles or isolated cemented layers.

As a result of these variable conditions, two different foundation systems were required to support the new bridge, including 6-foot diameter rock sockets in soft rock, and 6-foot diameter drilled shafts in the less consolidated material in the isolated paleochannel. Conventional methods of foundation analysis, including design of drilled shafts at the south abutment in the upper basin fill (paleochannel), and design of rock sockets in the lower rock unit, were utilized. Due to low strength of the lower unit, rock socket design lengths were comparable to the lengths of the drilled shafts in the paleochannel.

INTRODUCTION

This paper presents the results of a geotechnical investigation of the site of the proposed new three-span, 380-foot long, bridge structure bridge over Box Canyon Wash, located at Milepost (MP) 131.8 on U.S. Highway 93, approximately five miles south of the town of Wikieup, in Mohave County, Arizona (Figure 1). The bridge site is located near the south end of the Big Sandy Basin, on a gently sloping surface which extends from the Aquarius Mountains to the Big Sandy River in the valley floor. The geologic units underlying this area primarily consist of two sedimentary basin fill deposits, Tertiary or later in age, with different geotechnical characteristics. The subsurface investigation revealed an old, deep and very narrow paleochannel at the south abutment location which required special design consideration relative to analysis and design of foundations to support the new bridge.

The Box Canyon Wash bridge will be part of the planned improvements to the four-mile long Signal Road Section of U.S. Highway 93, which extends from the approximate limits of MP 129.0 to MP 133.0. The proposed improvements include construction of a new

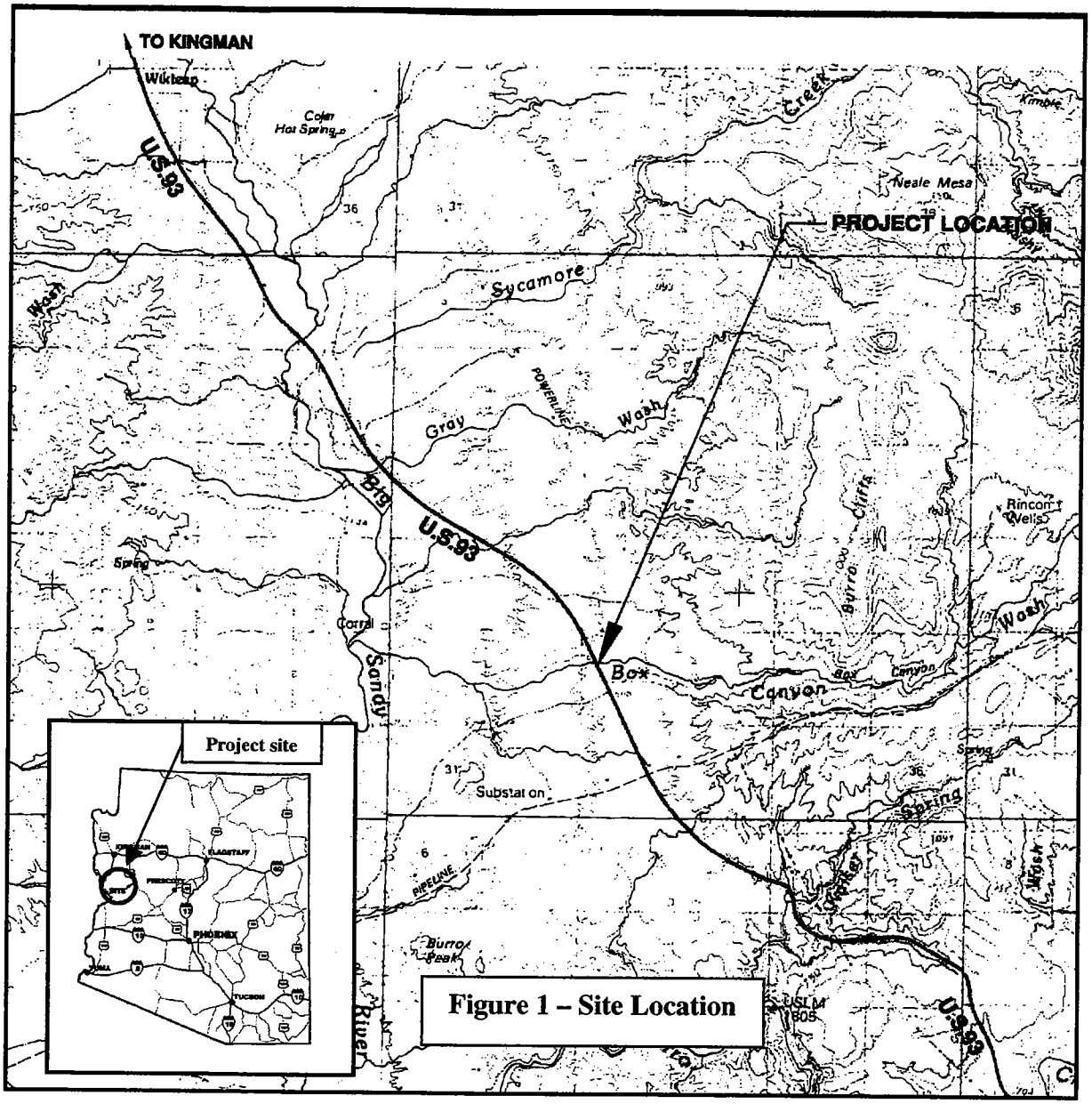


Figure 1 – Site Location

two-lane, southbound roadway which essentially parallels the existing highway, thus providing a four-lane divided highway with a variable-width median. A concrete box culvert currently is located at the existing U.S. 93 crossing of Box Canyon Wash.

The purpose of the project is to improve safety, capacity and operational characteristics of this segment of U.S. 93, while minimizing environmental impacts and preserving its scenic resources. This route is part of the National Highway System and has been designated as the Canamex Corridor.

The geotechnical investigation of the site included the following elements:

- Collection and review of existing data and imagery.
- Completion of surficial geologic mapping of the site.
- Subsurface exploration, including advancing four test borings at the planned bridge abutment and pier locations to depths ranging from 50 to 87 feet below existing grades, and four additional borings immediately north of the planned bridge (within a planned roadway cut).
- Completion of selected laboratory testing of representative rock core samples.

GEOLOGICAL SETTING

The Signal Road Section of U.S. Highway 93 (U.S. 93) is located within the southeastern portion of Mohave County in northwestern Arizona. This section of U.S. 93 lies in the southern portion of the Big Sandy Structural Basin within the Basin and Range Physiographic Province. The geomorphology typical of this province consists of a series of elongated and

parallel mountain ranges, oriented north-south to northwest-southeast, separated by intervening basins or valleys. The mountain ranges represent uplifted structural blocks, and the Big Sandy Valley, along which U.S. 93 extends, represents the intervening structural subsidence feature. The Big Sandy Basin is bordered by the Hualapai and Peacock Mountains on the west, which range in elevation from about 6000 feet to about 8300 feet above mean sea level, and the Aquarius Mountains and Cottonwood Cliffs on the east, which range in elevation from about 5000 to 6000 feet. The central floor of the Big Sandy Basin located between the mountain blocks ranges in elevation from about 4000 feet at the northern end to about 1700 feet within the southern portion.

The geologic units associated with this geomorphology and exposed in the region generally include Precambrian to Tertiary bedrock in the mountain areas and a Tertiary to Quaternary basin-fill sequence along the flanks of the mountain ranges and within the floor of the basin. The bedrock units in the mountains primarily include Precambrian granite and schist and localized Tertiary silicic to basaltic volcanic rocks and sedimentary rocks. The basin-fill sequence that has infilled the valley includes volcanic rocks similar to those in the mountain blocks and multiple Tertiary to Quaternary sedimentary deposits. The volcanic rocks of the basin-fill sequence are most abundant within the extreme southern portion of the Big Sandy Basin and represent a volcanic center derived primarily from eruption along a local cluster of vents associated with local Basin and Range structural features (Moyer, 1982). The Tertiary volcanic sequence located in this extreme southern portion of the basin completely filled the basin and resulted in damming or plugging of the ancestral basin floor, forming an internally-drained system within Big Sandy Valley.

The oldest (Tertiary) basin-fill sedimentary deposits, northward from this volcanic center include a progressively finer-grained sedimentary sequence of coarse-grained fluvial conglomerate near the volcanic center, sandstones, and ultimately a lakebed or lacustrine deposit of mudstones near Wikieup. The lacustrine deposit is evidence that the Big Sandy Valley once was internally drained. The sedimentary rocks are flat-lying and only slightly deformed, with bedding dips generally less than 10 degrees. Normal high-angle faults of slight displacement locally cut the sedimentary sequence. This sequence s referred to herein as the lower basin-fill deposit and is the youngest of the material characterized as a rock unit.

Following deposition of the lower basin-fill sequence and eventual overtopping and erosion of the volcanic dam, the basin returned to a through-draining system. The lower basin-fill materials were then subjected to renewed erosion and were partially removed along drainages as the river system cut into the preexisting basin floor. Upon temporary stabilization of the basin floor or uplift of the adjacent mountains, younger alluvial fans grading to the new river bed level were then deposited as a thin sheet on the flanks of the new basin floor and filled in the previous developed drainages. These younger alluvial fan deposits are likely late Tertiary to Quaternary in age and consist of coarse-grained material referred to herein as the upper basin-fill deposit. Subsequent to deposition of the alluvial fans, the floor of the basin was again lowered with local downcutting along current drainages and dissection of the previously developed alluvial fan surface, leaving erosional remnants of the deposit on ridge tops.

SITE GEOLOGY

The Box Canyon Wash Bridge site within the Signal Road Section of U.S. 93 lies along the gently sloping surface of the basin floor which slopes north-westward toward the Big Sandy River, which drains the central floor of the Big Sandy Valley to the south. The geologic units exposed in the sloping surface of the basin floor at the site include the lower and upper basin-fill deposits and very localized, thin deposits of alluvium as described below.

Lower Basin-Fill Deposits

The lower basin-fill deposit outcrops throughout a majority of the project site, and ultimately underlies the entire site at depth. The lower basin-fill deposit is the oldest consolidated unit exposed at the planned bridge site and consists of soft sedimentary rocks, including conglomerate and lesser sandstone. The different rock types generally are interfingered and, in some instances, the contacts are gradational in nature, with a generally finer grained texture and increase in amount of sandstone to the north. Exposures of the lower basin-fill rocks form the steep-walled portion of the active drainage of Box Canyon Wash. The surficial distribution of the geologic units in the immediate area of the planned new bridge is shown in Figure 2. The sandstone lithology is grouped with the conglomerate on the figure due to limited extent of the sandstone.

The lower basin-fill is unconformably overlain by the less consolidated upper basin-fill alluvial fan deposits of sand, gravel and cobbles in the flatter slopes on the ridges above the canyon walls both north and south of the Box Canyon Wash drainage, and by recent alluvium within the floor of the drainage. Since the upper basin fill unit was deposited upon the conglomerate on an irregular and undulatory erosional surface formed during

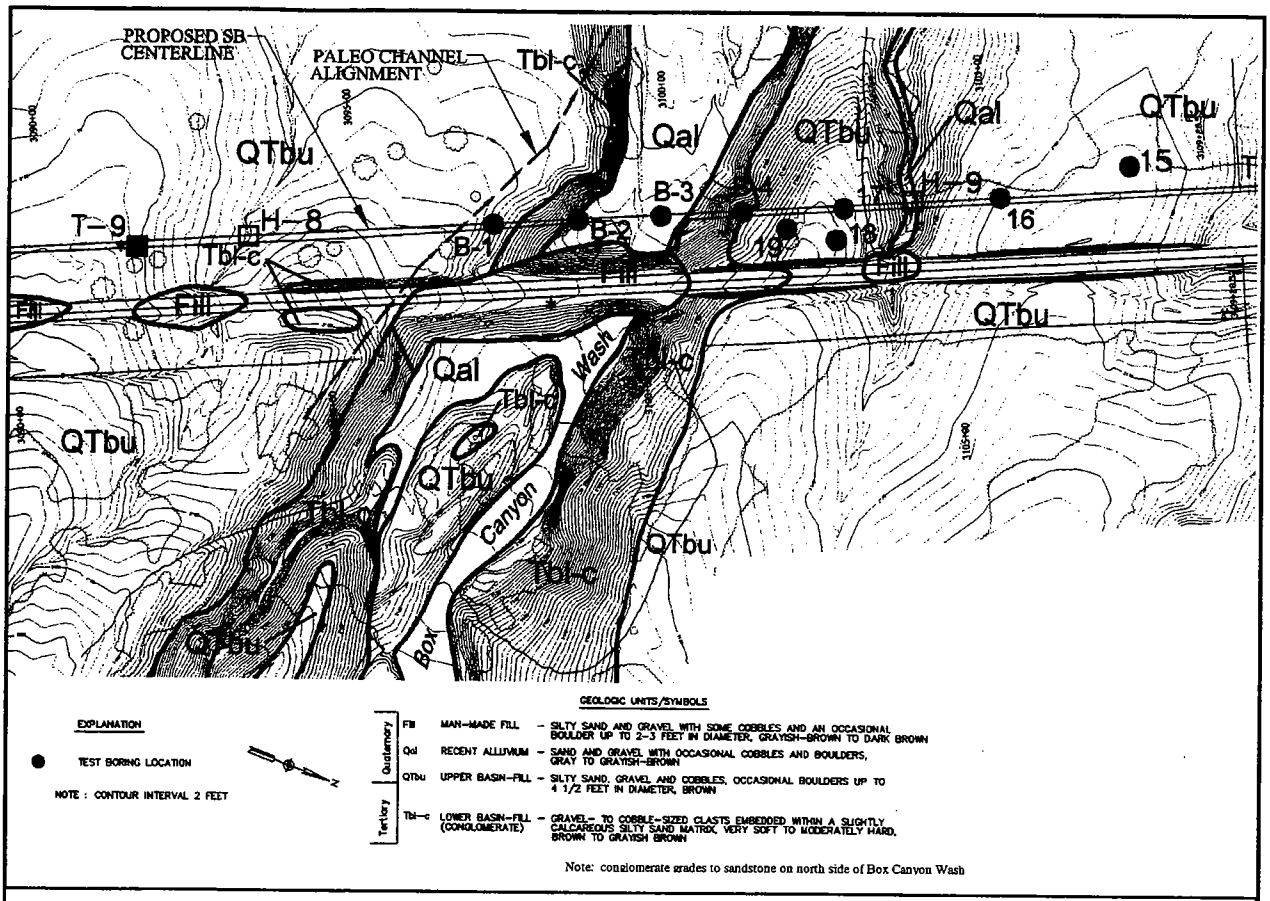


Figure 2 – Showing Geology & Test Boring Locations

downcutting of the basin floor, the contacts between the two units commonly can be very steep and locally vertical particularly where old steep-walled canyons have been filled in during deposition (paleochannels). An example of the irregular contact is shown in Photo 1. A geologic profile which depicts the subsurface distribution of the units along the bridge alignment is presented in Figure 3.

The texture of the conglomerate consists of gravel- to cobble-size, subrounded to subangular clasts embedded within a silty to clayey sand matrix. The silty sandstone layers are composed of fine to medium grained sand with some gravel clasts. The exposures of the two rock types are unweathered to slightly weathered, medium to thickly bedded, and brown to grayish-brown. The rock mass within exposures generally is unfractured with rare localized and discontinuous vertical joints exposed primarily only within the sandstone intervals, and occasional horizontal fractures observed within the borings. The unit generally is weakly indurated and is soft to locally moderately hard with a local calcareous matrix in stringers and lenses. Measured unconfined compressive strengths for the conglomerate are relatively low, ranging from 61 to 937 pounds per square inch (psi). Dry density test results from core samples ranged from 107 to 129 pounds per cubic foot (pcf).

Upper Basin-Fill Alluvial Fan Deposit

The upper basin-fill deposit is extensively exposed primarily in the ridges in the area of the Box Canyon Wash. The upper basin-fill is younger and less consolidated than the lower basin-fill sedimentary rocks and is classified as a soil, although locally cemented lenses are similar in character to the underlying conglomerate. The upper basin-fill is composed of sand, gravel and cobbles that were deposited as a series of alluvial fans on the sloping basin floor.

As a result of erosion subsequent to deposition of the alluvial fans, the upper basin-fill now blankets the hillsides located in the area of Box Canyon Wash, typically as capping layers on the ridges with the underlying lower basin-fill rocks more commonly exposed in the lower slopes of the drainages. The deposits also locally occur as filled paleochannels where previous drainage channels that cut into the surface of the lower basin-fill became filled during initial deposition of the fans (Figure 3).

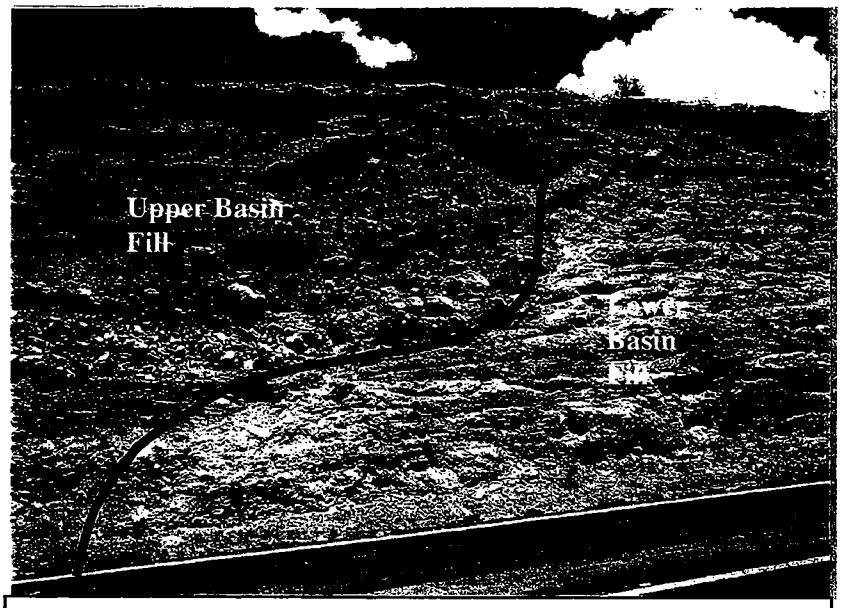


Photo 1 – Contact between upper basin fill and lower basin fill showing irregular and locally steep nature.

The upper basin-fill is weakly to moderately stratified with finer grained (sandy gravel) layers interbedded with coarser grained (sand, gravel and cobble) layers as shown in Photo 1. The unit includes some small-scale cross-bedding features, and unconformably overlies the lower basin-fill on an irregular erosional surface. The upper basin-fill consists of subangular to subrounded, predominantly medium- to coarse-grained sand; subangular to subrounded, well-graded gravel; and cobbles. Some boulders up to 4-1/2 feet in diameter occur within the unit. Minor lenticular clayey zones and discontinuous calcium carbonate-rich cemented layers also are present within this unit. The predominant cemented layer appears as a hardened, lime-cemented cap at numerous road cuts, although cemented layers also occur locally within the unit. The materials mostly are nonplastic and light gray to grayish brown.

Recent Alluvium – Stream Channel Deposits

The current topography of the basin floor in the site area, which is characterized by a gentle, west to northwest sloping floor dissected by numerous incised drainages, is the result of the latest stage of basin floor erosion. Recent stream channel deposits occur within the active drainage paths. The recent alluvium at the Box Canyon Wash crossing is about 10 feet thick and 150 feet wide. The alluvium primarily consists of sand and gravel with varying amounts of cobbles, silt and occasional boulders up to 3-1/2 feet in diameter. The deposits generally are comprised of medium- to coarse-grained, subrounded to subangular sand and well graded, subrounded to subangular gravel. The stream channel deposits typically are loose at the ground surface and medium dense to very dense a few feet below existing site grades. The deposits typically are nonplastic and brown to grayish-brown. Man-made fill composed of material similar to the recent alluvium and derived from local excavations also is present at the site as roadway embankment fill.

GEOTECHNICAL PROFILE AT BRIDGE SUPPORTS

The planned new bridge will span Box Canyon Wash and extend from approximate SB Stations 3096+54 to 3100+32. The south abutment is located on the edge of a southeast-northwest trending, gently dipping ridge south of Box Canyon Wash and the north abutment is positioned on the edge of a similar southeast-northwest trending, but slightly more incised and moderately dipping ridge north of Box Canyon Wash. Two bridge piers are planned within the drainage floor of Box Canyon Wash. The proposed roadway grade of the bridge will vary in elevation from about 2160 feet at the south abutment to 2150 feet at the north abutment. A summary of the borehole data at the bridge site is presented in Table 1, and discussions of the specific geotechnical conditions at each abutment and pier location are presented in the following sections. As shown in the table, three substructure units will be within the lower-basin fill and one will be within the upper basin-fill materials. It should be noted that the roadway centerline stationing was adjusted subsequent

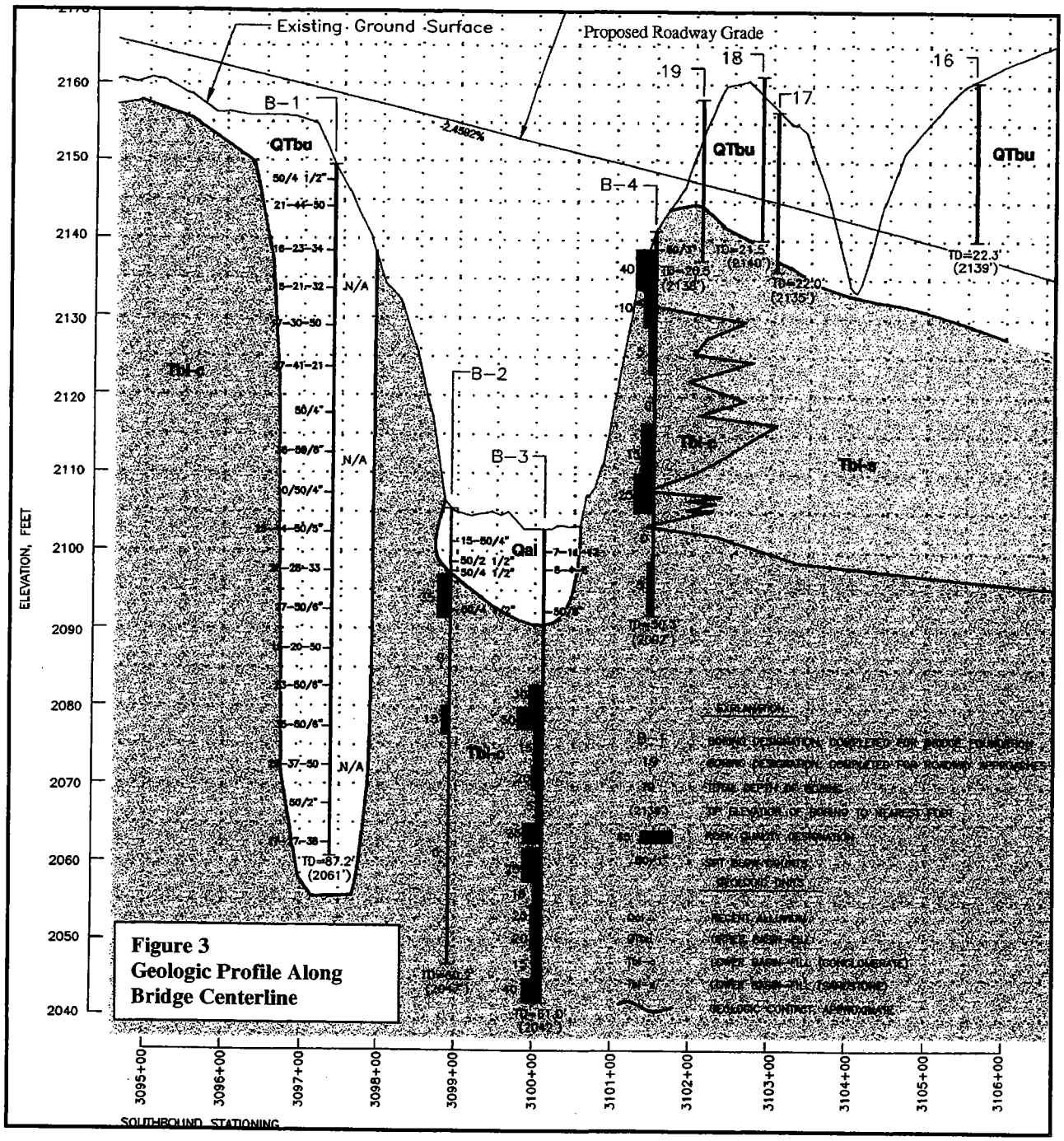


Figure 3
Geologic Profile Along Bridge Centerline

Table 1
Summary of Borehole Data – Southbound Bridge at Box Canyon Wash

Boring Designation	Boring Location (SB Station & Offset)	Bridge Location	Depths of Geologic Units (feet)		
			Recent Deposits	Upper Basin-Fill Deposits	Lower Basin-Fill Deposits
B-1	3097+47, ℄	South Abutment	--	0 – 87.2	--
B-2	3098+87, ℄	South Pier	0 – 9	--	9 – 60.2
B-3	3100+11, ℄	North Pier	0 – 12	--	12 – 61.0
B-4	3101+49, ℄	North Abutment	0 – 1	--	1 – 50.3

to completion of the subsurface investigation; therefore, the boring locations shown in Table 1 do not match the abutment and pier locations shown in Figure 4.

South Abutment

The south abutment of the planned bridge lies on the north facing sideslope of a gently dipping, southeast-northwest trending ridge at an elevation of about 2148 feet. The geologic units exposed in the area of the south abutment, as shown in Figure 2, primarily include the upper basin-fill alluvial fan deposit on the ridge top and lower basin-fill conglomerate in the adjacent canyon wall below the ridge top.

The upper basin-fill deposit dominates the footprint area of the planned abutment, and this unit was encountered throughout the entire depth of Boring B-1 (87.2 feet) to a tip elevation of 2061 feet, as shown in Figure 3. The upper basin-fill encountered within Boring B-1 consisted of very dense, weakly stratified, lenticular soils with variable amounts of fine-grained layers, and coarser layers with cobbles and boulders up to 2 feet in diameter. However, boulders up to 4-1/2 feet in diameter also locally occur within this deposit. The SPT blow counts for the upper basin fill are highly variable, and only locally are refusal blow counts typical within either the coarser grained or cemented zones. The blow counts are summarized on the geologic profile in Figure 3.

Also as shown in Figure 3, deposition of the upper basin-fill unit encountered in Boring B-1 represents a very narrow and steep-sided paleochannel. The filled paleochannel is an old drainage feature that was once a steep-walled canyon cut into the lower basin-fill. Based on the distribution of outcrops of the lower basin-fill at the site, it appears that the paleochannel trends to the west and is an isolated feature passing through the area of the south abutment. The estimated alignment of the paleochannel of upper basin-fill is shown in Figure 2.

South Pier

The south pier is located at the south edge of the drainage floor next to the wall of the Box Canyon Wash drainage. The geologic units exposed in the area of this pier, as shown in Figure 2, include lower basin-fill conglomerate in the slopes above the drainage and recent alluvium in the drainage floor. Boring B-2 encountered recent alluvium (unconsolidated stream channel deposits) from the ground surface to a depth of about 9 feet below existing grades. The alluvium was dense below a depth of about 4 feet. Lower basin-fill conglomerate was encountered below a depth of about

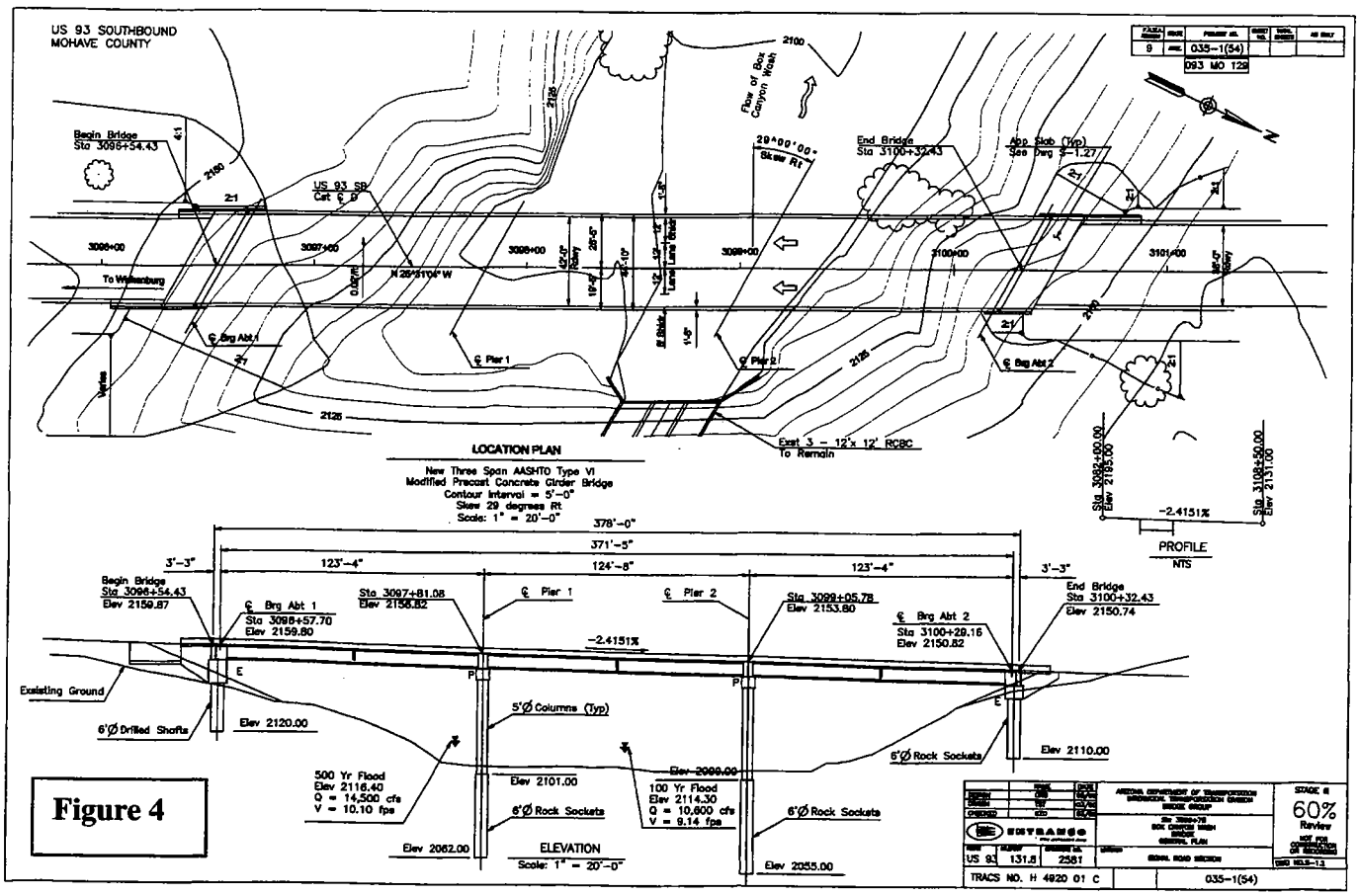


Figure 4

9 feet and throughout the remainder of the boring, to a depth of 60 feet (tip elevation of 2047 feet). The conglomerate generally is unfractured with only rare fractures evident. The low to zero rock quality designation (RQD) values recorded and shown on Figure 3 indicate the generally soft rock conditions, with some moderately hard layers, and thin very soft zones below a depth of 49.5 feet. Some lenses of clayey matrix and occasional oxidized zones were encountered within the boring.

North Pier

The north pier is located within the northern portion of the Box Canyon Wash drainage. The geologic units exposed in the area of this pier include lower basin-fill conglomerate and sandstone in the adjacent slopes and recent alluvium in the drainage. Boring B-3 completed at this pier location encountered recent alluvium (unconsolidated stream channel deposits) from the ground surface to a depth of about 12 feet below existing grade. The soils were loose to medium dense from the ground surface to a depth of 10 feet and very dense below 10 feet. Lower basin-fill conglomerate was encountered below a depth of 12 feet, and throughout the remainder of the boring, to a depth of 61 feet (tip elevation of 2042 feet). The conglomerate generally is unfractured, and low RQD values indicate generally soft rock conditions. The conglomerate encountered within the boring was moderately soft to moderately hard with thin soft to very soft zones. The conglomerate is locally interfingered with sandstone from this location northward, and an occasional thin, soft sandstone layer was encountered below a depth of 58.5 feet in the boring.

North Abutment

The north abutment of the planned bridge lies on the south-facing sideslope of a moderately dipping, southeast-northwest trending ridge, at an elevation of about 2142 feet. The geologic units exposed in the area of the north abutment primarily include the upper basin-fill in the upper section of the ridge and lower basin-fill conglomerate and sandstone in the lower canyon wall.

Boring B-4 encountered about 1 foot of residual soil overlying the lower basin-fill deposit of conglomerate and sandstone. The rock generally is unfractured, and low RQD values indicate the soft nature of the rock. The conglomerate is moderately soft to moderately hard with thin soft to very soft lenses below 10 feet. As shown in Figure 3, a facies change from conglomerate to sandstone occurs in the area of the north abutment. Four additional borings (nos. 16 through 19, inclusive) were completed along the approach to the bridge just north of the proposed north abutment, as shown in Figures 2 and 3. Borings 17 and 19 encountered the upper basin-fill overlying the lower basin-fill sandstone unit. Borings 16 and 18 encountered the upper basin-fill material from the ground surface to the bottom of the borings. The upper basin-fill consisted of weakly stratified silty sand, gravel and cobbles, with boulders up to 2 1/2 feet in diameter. The underlying sandstone is thinly to medium bedded and very soft to moderately hard. Fractures were absent within the depths of investigation of Borings 17 and 19, and the observed low RQD values indicate the soft nature of the sandstone.

BRIDGE DESIGN

Approach and Methodology

During the design concept development phase for the U.S. 93 corridor, the existing concrete box culvert at Box Canyon Wash was identified for replacement. This existing drainage structure has insufficient capacity to adequately pass the 100-year design flood. Further, the Box Canyon Wash site is acknowledged to be a major game crossing location. The requirement to pass the 100-year storm event flood, coupled with the requirement to provide a game crossing structure, warranted a bridge structure length of about 300 feet.

Several bridge types were considered for the site, including:

- Clear-span steel arch bridge
- Three-span, precast, prestressed, concrete AASHTO Type VI modified girder bridge
- Two-span, cast-in-place, post-tension concrete box girder bridge
- Two-span, structural steel plate girder bridge.

The selected bridge type, based on cost, ease of construction and maintenance, was a concrete AASHTO Type VI modified girder bridge.

The proposed bridge is located on a tangent horizontal alignment and a 2.5% vertical grade (Figure 4). The skew of Box Canyon Wash is 29° to the right of the bridge centerline. The cross section of the bridge is a total of 42 feet in width, including two 12-foot travel lanes, four-foot inside and 10-foot outside shoulders, and two feet of additional widening on either side for barrier connections. The bridge barrier will be a 2'-8" high, Type "F" concrete barrier.

The bridge length was determined to be 378 feet. Although the bridge height is approximately 28 feet above the 500-year flood event in the wash, the impacts of lowering the roadway profile are cost prohibitive. Lowering of the profile would generate a significant additional quantity of roadway excavation, due to the high cut slopes on either end of the bridge which flank Box Canyon Wash. Thus, the bridge length and height are controlled by the roadway profile and earthwork considerations and not by floodway requirements. Based on support of the bridge on paired drilled shafts, total dead plus live loads at the pier and abutment drilled shafts are 1,200 kips and 1,600 kips per shaft, respectively (one kip equals 1,000 pounds).

Foundations

As shown in Figures 3 and 4, the southbound roadway at the Box Canyon Wash bridge abutments will be located at about elevations 2160 to 2150 (south to north abutment) due to the planned roadway grade, or about 45 to 55 feet above the flowline of Box Canyon Wash (elevation 2105 feet). The intermediate bridge piers will be located within the wash channel. Based on the borings, the depth to competent bedrock, defined herein as the lower basin-fill (conglomerate) varies along the length of the bridge structure as discussed previously. The conglomerate was not encountered to a depth of 87 feet below existing grade (elevation 2061 feet) at the south abutment, due to the apparently steeply inclined geologic contact between the conglomerate and the adjacent paleochannel. However, the conglomerate was encountered at the piers and the north abutment at depths of about one to 12 feet below existing grades. It is noted that the roadway cut located to the north of the bridge site encountered the sandstone unit of the lower basin-fill; the geologic contact between the conglomerate and sandstone units is thought to be interfingered and gradational, such that sandstone layers may be encountered in the conglomerate at the north bridge abutment foundation excavations.

The planned abutment foundations will be laterally and vertically removed from the active channel bottom of Box Canyon Wash by about 100 feet or more of the conglomerate unit as shown in Figure 2; therefore, the effects of scour within Box Canyon Wash on the proposed abutment foundations did not merit consideration. However, the bridge pier foundations within the bottom of Box Canyon Wash required consideration of the effects of scour. Based on the results of rock strength and density tests, the compressive strength and density of the conglomerate (silty to clayey sand matrix) were variable; the mean and standard deviation of the unconfined compressive strength data for the conglomerate (8 values) were about 425 psi and 287 psi, respectively. Given this data, it was anticipated that the conglomerate would not be susceptible to other than very localized scour effects within softer zones of the rock unit, or within zones near the contact with the overlying wash bottom alluvium. Adequate penetration of the pier foundations into the conglomerate should serve to minimize or eliminate the potential effects of scour.

Given the variable depth to competent conglomerate bedrock across the site, it was recommended that the south bridge abutment be supported on drilled shafts founded in the upper basin-fill silty sand, gravel and cobble unit, and further that the bridge pier foundations and the north abutment be supported on either rock sockets (drilled shafts) socketed into the conglomerate, or on spread-type footings (abutment only) penetrating and bearing directly on the conglomerate unit. Use of spread-type footings for support of the south abutment was considered as an alternative, as the potential for excessive differential settlement was deemed to be low, due to the dense to very dense nature of the upper basin-fill

Although spread footings were analyzed as a foundation alternative at both abutment locations, it was recommended to use drilled shafts for the final design. The potential exists for localized zones of softer upper basin-fill materials creating excessive or differential settlements after application of loads, resulting in the need to overexcavate and replace these zones with lean concrete slurry, were spread footings selected. Drilled shafts comprise the preferred foundation alternative, given the existing subsurface conditions. Although spread footings could be utilized at the north end, drilled shafts (rock sockets) were selected for this abutment to preserve consistency, lower the drilled shaft unit cost, and help simplify design and construction documentation preparation.

Based on SPT blow counts, the upper basin-fill unit encountered at the south abutment generally is medium dense to very dense and contains occasional boulders. Below a depth of about 30 feet in the boring, refusal blow counts typically were recorded, indicating very dense conditions except for an occasional medium dense stratum. Given the relatively dense nature of the unit, it was anticipated that potential differential settlement between the south abutment drilled shafts and rock sockets founded in the conglomerate for the piers and north abutment would be minimal.

Pier considerations included 4-, 5-, 6- and 10-foot diameter rock sockets. The final design of two 6-foot diameter rock sockets at each pier was selected for cost considerations. The round columns will be formed as an

extension of the drilled shafts and will support 5-foot diameter columns and a pier cap as shown in Figure 4.

The AASHTO Type VI Girder alternative was deemed to be the most cost effective layout. The ease of construction and the minimal impact to the existing wash made this option the recommended bridge type.

South Abutment

Drilled shafts bearing in the upper basin-fill sand, gravel and cobbles were proposed for support of the south bridge abutment. Drilled shaft capacities for compressive loads were computed using the "Beta" method recommended by AASHTO (1996). Vertical compressive drilled shaft capacities were estimated using the governing equation which equates the shaft compressive capacity to the sum of the shaft side resistance and tip resistance, minus the shaft weight.

Shaft side resistance is a function of the shaft diameter, a load factor coefficient for the load type, depth from the ground surface to midpoint of the soil layer of interest, and the thickness of the soil layer. Shaft tip resistance is the product of the unit shaft tip resistance [determined in accordance with Table 4.6.5.1.4A in AASHTO (1996)] and the shaft tip end area. A factor of safety of 2.5 was applied to the ultimate compressive capacities to determine the safe capacities presented in the design charts. Representative, effective soil unit weights were assigned to the upper basin-fill sand, gravel and cobble unit.

The design chart in Figure 5 presents the recommended safe downward capacities of various diameters of straight, drilled, cast-in-place concrete piers for the south bridge abutment. The weight of the shaft (below lowest adjacent finished grade) is included in the calculations. A minimum shaft diameter of 4.0 feet was recommended for all shafts, considering the potential for encountering cobbles and boulders at the foundation locations. The south abutment foundations will consist of two 6-foot diameter by 23-foot deep drilled shafts.

Utilizing methods presented by Reese and Wright (1977), it was estimated that total settlement of drilled shafts designed and constructed in accordance with the design chart will not exceed 3/4 inch. Settlement should be essentially complete following initial application of live loads.

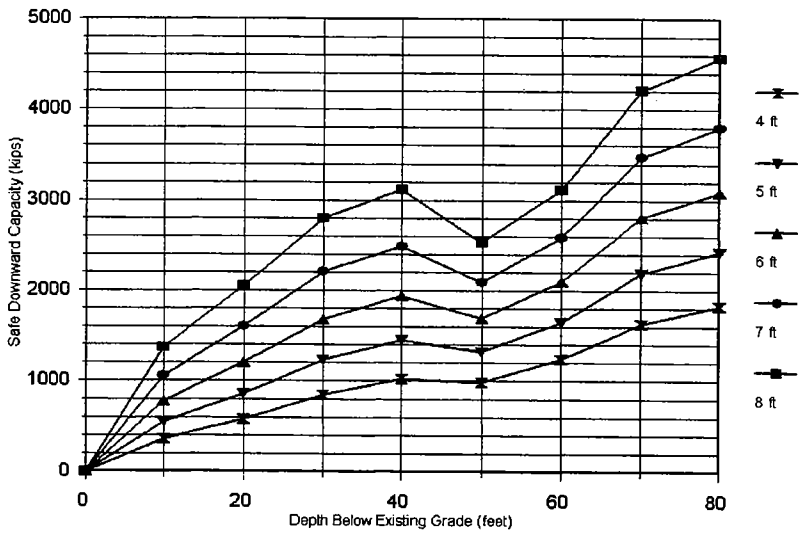
It was anticipated that excavations for drilled shafts at the south abutment may encounter caving and sloughing within the coarse grained, sand, gravel and cobble upper-basin-fill materials. Concrete quantities required in these materials soils likely will be higher than those based on neat excavation volumes. Special stabilization techniques, therefore, may be necessary for shafts within the upper basin-fill.

Bridge Piers & North Abutment

Drilled shaft foundations (rock sockets) were recommended for support of both the piers and north abutment since soft rock conditions are present at relatively shallow depth at these locations. Straight drilled shafts socketed into intact conglomerate of the lower basin-fill unit were deemed to provide suitable downward foundation support and resistance to lateral loads. The axial capacity of rock-socketed drilled shafts was computed considering only the shear resistance between the shaft sides and rock. The method outlined in Section 4.6.5.3 of AASHTO (1996) was used, assuming an unconfined compressive strength of 300 psi for the conglomerate and a safety factor of 2.5. End-bearing of the shaft tip (in accordance with the method) and the side shear contribution from soils overlying the rock were neglected.

The design chart in Figure 6 presents recommended safe downward capacities of straight, drilled, cast-in-place concrete rock sockets as a function of depth of penetration into intact conglomerate. The recommended minimum depth of penetration of the drilled shafts into

FIGURE 5
RECOMMENDED SAFE DOWNWARD CAPACITY OF STRAIGHT DRILLED, CAST-IN-PLACE CONCRETE SHAFTS



competent conglomerate is 2.5 times the diameter of the drilled shaft. The bridge pier and north abutment foundations will consist of two 6-foot diameter rock sockets at each location, with depths of 28 feet at the abutment and 39 and 44 feet at the piers.

Total settlement of rock socket foundations with straight sides designed and constructed in accordance with the design chart are estimated to not exceed 1/2 inch. The settlement primarily will be elastic in nature and will occur during construction of the structure.

CONCLUSIONS

The proposed new U.S. 93 southbound bridge over Box Canyon Wash will be supported on two foundation types due to two different geologic units present at the site. The primary geologic unit underlying a majority of the site is a soft sedimentary rock composed mostly of conglomerate. This rock unit is exposed in most of the walls of Box Canyon Wash and will provide good foundation support for the bridge. Prior to investigation of the site, it was anticipated that this rock type would be encountered at relatively shallow depths at all planned pier and abutment locations. However, the investigation revealed an old buried paleochannel filled with younger, less consolidated alluvium that has less capacity than the soft rock for founding the bridge. As a result, two types of foundation systems will be required to mitigate against differential settlement of the bridge. The foundations on soft rock with the higher capacities will consist of rock sockets, whereas the foundations on the alluvial fan deposits will consist of drilled shafts. The rock socket design relies solely on side resistance whereas the drilled shafts incorporate both side resistance and tip resistance. Since the drilled shaft calculation method incorporates both tip and side resistance in the upper basin fill deposits, the geometry of the drilled shafts and rock sockets are similar.

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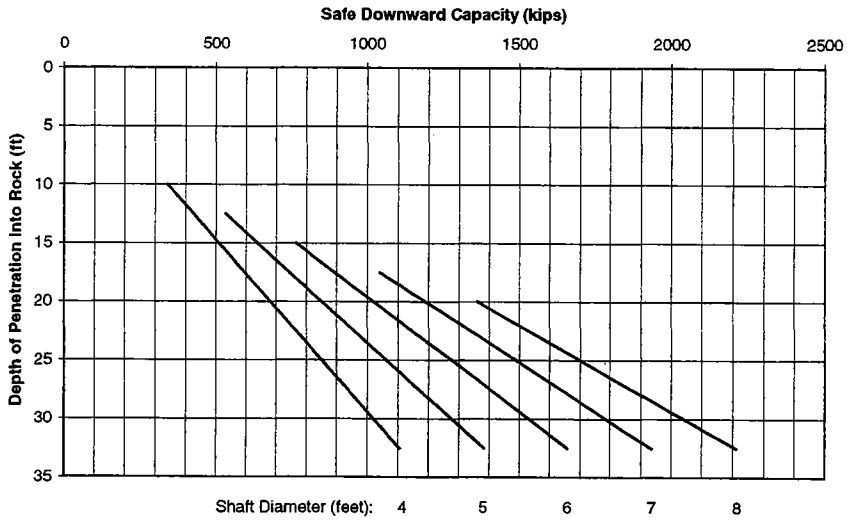
FIGURE 6
RECOMMENDED SAFE DOWNWARD CAPACITY OF
STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE SHAFTS (ROCK SOCKETS)
NORTH ABUTMENT & PIERS



Jet Grouting at Pier 3
Bill Emerson Memorial Bridge
Cape Girardeau, Missouri

John F. Szturo R.G.
HNTB Corporation
1201 Walnut, Suite 700
Kansas City, Missouri 64106

Marcelo Chuaqui, M.Sc.
MC Grouting, Inc
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ABSTRACT

In 1927, the Missouri Department of Transportation constructed a 4,744-foot crossing of the Mississippi River near Cape Girardeau, Missouri. Now, at age 73, this two lane bridge is scheduled for replacement with a new four lane cable-stayed structure.

The new bridge has an overall length of 3,956 feet, and will be of steel/concrete composite construction. The main span unit is comprised of a three-span, 2,086-foot cable-stayed unit with an 1,150-foot navigation span. The Illinois approaches are of typical steel plate girder construction.

This new bridge is located within the New Madrid Seismic Zone, the location of three of the largest seismic events to occur within the interior of the North American tectonic plate, and is a candidate to experience a significant earthquake within its design life.

During the early stages of construction of Pier 3, near the center of the main channel of the Mississippi River, unexpectedly large karstic features were discovered in the founding limestone formation. Work was suspended pending further investigation of the conditions encountered and development of a satisfactory solution which retained the original intent with respect to the design of the bridge.

The following foundation alternatives were considered:

- Deeper Foundation
- Larger Foundation
- Tremie Footing
- Drilled Shafts
- Micropiles
- Ground Treatment

Ground treatment by three fluid jet grouting was considered the most viable alternative, as this method facilitates removal and replacement of low strength, naturally occurring material with an engineered cementitious grout.

Since the founding formation lies some 70 feet below the surface of the Mississippi River, a location which is all but impossible to inspect, emphasis on quality control and verification were key requirements for the work.

In order to verify that the means and methods being used were resulting in adequate ground treatment prior to treating the structurally more critical corners of the foundation, the jet grouting was executed through subsequent phases. These phases started along the centerline of the bridge and gradually moved towards the corners. Verification coring and/or review of all the available data were performed after each phase of the work to determine the effectiveness of the ground treatment.

INTRODUCTION

Missouri Route 74 adjoins Illinois Route 146 on a structure over the Mississippi River at Cape Girardeau, Missouri. This structure, completed in 1928 is a through truss structure which is not only functionally obsolete, but insufficient for anticipated traffic volumes. The Missouri Highway and Transportation Commission has elected to replace this bridge with a new 4-lane

crossing located immediately downstream of the existing bridge and selected HNTB to perform structure type studies, preliminary and final design, and to prepare plans, specifications and estimates for the new bridge.

With a minimum horizontal clearance of 800 feet, the conceptual structure types considered were limited to through truss spans, trussed arch spans, and cable-stayed spans. Pier locations were an integral part of the study and the location of substructure elements as well as the effect on the superstructure span lengths (ultimately a cost consideration) were key to the recommendation to the Department to design and construct a 3-span, symmetrical cable-stayed navigation unit.

At the time of the conceptual studies, there was no subsurface investigations, but HNTB did have the original bridge plans as reference, and estimated the foundation requirements based on the information shown on those plans. Although that information included only the general characteristics of the alluvium and the approximate top of rock elevation, the data was considered sufficient to establish reasonable cost comparisons between conceptual bridge types.

The conceptual studies were completed in October, 1991 and as a result, the Department selected the recommended cable-stayed structure type with spans of 468 ft. - 1150 ft. - 468 ft. (see Figure 1) to be designed as both steel and concrete superstructure alternatives.

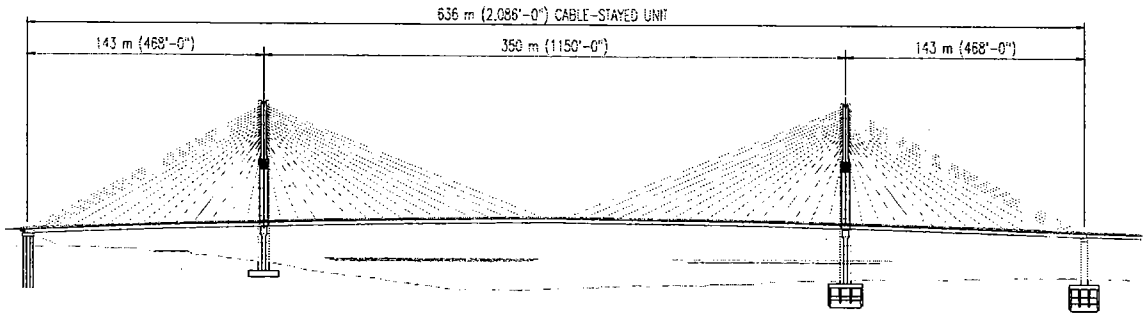


FIGURE 1

Once the conceptual study had been accepted, HNTB and MoDOT negotiated a professional services agreement which included investigative borings at each pier location. These borings were taken in early 1992 and the logs and subsequent laboratory testing were the basis for the *Geotechnical Investigation Report* dated April, 1992.

SITE GEOLOGIC INFORMATION

Physiographic Setting

The project is located on the eastern border of the Ozark Uplift and the Southwestern border of the Illinois Basin. At the bridge site, the Mississippi River is located at the extreme western edge of the alluvial plain. Bedrock outcrops on the west bank, with the City of Cape Girardeau constructed adjacent to the bank. To the east (Illinois), the alluvial plain extends approximately five miles. At the east bank, bedrock is approximately 100 feet below the surface. Just south of the site, the coastal plain deposits of the Mississippi delta begin and extend for hundreds of miles to the Gulf of Mexico.

Seismic Setting

The site is located approximately 50 miles north of New Madrid, Missouri. The New Madrid region has been the most seismically active region of central and eastern North America. The events of the winter of 1811-1812 are well documented, and have been the subject of a significant volume of research over the years. Nuttli's study of the damage and felt effects of these events indicates that the surface wave magnitudes were on the order of M_s 8.5. Other studies have reached similar conclusions regarding the magnitude of that series of events.

Between 1813 and 1990 over 23 earthquakes having magnitudes of 4.5 or greater have been documented in the New Madrid area. Considering that M_s 8 or larger events are anticipated every 550 to 1,200 years, the design earthquake is essentially a repeat of the 1811 and 1812 events.

Site Geology

As the river channel is located at the western edge of the alluvial plain, bedrock outcrops on the west bank. The east bank and approach are located on the flat surface of the alluvial plain immediately inside the levee system. The surficial geology of the site and alluvial plain consists of 75 to 100 feet of mainly fine to coarse sand with lesser amounts of clay and gravel either in mixture or in lenses. The liquefaction potential of these materials was also a design consideration.

The bedrock at the site consists of carbonate rocks of Ordovician Age. The primary units involved at the bridge site are the Plattin and overlying Kimmswick Formations. The Kimmswick outcrops in the bluffs to the west of the bridge site and consists of coarsely crystalline, nearly pure limestone. Heavy solutioning is readily visible in the outcrops of the Kimmswick in the bluffs. The solutioning of the bedrock in this area is not cavernous but appears as widened joints. All the solutioned portion of the rock is filled with various materials. The initial borings taken at Pier 3 in the river found limestone of the underlying Plattin Formation, a very hard and brittle limestone containing various amounts of chert. As evidenced in a nearby quarry the Plattin Formation is several hundred feet thick.

Geologic maps of the area indicated faulting, the majority inferred, as most of the bedrock is obscured, covered by surficial soils, urban development and the alluvial plain. Most of the faults are considered normal, however, some movement is thought to have been strike-slip. The faults are considered inactive, although some are thought to have occurred near the beginning of Holocene time.

SITE CHARACTERIZATION

Original Explorations

At the onset of preliminary design, a geotechnical investigation was undertaken to characterize subsurface conditions at each of the proposed pier locations. Borings were taken at all proposed pier locations according to MoDOT standards. Where borings indicated unusual conditions, additional borings were requested and accomplished. Solutioning, karstic conditions and poor rock conditions were noted in minor amounts at areas near the West Abutment, Pier 4, and occasionally at the east approach structures. At Pier 3, the tower located at mid river, the borings taken indicated limestone of excellent quality with near perfect recovery and RQD.

CONSTRUCTION

The Foundations

The west abutment is founded on drilled shafts with rock sockets in the underlying Plattin Limestone Formation. The west tower pier (Pier 2) for the cable stayed main span is founded on a massive spread footing bearing on limestone bedrock, also of the Plattin Formation. The mid river tower (Pier 3) was planned to be founded on a 67' x 109' dredged caisson with a five foot key excavated from the underlying bedrock. The original contractor planned to drill and blast the rock key prior to constructing the cofferdam and sinking the caisson. Pier 4, is also to be constructed on a smaller dredged caisson with a planned rock key. The remainder of the piers 5 through 14 (now complete) are founded on groups of drilled shafts with rock sockets extending into the underlying bedrock.

West End Bent

The drilled shafts and rock sockets constructed at the West End Bent revealed occasional solution features in the limestone. Solution widened joints were found at the rock sockets for the west end bent, however the solutioning was minor with maximum widths of the widened joints of less than one foot. The sockets were designed in side friction and, as the widened joints were encountered, rock sockets were lengthened accordingly. A maximum allowable design load of 3.5 tons per square foot in side friction was given for the rock sockets. The features decreased in width with depth and sometimes were totally excavated with the addition of length to the rock socket.

Pier 2

Solution widened joints were also encountered in the original borings and during excavation of the massive spread footing for the tower pier 2. The spread footing as designed and constructed at Pier 2 is 60 by 100 feet and 12 feet thick – a nearly 2700 cubic yard continuous pour. The original geotechnical assessment predicted the solutioned condition at this large excavation in this formation would require remediation. Quantities for the remediation, which consisted of excavation of the widened joint filling and replacement with concrete were included in the plans. The design took into account the solutioning and bearing pressures reduced accordingly. A maximum design allowable bearing value of 15 tons per square foot was used.

Pier 2 is located on the west bank of the river and was constructed in the dry. This construction method enabled the visual examination and inspection of the widened joints and quality of the founding bedrock. It also enabled the removal and replacement of joint filling.

As the excavation for Pier 2 obtained proper depth and the joints were exposed, additional rock coring was undertaken in the bottom of the excavation. These cores did not reveal any deteriorated conditions outside of the solutioned joints. The joints were excavated to generally one and one half times their width and filled with concrete.

Pier 3

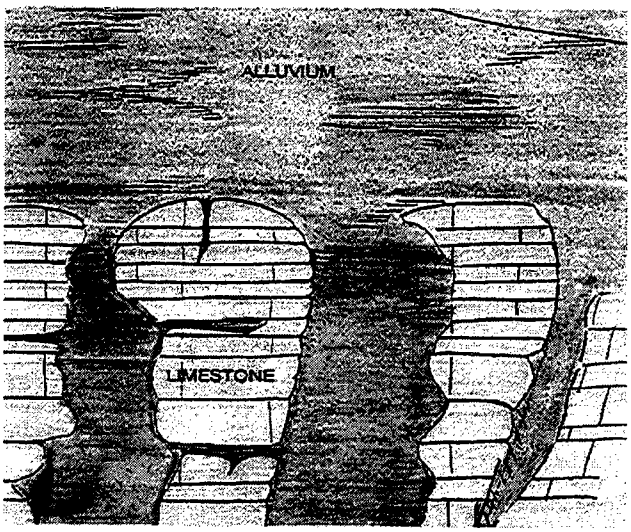
The foundation for pier 3 was designed as a dredged caisson. First, a cofferdam is constructed with steel sheet pile driven to rock in the river. The cofferdam is filled with sand to provide a working surface and provides a more controllable, removable material to facilitate sinking of the caisson. The caisson consists of a structural steel cutting edge (67' by 129' by 12'0 high) prefabricated in three sections. As subsequent lifts of structural concrete are constructed on the cutting edge, material is excavated through the round dredge wells and the caisson lowered to eventually rest on the underlying bedrock. After the caisson reaches rock, the previously blasted keyway would be excavated and the bottom inspected with an underwater camera. Upon successful inspection, the dredge wells are filled with seal concrete. A cap is constructed on the caisson and construction of the tower pier begins.

As the first step of construction, the contractor elected to blast the required five foot keyway into the underlying bedrock at Pier 3. The blasting plan called for a perimeter of closely spaced presplit holes along the perimeter with series of production blasts in the interior. The closely spaced presplit holes were to form the neat line of the keyway.

The holes were begun from a barge anchored on location in the river with a template for the locations secured to the edge. Variations in the bedrock and problems with percussive drilling occurred right away with some locations along the perimeter indicating a lack of competent rock and interconnectivity between the pre-split holes.

MoDOT halted the drilling and blasting for the keyway and a supplementary exploration program of coring the bedrock was undertaken. After an immediate mobilization, an around the clock exploration program consisted of 23 additional borings. The borings were taken from a barge anchored at Pier 3.

Most of the cores (NQ triple tube) extended 50 feet into the rock. Geophysical logs with hole diameter caliper were also utilized. HNTB also contracted with subconsultants Dr. Don Deere Sr. and also Dr. Donald Bruce of ECO Geosystems. Review and analysis by the team confirmed that a large number of features existed throughout the extent of the foundation; however, the occurrence of the features seemed to diminish with depth.



Solutioned Condition of Limestone

Analysis of the explorations indicated areas of solutioning of the limestone. The solutioning was not cavernous and thought to occur along the joints of the limestone. As the roadway centerline runs directly east – west, the primary joint set occurs at 5 , secondary 90 and tertiary set at 45 . Most of the solution features are associated with the nearly north – south joint set. Analysis of the additional cores indicated Pier 3 was located with the interior one half on competent rock and the west and ease quarters on solution widened joints (features). The features are filled, as no voids or open cavities were indicated. The solution filling occurs as mostly fat clay, with various amounts of residual chert left in the matrix. Some of the filling consists of alluvial material scoured and replaced in the rock, or rock of various stages of deterioration.

SOLUTIONS

It was not until after construction of the bridge was underway and the Contractor had begun drilling the closely spaced blast holes into the rock below Pier 3, that the extent of the karstic features was determined.

First, a brainstorming session was held between the geotechnical and structural engineers as well as with MoDOT and FHWA staff to determine the potential solutions to problem. The alternatives considered consisted of:

Increasing the depth of the foundation
Increasing the plan area of the foundation
Replacing the dredged caisson with a tremie footing
Replacing the dredged caisson with drilled shafts
Adding micropiles beneath the dredged caisson
Improvement of the founding limestone with ground treatment

Each of these alternatives was studied in detail and the results were as noted below.

Deeper Foundation

One of the possibilities put forth was to increase the depth of the dredged caisson foundation to a level of rock which indicated a marked reduction in the extent and number of features. Although this solution may have proven a feasible alternative for some structures, this foundation is located some 60 feet below the surface of the Mississippi River, and excavation through an additional 50 feet of rock was estimated to cost in excess of $6 million. Additionally, each foot of depth would increase the moment arm from lateral earthquake forces from the superstructure, resulting in a larger plan dimension, further increasing the cost.

Larger Foundation

Another consideration put forward was for a larger foundation. However, although a larger foundation can reduce the bearing pressures on the rock, founding the bridge at elevation 250 feet does nothing to provide stability within the rock mass itself. Thus, this alternative was rejected because determining an allowable bearing pressure to ensure stability within the rock mass is next to impossible.

Tremie Footing

A tremie footing alternative was proposed by the Contractor as an alternative to the dredged caisson foundation. Although this alternative did nothing to provide remediation of the rock mass, it did provide for a foundation which would not require nearly as much fabrication labor or planning as the dredged caisson. In this respect, a tremie foundation could be considered superior to a dredged caisson; but it falls far short as a structural component for supporting the bridge during the design seismic event. In order to be effective from a bearing perspective, a tremie footing would be very large in diameter and contain more than 12,000 cubic yards of concrete. Placed in the comfort of an excavation on dry land this would be a formidable foundation; as a tremie foundation in the middle of the Mississippi River, the task was also considered next to impossible. Thus the potential of a tremie footing was discarded both, on its constructability and the fact that it did nothing to improve the rock mass.

Drilled Shafts

Drilled shafts, used either as a substitute for – or in combination with – the dredged caisson foundation were the most promising structural alternatives considered. By using drilled shafts, vertical and overturning loads from the structure could be transferred to a depth sufficient to successfully carry the loads; but these alternatives fell short as well. In order to meet the structural requirements, large diameter shafts were required, but the number and spacing were such that the plan area of the

footing would increase by a factor of four. Additionally, there is some doubt as to whether the corner shafts would actually see the computed loads, and whether the shafts could resist the large earthquake induced shear forces.

Micropiles

Micropiles were considered only briefly. Micropiles at this location were considered to have insufficient shear capacity for seismic loads.

Ground Treatment

Ground treatment was considered superior to any of the structural modifications because the treatment is designed to focus on the problem, to treat the karst so that the limestone bedrock will act as anticipated during the design earthquake, and to minimize (or even eliminate) changes in the structural behavior. Since the condition to be treated is karstic limestone, some methods of soil improvement are totally inappropriate. So the design team's considerations centered around a grouting solution. The three grouting methods traditionally used for ground treatment are compaction grouting, permeation grouting, and jet grouting. Both compaction grouting and permeation grouting were considered inappropriate for this site, so various jet grouting techniques were studied.

Ultimately, jet grouting was determined to be the grouting method most closely identified with an appropriate remediation method. It was estimated that the use of three-fluid jet grouting would require on the order of 6000 cubic yards of grout at an estimated cost of $4,000,000.

EVALUATION CRITERIA

With a number of potential solutions available, the focus was then to evaluate the possible solutions, compare those solutions to the design requirements for the bridge, and select a minimum cost alternative which meets all of the requirements of the project.

The first requirement of the final proposed solution was that it meet all of the original design requirements for the bridge. In other words, would the structure perform as expected under the anticipated loading conditions after the completion of the proposed modifications.

Next, consideration was given as to whether the proposed modification would change the response of the bridge to various transient loads such as wind and seismic loads. Any changes in the general stiffness of the substructure could potentially modify the response of the bridge to different load conditions, and therefore, require that the modifications extend into other areas such as the tower legs, cables, or superstructure.

The third consideration was the delay to the general contractor based on the time required to design and to prepare plans and specifications for remedial work to be performed. In short, the sooner the general contractor could be permitted to continue at Pier 3, the more economical the solution.

Finally, the proposed solution had to be acceptable to the owners of the bridge, both the Missouri Department of Transportation and the Federal Highway Administration.

With these selection criteria in mind, it was determined that jet grouting would be the preferred solution. It has the advantage of improving the bearing capacity of the bearing mass without modifying the stiffness of the bridge. Jet grouting could be performed after the completion of the foundation, allowing the Contractor to return to work immediately, with the added advantage of providing a stable surface from which the jet grouting contractor could then work.

The jet grouting could also be accomplished prior to foundation construction, off floating equipment, although it was preferable to grout through the foundation, after the general contractor had erected Pier 3 above Elevation 398, and prior to the erection of the Pier 3 superstructure. By grouting after the fact, the jet grouting specialist would be able to work from a more stable platform (i.e. the top of the caisson) than would be possible with a barge.

The recommendation was based on the lowest cost alternative which meets the requirements of the project design criteria as noted below:

Jet Grouting	$4,000,000
Larger Dredged Caisson	7,000,000
Deeper Dredged Caisson	13,000,000
Drilled Shaft Foundation	21,000,000

Therefore, it was recommended that the general contractor be directed to continue with construction operations at Pier 3 immediately. The work at Pier 3 would remain as shown in the contract plans with the exception of the 5-foot rock key. Therefore, as far as the contractor's proposed caisson sinking plan, there were only two modifications required; the elimination of the rock key and additional seal concrete for dewatering his follower cofferdam.

CONTRACT TERMINATED – SELECTION OF JET GROUTING CONTRACTOR

At this point, for various reasons the construction contract between the general contractor and MoDOT was terminated by consensual agreement. In order to move the progress of the work along, MoDOT decided to procure Nicholson Construction Company to remediate the foundation material while reletting the general contract.

DETAILS OF THE DESIGN REQUIREMENT

As was previously stated, it was and is considered imperative that the solution for the conditions encountered at Pier 3 do nothing to adversely affect the behavior of the bridge. Therefore, the details of the grout design are of utmost importance. First, we must recognize that the founding formation lies some 70 feet below the surface of the Mississippi River; a location which is all but impossible to inspect. This was the overriding determinant to the emphasis on verification. The rock surface and any grout which may be exposed to direct bearing stress from the bridge must be fully capable of resisting that stress without failure. The analysis indicated a peak bearing pressure of 29 tons per square foot, so by providing a 25 percent increase to that, grout strength was specified to be a minimum of 500 psi. Furthermore, it was desired to ensure that the karst features be sufficiently removed and replaced with grout to a depth that would provide the confidence that the performance of the structure was not at risk. This was determined to be approximately Elevation 200 ft (50 feet below the top of rock) as evidenced by a statistically marked reduction in features with depth. Combined with the reduction in the capacity demand of the formation with depth, Elevation 200 ft became the lower boundary of the treatment.

Selection of Three Fluid Grouting

Jet grouting involves treating soil in situ by using high pressure jets to erode, remove and either replace or mix the soil with grout. The treated mass is created by injecting fluid at high pressure through nozzles at the end of a monitor. The monitor is attached to specialized drill rods, and a drill rig is used to rotated and move the monitor in a controlled fashion.

There are basically three methods of jet grouting:

One fluid system (F1) – Grout is the fluid used. It simultaneously erodes and mixes with/replaces the soil;

Two fluid systems (F2) – Grout and air are the two fluids used. An envelope of compressed air surrounds the grout jet. The compressed air allows the grout jet to travel further permitting larger diameter columns to be constructed;

Three fluid systems (F3) – Grout, water and air are the three fluids. A water jet, within a compressed air envelope, is used to erode and remove the soil, while grout is injected to replace the soil. The grout can either be jetted or simply injected through the drill bit, depending on the type of system being used. Three fluid jet grouting is the most complicated of the three methods, but generally provides the highest degree of soil replacement and the largest diameter columns.

Since three fluid jet grouting provides a higher degree of soil replacement and can produce larger diameter columns than single and two fluid jet grouting, it was selected. A higher degree of clay replacement was required to meet the 500 psi minimum unconfined compressive strength criteria. Larger diameter columns would help ensure that the Karst features were sufficiently removed and replaced with grout. Since drilling conditions were difficult reducing the number of holes required for ground treatment was also advantageous.

Performance Type Specification

The specialty grouting contractors tend to have unique experience and expertise. Many typically use specialized variations of the generalized description of three fluid jet grouting given above. In order to allow each of the contractors to utilize their specific systems and expertise, the specification was prepared as a performance specification with certain performance requirements and criteria to be met by the contractor.

Table 1 outlines which criteria were established by the specification and which were the responsibility of contractor.

Table 1: Summary of specification requirements and responsibilities.

Established in Specification	Contractor's Selection/Responsibility
Extent and Depth of Treatment (Between Elevation 200' and 250')	Study All Available Data
Contractors Experience Levels	Provide Staff, Equipment and Instrumentation
Three Fluid Jet Grouting to be Used	Meet listed Performance Criteria
Locations of Test Area	Submittals for Means and Methods
Standards for Materials	Quality Control Testing
Grout Mix to be Stable (low bleed)	Design of Methods and Grout Mix
Strength Acceptance Criteria (500 psi)	Selection of Jetting Parameters
Minimum Acceptable Length of Lateral Treatment (3')	Methods for Spoils Control and Measurement
Need for Acceptable Test Areas Prior to Production Grouting	Execution Scheduling and Sequencing
Selection of Coring Locations	Coring
Core Acceptance Criteria	Grouting Test Area
Hole Location for Primary Holes	Wet Grab Sampling
Need for Allowing Spoils Venting	Automatic Parameter Recorder
	Submit Quality Control and Grouting Records

Column Confirmation Testing

The specification required a column confirmation test (CCT) be performed in the structurally less critical zone, along the centerline of the bridge, to confirm that the contractors selected means and methods were meeting the performance and acceptance criteria established in the specification.

Hole Layout

Since the location and extent of features in a Karstic limestone are unpredictable, a systematic approach to treatment location was selected. Primary holes would be drilled and grouted in a regular grid. Where zones with large features were encountered, secondary holes would be drilled and grouted. In zones where large features were again encounter, in the secondary holes, tertiary columns would be drilled and grouted.

COLUMN CONFIRMATION TESTING AND LIMITED PRODUCTION GROUTING

The contractor attempted three column confirmation tests that did not adequately demonstrate the ability to meet the performance criteria to the satisfaction of the resident engineer. A fourth column confirmation test was executed on a force account basis with procedures and parameters established by MoDOT/ECO Geosystems/HNTB with input from the contractor. Although the verification data from CCT-4 did not conclusively demonstrate that all the performance criteria had been met, the testing data and site observations were sufficiently promising that limited production grouting based on modified CCT-4 procedures was initiated. Limited production was confined to the areas adjacent to the CCT testing zone but still away from the most critical areas, the corners of the footing.

The original criterion of 500 psi (ucs) was based on an analysis of peak bearing pressure at elevation 250'. It was noted that the bearing pressure is dissipated rapidly with depth. In order to focus the treatment were it was needed, after further analysis, the strength criterion below elevation 240' was relaxed to 200 psi for limited production grouting.

Verification coring followed closely behind limited production grouting and, after adequate ground treatment was confirmed by the coring, the contractor was allowed to start full production grouting.

MEANS AND METHODS EMPLOYED TO EXECUTE THE GROUTING

The means and methods were adjusted after each CCT. The means and methods described in this paper will be those used for limited production and production grouting.

The drilling and grouting operations were conducted from the existing platform over the cofferdam. The elevations were as follows:

top of the platform approximately at elevation 350';
top of sand island approximately at elevation 320';
top of rock approximately at 250'; and
lower boundary of treatment at elevation 200'.

Barges were used for the other support operations, including cement storage, grout batching, grout pumping, and spoils containment.

Drilling Pilot Hole and PVC Pipe Installation

The drill rig used to drill the holes and install the PVC pipe was a Casagrande C12. A 12-¾" casing was installed to top of rock or elevation 240'. The drilling method was rotary (i.e. an open ended casing with a cutting shoe and water flush). A down-the-hole hammer was then lowered to the bottom of the casing and a 9" hole was drilled through the formation. The drilling was carefully logged to identify the location of features. The hammer was then removed from the hole and an 8" ID PVC pipe was installed to top of rock or the bottom of the casing. Once the PVC pipe was installed, the casing was removed.

The purpose of the PVC pipe was to provide a pathway for the spoils from the jet grouting to vent. If top of rock was not encountered prior to elevation 240' or the PVC did not reach top of rock, a short zone below the bottom of the PVC was jetted prior to full depth treatment. This was done to ensure that sand would not collapse into the hole and prevent spoils venting during grouting.

Redrill With Monitor

The hole was then redrilled with the jet grout rig, which was equipped with specialized three fluid jet grouting rods and a monitor. The upper nozzle (air/water) was spaced approximately three feet above the lower nozzle (grout). Once the hole was redrilled to full depth the jet grouting operations commenced. The monitor was then rotated and lifted at controlled rates.

Jetting Parameters

The jet grouting parameters consisted of:

Air Pressure – 10 to 15 bar;
Water Pressure – 350 to 450 bar;
Water Flow – 70 to 80 l/min;
Grout Pressure - 350 to 450 bar;
Grout Flow – 310 to 375 l/min;
Lift Rates
    Limestone - 5 seconds per 4 cm lift;
    Feature between elevations 200' to 240' - 35 seconds per 4 cm lift;
    Feature elevations 240' to 250'- 48 seconds per 4 cm lift in feature.

These parameters result in a specific energy of 250 MJ/m below elevation 240' and 330 MJ/m above elevation 240'. The volume of grout injected at these rates correspond theoretically to an 8' diameter column at the 35 second per 4 cm lift rate and a 9' diameter column at 48 second per 4 cm lift rate.

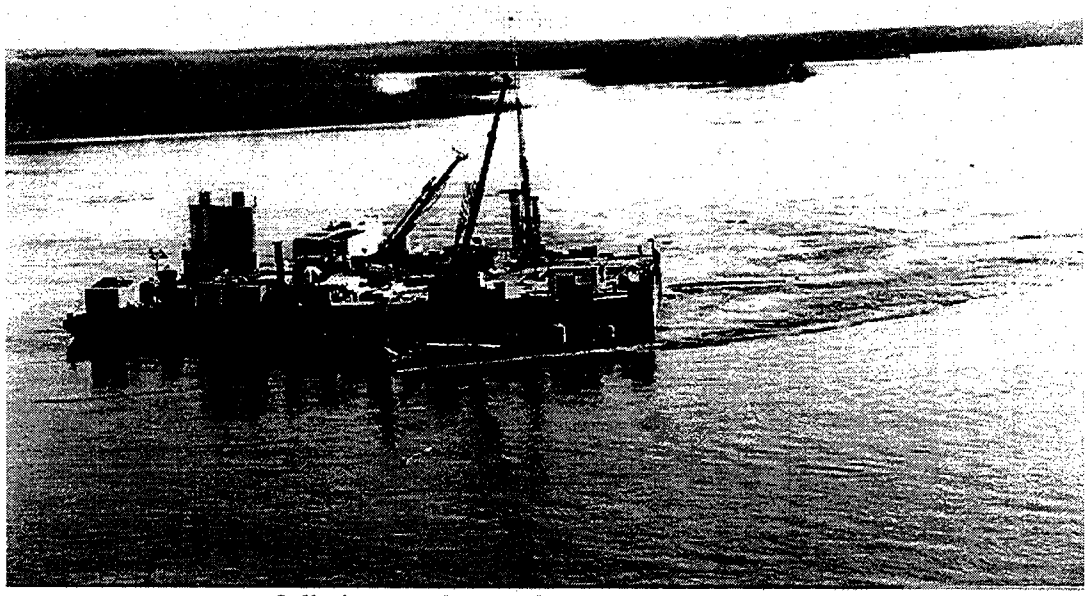
Jetting Equipment

The jet grouting equipment consisted of:

a Casagrande model C8 track drill;
a Rodio model IM20 batching/mixing system;
a GeoAstra Triplex Pump (water pump);
a Tecniwell TW400 Triplex Pump (grout pump);
two 900 cfm, 350 psi compressors;
a bentonite hydrating system;
three fluid jet grout rods;
several Jean Lutz CL-88 parameter recorders.

Grout Mix and Clay Dispersant

The grout had a water:cement ratio of 0.8 by weight, with super-plasticizer at 1.25% by weight of cement and hydrated bentonite at 3% by weight of water. During grouting, a clay dispersant (MBT: PT-1158) was used at 0.75% by weight of cutting water to enhance the removal of clays and venting of spoils from the formation.



Cofferdam – Jet Grouting Operation Pier 3, Mississippi River

VERIFICATION METHODS AND EMPHASIS ON QA/QC

As was noted earlier, the verification of successful grouting was of the highest priority. To ensure the satisfaction of the Owners, a rigorous verification and QA/QC program was initiated at beginning of the program. Only technically qualified, experienced contractors were invited to submit bids for the project.

Prior to executing any work in the structurally critical zones, the contractor was required to execute a CCT test to demonstrate that his means and methods were producing ground treatment that met the specified requirements.

Quality Control Program

In order to ensure repeatability, a tight quality control program was maintained throughout to document and eliminate variations in the grouting parameters and methods.

For production and limited production grouting, column evaluation sheets were developed to provide a method of communicating the Engineers field observations and comments to the Contractor. A column evaluation sheet was filled out for each column installed and a copy provided to the Contractor when the column was completed.

For each specific process or phase of the column installation, the work was evaluated with respect to being in general conformance with the specification and the approved procedure (based on field observations). In addition, the overall installation of the column was evaluated with respect to being in general conformance with the specification and the approved procedure. The criteria to be met for each process or phase of the work are listed in Table 2.

Table 2: Column Evaluation Sheet Criteria

Category	Requirement being evaluated	Test
Drilling of Pilot Hole	Has 9" hole been drilled to elevation 200'?	Review of drill log.
PVC Pipe Installation	Has the PVC pipe been seated in top of rock (if top of rock is above elevation 240')?	Review of drill log.

Redrill with Monitor	Has the hole been redrilled to bottom of deepest feature?	Review of drill log and Lutz display.
Grout Sheets	Have zones logged as either feature or fractured rock been identified for treatment at the lift rates established in the approved procedure. Has a foot been added to the top and bottom of each feature or fractured rock zone? Does the sheet indicate that the lift rate will be decreased as the water jet reaches the bottom of a feature?	Review of drill log and grout sheet.
Venting of Spoils	Was return flush of spoils steady and continuous, or sporadic to non-existent?	Physical observations.
Dispersant	Was dispersant added to the cutting water at the appropriate dosage?	Pre and post grouting measurement at dispersant tank.
Grout Mix	Has the grout been batched adequately?	Marsh Cone test: less than 90 seconds: Measure specific gravity with Baroid mud balance: Between 1.57 and 1.62.
Lutz Recorder	Is the Lutz recorder displaying the required data?	Physical observation and comparison to gauges.
Parameters	Are the parameters within the submitted ranges? Grout Flow: 342 +/- 33 lpm Grout Pressure: 400 +/- 50 bar Water Flow: 75 +/- 5 lpm Water Pressure: 400 +/- 50 bar Air Pressure: 12.5 +/- 2.5 bar Rotation: 6 +/- 2 rpm Lift Rate: 35 or 48 +/- 3 seconds	Physical observation of gauges, Lutz display and rod lifts.
Overall	Has the column been installed in overall conformance with the procedures and specification?	Review of all top portion of sheet.

An additional component to the quality control testing was a spoils testing program. Spoils samples were taken from the spoils line at regular intervals during grouting and tested to determine cement content based on the heat produced to neutralize the soil-cement sample. The clay content and water content could then be calculated.

Potential Impact of Non-Conformant Work

The distinction between non-conformant versus non-acceptable work became an issue during the execution of the work. The potential impact to quality of each of the non-conformance is listed in Table 3.

Table 3: Potential Impact of Non-Conformant Work

Deficiency	Potential Impact
Grout Rheology	Variable grout mix rheology can result in variable jetting parameters (pressure and flow). High viscosity grout will not jet as efficiently and thus not cut and mix as efficiently. High viscosity grout will also reduce the ability to freely vent spoils.
Lutz recorder	The Lutz recorder not working correctly impacts the ability to analyze the grouting operations. It does not directly impact the construction of the column (unless the wrong parameters are being used and the Lutz does not allow anyone to realize it).
Wet Grab Sampling	If wet grab samples are not obtained some of verification data is no longer available.
Prewash/Grout Sheets	Errors on the grout sheets result in some features not getting the complete cutting time and grouting time. This will directly affect the size of the column created at specific elevations.

Grout Sheets	Have zones logged as either feature or fractured rock been identified for treatment at the lift rates established in the approved procedure. Has a foot been added to the top and bottom of each feature or fractured rock zone? Does the sheet indicate that the lift rate will be decreased as the water jet reaches the bottom of a feature?
PVC standpipe	When the PVC is not installed and seated into rock or is damaged during installation the sand and gravel often collapses in. This results in the need to down stage grout. The zone that is down staged may not be treated as effectively due to collapsing ground (poor spoil venting).
Dispersant	The clay dispersant enhances spoils venting by reducing the viscosity of spoils. If clay dispersant is not used it is harder to maintain spoils return. In addition, the clay dispersant also enhances the cutting ability of the water jets. Therefore if dispersant is not used the cutting ability may be reduced.
Parameters	Changing the jetting parameters will affect the size of column created and the quality of the column created. When the parameters are changed, the approved procedures are no longer being used.
Spoils Venting	If spoils are not permitted to vent freely, pressure builds up in the formation and reduces the effectiveness of the jetting. In addition, clay is not being removed from the formation. The size of the column and strength of column may be compromised. In addition, the spoils which are not being vented may be forced into the sand island.

Of the deficiencies listed in Table 3, it was felt that lack of spoils venting was the most critical. If the clay cannot vent then it is difficult to replace it. Furthermore, pressure builds up in the formation, which reduces the effectiveness of the jetting. The coring from limited production columns verified that columns constructed with adequate venting of spoils produced better ground treatment over columns constructed with poor venting of spoils (discussed in next section). Columns that were grouted with poor spoils venting over large sections of feature were recommended for replacement. In total 10 columns, of the approximately 190 columns installed during production and limited production grouting, were replaced due to poor spoils venting.

Verification Program

As part of the first column confirmation test pre and post grouting seismic surveys were conducted for comparison survey. The seismic testing program was not intended to be part of the acceptance criteria per se. Rather the data were intended to illustrate the overall improvement of the foundation mass (i.e. reduction in distribution of significant low velocity clay filled Karst) as a result of the jet grouting. Originally conceived as being a global demonstration to be conducted over the entire foundation, the seismic testing program was actually used as a local verification of the area treated during CCT-1 only. The work was sub-contracted to NSA Engineering Inc.

The verification data from limited production and production work will be covered in this section. Quantitative verification methods consisted of wet grab sampling, spoils testing, and coring.

Wet Grab Sampling

The contractor was required to obtain samples of the treated soil by wet grab sampling from the freshly jet grouted holes at the locations and depths selected by the Engineer. The Contractor was required to prepare cubes and test these cubes for unconfined compressive strength in accordance with ASTM 109 at 7 and 28 days.

Two different types of wet grab samplers were proposed, but for the most part a cable sampler was used to obtain all but one or two of the samples.

The sampler operated by:

Lowering a weighted cable down to the desired elevation;

Dropping the tube section down the cable;

Then dropping the second weight down the hole.

The two weights act as caps on either end of the tube containing the wet grab sample.

The anticipated sampling frequency was one sample per column during limited production grouting and two columns per week during production grouting.

The specification required the average compressive strength of all treated soil samples tested to be greater than the design strength (500 psi later relaxed below elevation 240' to 200 psi). Samples with less than 70 percent of the design strength would indicate non-conformance.

Of the 56 columns installed during limited production, 42 wet grab samples were retrieved. Compressive strength for wet grab samples was high with only one 28-day strength under 500 psi.

The number of wet grab samples required was reduced to 2/week when the contractor started working in the corners as per the specifications for production work. Of the wet grab samples obtained, the strengths continued to be high, with most 7-day breaks coming in above 500 psi.

Coring

The contractor was required to obtain samples of the treated soil by coring and to test these samples for unconfined compressive strength in accordance with ASTM D2938-86. The specification required the use of triple tube and wireline methods. The specification also required that the coring methods and treated ground conditions to be such that 90 percent recovery be obtained as part of the acceptance criteria for the grouting work.

It was anticipated that 10% of the columns would be cored during production. An estimated four samples of core were to be selected by the Engineer from each hole for testing. The specification required the average compressive strength of all treated soil samples tested to be greater than the design strength (500 psi later relaxed below elevation 240' to 200 psi). Samples with less than 70 percent of the design strength would indicate non-conformance. The core locations were selected by the Engineer.

Limited Production Coring

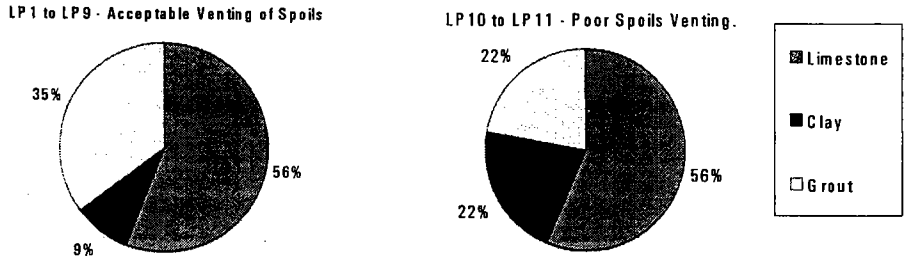
A total of 13 holes were core in the limited production areas. Core locations for LP1 to LP9 were selected to intersect columns, which had been grouted in conformance with the specifications. The goal was to determine if adequate grout treatment was being achieved when the procedures were followed strictly.

The total amount of feature that would have been encountered in core holes 1 through 9 if drilled prior to grouting is 44 %. The post grouting results showed only 9% untreated feature or 4.5' in 50'.

Core locations for LP10, LP11 and LP 13 were selected to intersect columns that had been grouted with some variation to the procedure (non-conformance). The goal was to investigate the effect of these variations to the procedures on the quality of the ground treatment. Core holes LP10 and LP11 were selected to intersect holes with assessed to have poor venting of spoils. Core hole LP12 was selected to intersect a column installed through a smaller diameter PVC pipe. Core hole 13 did not intersect any feature.

The two holes with poor spoils venting (LP10 and LP11) if cored prior to grouting would have encountered 44%. The post grouting results showed 22% untreated feature. Figure 3 contrast the core results obtained from holes assessed to have acceptable and those as assessed as poor spoils venting.

Figure 3: Comparison of Core Results.



Treated soil samples were selected from the core and tested for unconfined compressive strength. The coring process disturbed the treated soil making it difficult to select samples of sufficient length for testing. In addition, handling the samples also damaged some of the samples, which likely had sufficient unconfined compressive strength but sheared easily along the axis perpendicular to the core.

Of the samples that were tested, only one sample did not meet the 500 psi criterion above elevation 240', and all the testable samples met 200 psi criterion below elevation 240'.

The average core recovery over 13 holes was 89.4%. The specification required that the coring methods and treated ground conditions to be such that 90 percent recovery be obtained. The penetration rates and composition of return flush were looked at in areas of core loss to classify the material being penetrated. The reason for core loss was for the most part due to soft untreated material but on some occasions, the core loss was due to chert grinding the core within the core barrel or mislatching during coring. The grouting records for neighboring columns, as well as the distribution and elevation of soft zones were all reviewed prior to releasing the Contractor to perform production grouting in the corner areas.

Production Coring

A total of 10 holes were cored in the production areas. Tables 4 and 5 list the production coring results.

Table 4: Coring Results P 1 to 10.

Hole	Area	Limestone (feet)	Untreated Clay (feet)	Grouted Material (feet)	Total Length (feet)
P1	4	32.6	0	11.1	43.7
P2	4	11.9	0	30	41.9
P3	6	14.6	11.8	20.1	46.5
P4	6	10.7	13.7	19.5	43.9
P5	5	11.4	0	36.9	48.3
P6	5	13	0	36	49
P7	7	3	10.6	20.8	34.4
P7A	7	19.8	9.7	6.7	36.2
P8	7	6.8	10.4	34.7	51.9
P9	7	19.9	1.5	27.4	48.8
P10	6	20.8	5.8	22.9	49.5
Total		164.5	63.5	266.1	494.1
%		33.3%	12.9%	53.9%	

Table 5: Percentage Recovery

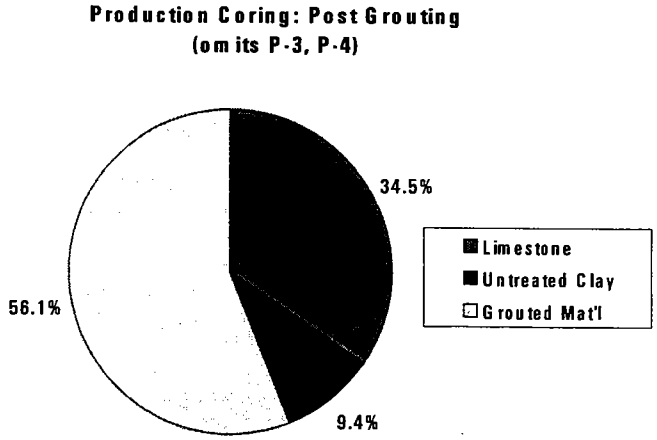
Core Hole	Area	Deviation (degrees)	Recovery (%)
P-1	4 (NE)	Less than 1	96
P-2	4 (NE)	1.0 - SW	95
P-3	6 (SE)	2.0 - N20E	66
P-4	6 (SE)	2.0 – N80W	89
P-5	5 (SW)	1.0 – S	96
P-6	5 (SW)	0.5 – W	95
P-7	7 (NW)	0.5 – S	82
P-7A	7 (NW)	1.75 – NE	
P-8	7 (NW)	0.5 – SW	80
P-9	7 (NW)	0.5 – NE	99
P-10	6 (SE) angled	24 – S10E	90

The production grouting was executed in the four corners of the foundation. These zones, based upon structural considerations, are the most critical zones. Therefore, any zones showing untreated clay were carefully analyzed to determine if remediation was warranted.

Of the holes with less than 90% recovery:

- P3 was remediated by an additional column;
- P4 was remediated by an additional column;
- P7 and P7a were not remediated based upon the elevation and distribution of the soft material.
- P8 was not remeadiated based upon some of the core loss being ascribed to chert grinding some the grouted core, and also upon the elevation and distribution of the soft material the clay.

Figure 4 depicts the overall production coring with the omission of P3 and P4 because these areas were remediated.



All but one of the core samples met the compressive strength criteria of 500 psi above elevation 240' and 200 psi below elevation 240'. Sample P2-A had an unconfined compressive strength of 208 psi at elevation 242'. It is worth noting that the data is increments of 208 psi, which is likely reflective of the accuracy of testing apparatus.

Conclusions

After the CCT work and prior to the start of limited production grouting the treated soil strength criteria was relaxed from 500 psi to 200 psi below elevation 240'. The strength criteria between elevations 250' and 240' remained unchanged. The original criterion of 500 psi (ucs) was based on an analysis of peak bearing pressure at elevation 250'. It was noted that the bearing

pressure is dissipated rapidly with depth and lower strength material would be acceptable below elevation 240'. The strength criteria was relaxed to allow the contractor to focus treatment were it was needed.

All of the verification data indicate the pier 3 foundation was successfully treated. These data included wet grab break data that met the specified strength criteria, core recovery that on average met the 90% recovery criteria, core with less than 10% untreated feature, and core break data that met the specified strength criteria. In addition, the observations made during column installation were also consistent with adequate grout treatment. These included verification of generally consistent parameters and procedures during column installation and verification that spoils were venting. The spoils testing also provided evidence of adequate ground treatment.

The next contract has been let and further construction of bridge is currently underway.

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STEEP NAILED EMBANKMENT TECHNOLOGY: 2 SNET CASE STUDIES

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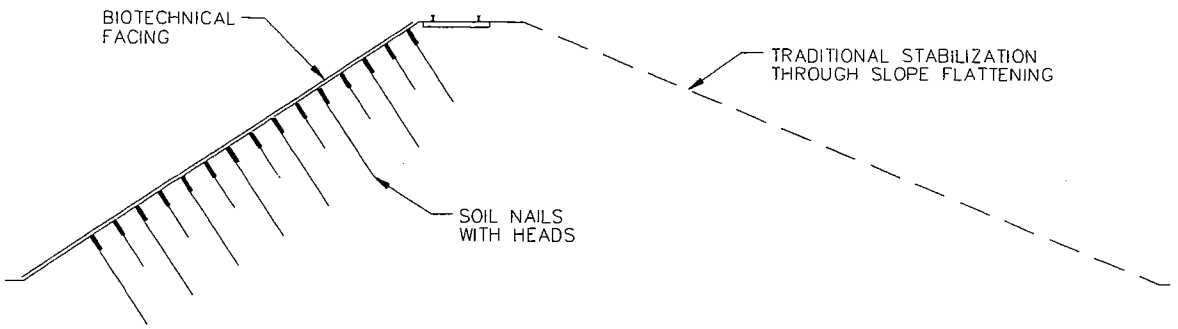
Abstract: This paper describes an innovative and economical method of stabilizing existing embankment slopes where there is insufficient right-of-way for flattening the slopes, where environmental impacts need to be avoided or where there are constraints to equipment and material access. This steep nailed embankment technology (SNET) draws from two well proven technologies, namely soil nailing and biotechnical stabilization.

The concept involves a slope reinforced with steel bars (nails) that are installed without drilling or grouting, together with a biotechnical facing. Design methods developed for the various failure modes, the results of full scale model studies and development of economical installation methods are described. Two case studies are reported on SNET as applied to railway embankments: (a) a 16 m high fill in cohesive materials and (b) an 11 m high fill in cohesionless soils. Performance measurements are presented and discussed for this proprietary technology.

Introduction

Fill embankments are an economical way of achieving grade changes for highways and railroad tracks over low areas. With the advent of geosynthetic reinforcing materials over the past few years, new embankments can be built with slopes much steeper than the traditional (typically 2 horizontal to 1 vertical) fill slopes required to achieve stability (for example, TRB, 1987 and Xanthakos, 1994). Steeper slopes mean less property requirements and less environmental impacts. Unfortunately, these methods are impractical for dealing with existing embankment slopes that need additional stability (often as a result of grade raises, widenings, erosion or heavier traffic loadings), given that the embankment would have to be dismantled to incorporate the geosynthetics. Where sufficient right of way is available, equipment access is simple and materials are available, slope flattening is often the most economical solution. Conventional methods available for stabilizing oversteepened slopes without flattening are very costly, requiring sheet piling, driven or bored piling or retaining walls (for example, Taylor and Voor, 1998)

Figure 1 – Typical solution for over steepened embankment



Given the above, there is a need for a solution that stabilizes oversteepened slopes economically. Such a system should ideally meet the following criteria: (a) a design basis using engineering analysis, (b) installation without affecting traffic on the embankment, (c) installation in areas with difficult access, (d) be of similar cost to conventional slope flattening/widening, (e) have minimum impact on the environment, (f) be aesthetically pleasing, and (g) have the ability to be monitored for performance. A new system has been developed at DST Technologies Inc. (a Canadian firm specializing in geo-solutions) that will meet these criteria. It combines 2 proven technologies, namely soil nailing (traditionally installed with a shotcrete facing on almost vertical cut slopes) and a biotechnical facing (traditionally used to stabilise slopes against surface erosion).

Traditional soil nails consist of steel bars inserted generally perpendicular to the slope, usually by drilling and grouting methods. The surface is covered with steel mesh and shotcrete is then applied to complete the stabilized cut. To meet the criteria for fill slopes, new installation methods had to be developed that avoided the costly drilling and grouting, and permit installation without requiring equipment at the top of the embankment. This involved field installation trials to develop. Furthermore a facing was required to replace the costly shotcrete facing while still stabilizing the shallow soils between the nails. This required large scale testing to establish analytical methods for shallow failure modes of soil slippage around nails.

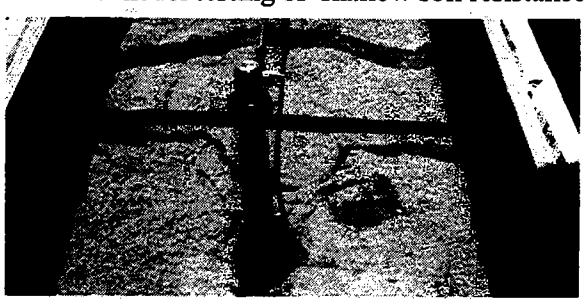
The Technology

The fundamental concept of SNET consists of reinforcing the soil using 'passive' closely spaced bars together with an engineered surficial facing comprising plant root reinforcement. Designs are site specific, and often require geotechnical data to optimize the design. With SNET, the nails are steel bars, typically 25 to 75 mm in diameter with lengths of 2 to 15 m, and are designed with an allowance for corrosion over their design life. Nails are typically spaced at 1 to 2 m, extending about 1 m into stable materials. This system requires small movements within the slope in order for the required resistance to be mobilized. A typical system is illustrated in Figure 1.

Soil nail technology is well developed for vertical slopes for temporary excavations, including design methods (for example, TRB, 1987 and FHWA, 1991). A shotcrete or precast concrete facing deals with shallow slip of near surface soils, while the deeper potential slip failures are stabilized with the lateral resistance of the nails across the slip plane. Fill slopes, on the other hand, are flatter, (less than 45 degrees) and do not require high strength facings such as shotcrete. Furthermore, in many instances a shotcrete facing is not suitable where frost action is a long term consideration.

Full scale model studies at DST's laboratories have shown that a nail head detail is required to stabilize the slope against a failure mode whereby shallow cohesionless soils could flow around the embedded nails (Figure 2). New analytical methods therefore had to be developed to deal with this. As a result, various types of nail heads were investigated for working in association with a shallow root system over the slope surface. The optimum head was found to be a 200 x 200 mm shear plate welded near the top of the nail and oriented parallel to the slope (Figure 3). This was developed to achieve a minimum of 5 kN resistance per nail.

Figure 2 – Full scale model testing of shallow soil resistance around nail



Another way that embankment slopes are different than steep cut slopes is the need to maintain a stable shoulder. This is particularly important for railway embankments where the narrow shoulder provides lateral support to the ties. Shoulder stabilizations have been developed that integrate with SNET, including epoxy stabilization, precast tied and nailed concrete retainers and nailed retainer strips (Figure 4)

Figure 3 – Nail head detail with continuous strip retaining system.

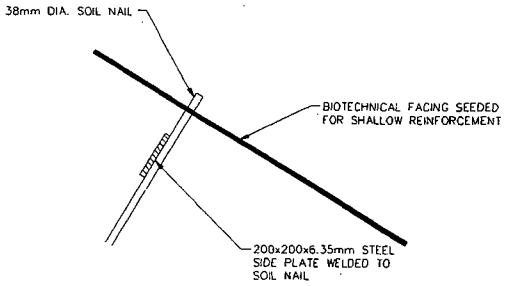
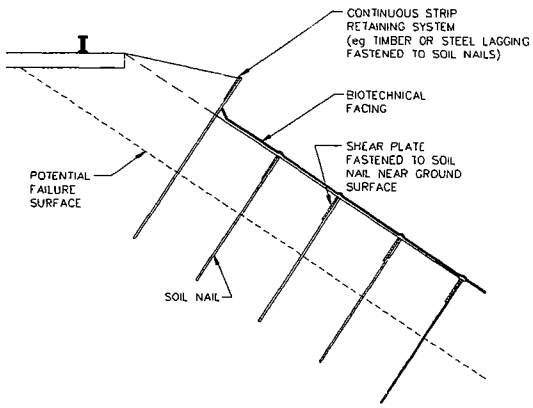


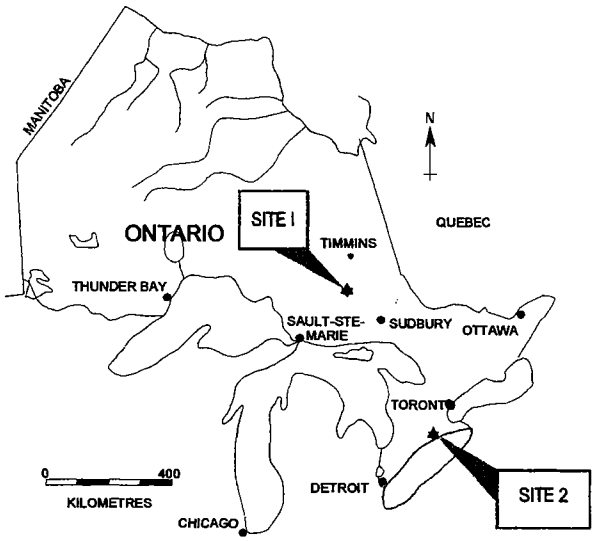
Figure 4 – Detail section of nailed retainer strip.



A biotechnical facing is an integral part of the system. This incorporates a layer of surficial roots that impart an effective cohesion to the soil properties (see, for example, Gray and Solir, 1996). Furthermore, the facing keeps the near surface soils moist, imparting a small additional cohesion which is known to extend deeper than the roots themselves. The facing is engineered with respect to root depth, depending on the site specific conditions.

Developing economical installation methods that do not affect train or highway traffic is an important hurdle in unlocking the power of this technology. To that end, methods were developed for installing the nails with equipment that could access the steep slopes (as steep as 45 degrees) and using percussive, vibratory and rotary methods. This eliminated the traditionally high drilling and grouting costs while providing a number of rapid and economical installation methods that could be selected to fit site specific soil conditions. The ability to monitor the installation resistance of the nail also proved to be a valuable tool to identify localized variations in the stratigraphy, allowing field adjustments during installations to optimize the design.

Figure 5 – Site locations.



The steep nailed embankment technology (SNET) described above is well illustrated with 2 case studies in Canada (Figure 5). These are both railway embankments, one an 11m high embankment of cohesionless soils located in northern Ontario, the other a 16 m high embankment in cohesive soils located in southern Ontario. In both cases, a solution was needed that reduced continuing maintenance costs and as well as the risk of serious impacts from slope failures. Installations were completed in 1999.

At the 2 sites described in this paper, the nails were installed using percussion equipment installed on a specialized excavator. This equipment is self propelled and capable of climbing and sitting on slopes as steep as 45 degrees while driving in the nails. An installation rate of 100 nails per day was not uncommon.

Background and Site Conditions

Site 1

This remote site is located on a Class 1 railroad in northwestern Ontario. Following a culvert failure and repair that involved cutting into the slope of an 11 m high embankment, the track required resurfacing and had problems with insufficient shoulder width. The slopes are as steep as 40 degrees to the horizontal. The subsequent geotechnical investigation concluded that internal embankment (dry sands and non-cohesive silts) soils and foundation soils (stiff clay) were unlikely to present long term problems. However, to achieve stability of the oversteepened slopes, it was decided to stabilize the embankment.

The diagnosis of the problem was as follows:

- Difficulties maintaining shoulders due to lateral train forces and sun-kinks in tight curve with steep slopes.
- Very loose sides (both) as a result of culvert repair work.
- Very loose, steep sides as a result of previous bank widening (through dumping from the top) of shoulders on outside of curve.
- Loose embankment zones due to some soil loss into culvert prior to its repair.
- Low factor of safety against shallow slips (parallel to slope) due to steep incompletely vegetated slope.
- Problems were exhibited over about 50m of embankment, centred on the culvert.

To address the above, an objective was set to improve the stability by 20%. Site constraints for traditional slope flattening proved to be difficult: access to the toe of one side of the embankment was difficult, there were no nearby fill materials, and the signals pole line would have to be relocated. After receiving bids for alternative solutions, DST's steep nailed soil technology was selected in conjunction with an epoxy stabilized shoulder to resolve the problem.

Site 2

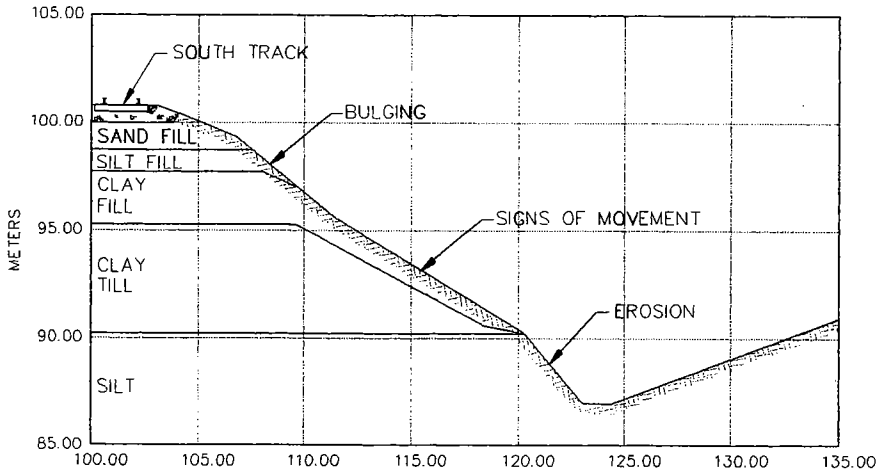
This 30 m long slope instability is located on the south side of a double track railway embankment on a Class 1 railroad near St. Catherines in southern Ontario where double tracks cross a 360m wide creek valley on an embankment up to 20 m in height. In the location of the problem, the embankment is a side-hill fill. The slope is as steep as 43 degrees to the horizontal.

The south track was undergoing continued settlement and lateral shift (several centimetres a week) over about a 30m length of the track, close to the west end portion of the crossing. In the area, the embankment is 12 to 16 m in height. The settlement resulted in continuing maintenance of the south track and a slow order had been issued for all rail traffic.

A geotechnical investigation identified essentially stiff clay materials, with some non-cohesive materials above (Figure 6). The geotechnical report identified potential failure planes, both shallow and deep, and concluded that a) the south embankment slope was in borderline stability and b) that further movements could be expected to occur. Furthermore, the toe of the south embankment had signs of active erosion from runoff along the gully at the toe, and could be contributing to the problem. The preliminary design concept involved piling for stabilizing potential upper level shallow slides and slope flattening for the deep slide case.

To address the above, an objective was set to increase the stability at shallow depth with a 20% degree of improvement combined with control of future erosion at the toe. After assessing costs for the conventional solutions, DST's steep nailed embankment technology was selected. This solution also had the advantage of requiring no environmental approvals (as required for slope flattening), conserving the existing trees on the slope and would have no impact on train traffic since installation equipment could work from the slope itself.

Figure 6 – Subsurface conditions, Site 2



Results of Analysis

In order to design a SNET system, it is necessary to establish design criteria and check the design against all possible failure modes. The following design criteria were established for the 2 sites:

1 installation method to avoid disturbance of a buried fibre optic cable and pole line/signals,
2. installation method to avoid any impact on train traffic,
3. thirty year design life,
4. 20% improvement in safety factor against failure for both shallow transitional slips and deeper wedge or circular slips, and
5. shoulders stabilized without embankment widening to an equivalent standard section for railroad embankment shoulders, also adequate for support of construction traffic.

For the steep nailed embankment technology, the following failure modes need to be analysed using various potential slip locations as well as various nail lengths, diameters and spacings.

1. rupture of the steel nail in shear
2. lateral bearing failure of soil against nail below a potential slip surface(both short rigid and long flexible modes)
3. lateral bearing failure of soil against nail above a slip surface
4. nail pull out
5. shallow soil slip between nails
6. deep seated failure through soils below the nail
7. surface erosion from surface run off

Based on the geotechnical site report (which utilizes information from boreholes through the embankment and from laboratory testing), soil parameters are selected and a subsurface model developed. This is checked with a back analysis to confirm that reasonable safety factors result, and

then applied to find the critical failure modes and locations using routine computer analyses. For the 2 sites in question, the following conditions were established for design purposes.

Site 1

The embankment materials here are dry and granular with sand and gravel on the outside of a silt core. No deep seated sliding planes are expected to have occurred in the past.

The following parameters were selected for analysis:

- sand and gravel: friction angle of 40 degrees (based on laboratory direct shear tests). When moist and unsaturated, add an equivalent cohesion of 1 kPa.
- silt: friction angle of 35 degrees (based on laboratory direct shear tests). When moist and unsaturated, add an equivalent cohesion of 2 kPa.
- groundwater table: at or below the base of embankment

The analyses indicated that in order to achieve the design criteria, 3 m long 38 mm diameter nails with 200 mm x 200 mm shear plate heads were required at a rate of 6 nails per lineal metre of embankment slope. These were concentrated within the upper half of the slope due to the drier nature of the soils here and the increased effects of train vibrations here. The design root depth for the facing was 150 mm, and a seed mix together with a biodegradable coconut fibre mat was specified for areas where existing vegetation did not meet this requirement. A total of 560 nails were installed (Figures 7, 8, 10)

To provide a stable shoulder, an epoxy ballast stabilisation treatment was applied. Epoxy glue was applied over the existing shoulder surface extending outward to the embankment crest with a pressurised spray operation. This product penetrates and cements the loose ballast while allowing it to drain freely and remain in place, reducing potential lateral movements of the ties (Figure 9).

Figure 7 – Site 1 Plan view

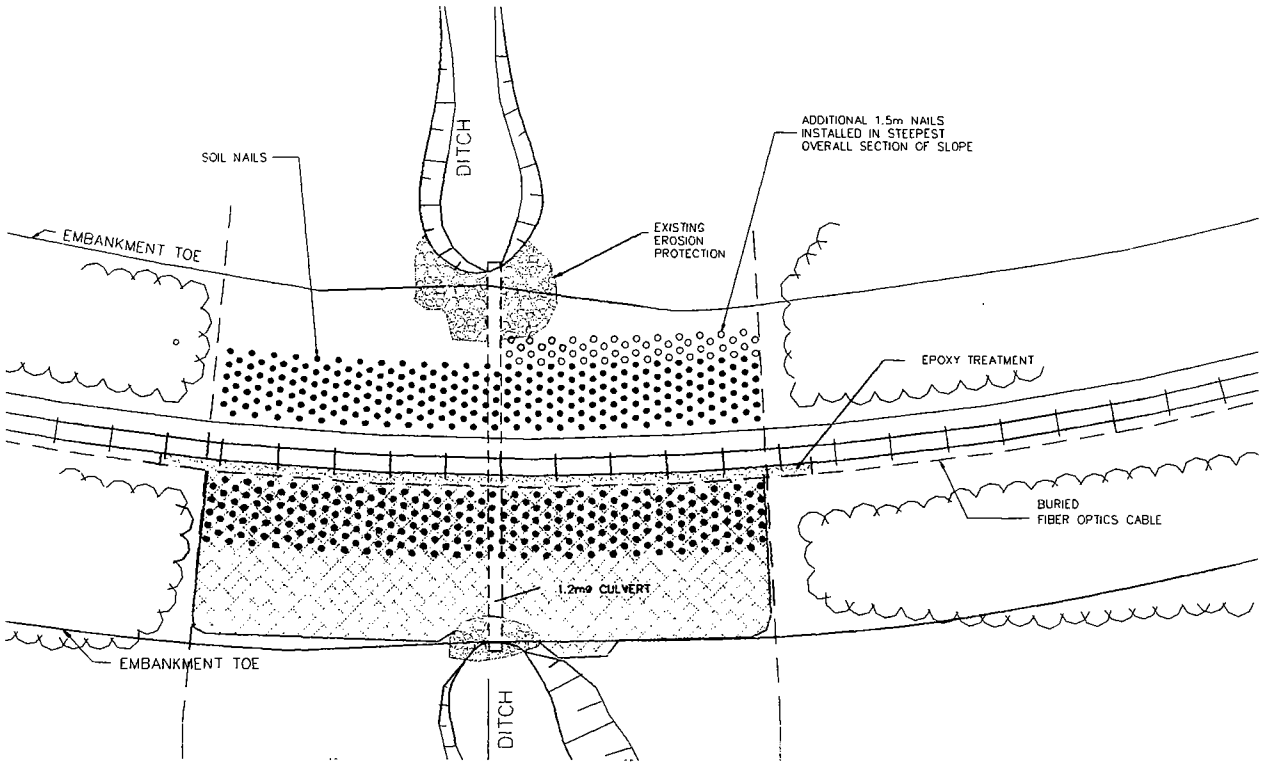


Figure 8 – Nail Installation, Site I

Figure 9 – Epoxy Application

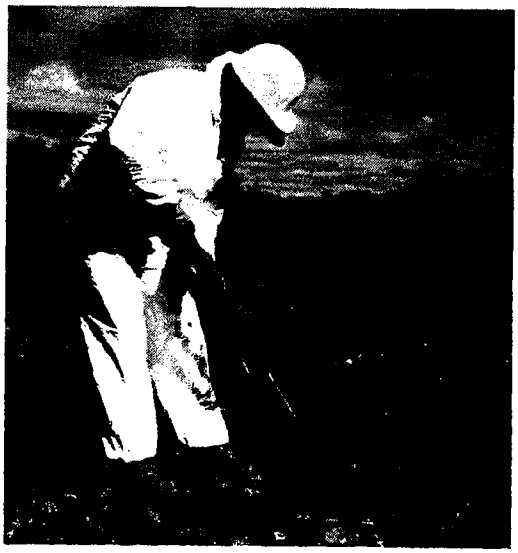
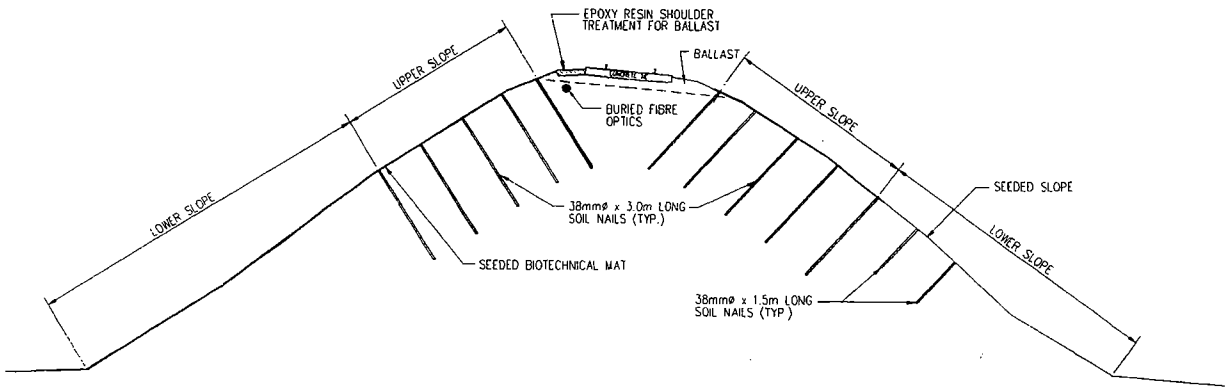


Figure 10 – Site I, Stabilized Embankment Section



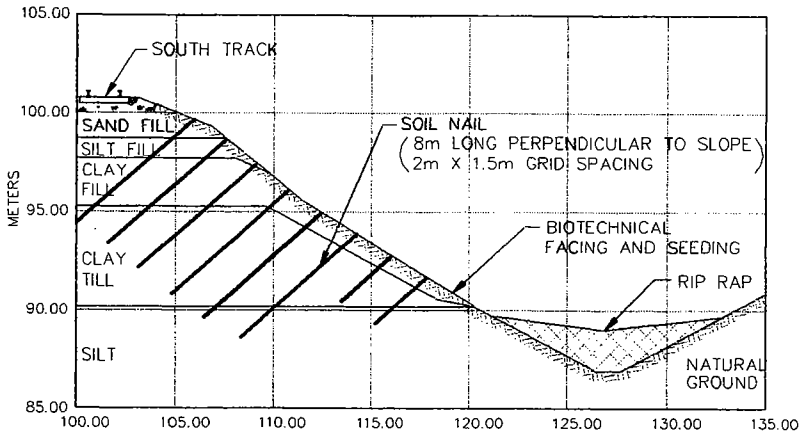
Site 2

Based on the geotechnical report for this site, the embankment materials consist of silt and clay fills to a depth of 4.6 m below base of rail. These overly a stiff clay till which overlies a deep deposit of firm to very stiff clay below 9 m. On the slope a layer of sandy materials also overly the clay fill materials. Given the general topography, this embankment is a side-hill fill on ground that slopes to the southeast.

The following soil parameters were selected:

- Cohesive soils - effective stress parameters: drained angles of internal friction of 27 degrees (fill), 28 deg. (clay) and 35 deg. (till), with a cohesion of nil.
- Cohesive soils - total stress parameters: undrained shear strength of 25 kPa (clay) and 50 kPa (till).
- Cohesionless soils, a drained angle of internal friction of 28 deg. (silt till) to 31 deg. (sand fill).

Figure 11 – Site 2, stabilized embankment section.



Limit equilibrium analyses were carried out for slip surfaces passing through a weak zone within the clay fill at depths ranging from 2.6 to 4.6m below the top of the embankment. This mode of failure is referred to as "shallow slide". As well limit equilibrium analyses were carried out for slip surfaces passing through the firm grey clay layer from 9.2 to 10.7 m below the top of the embankment. This mode of failure is referred to the "deep slide".

The analyses indicated that in order to achieve the design criteria, 2.5 to 8 m long 38 mm diameter nails with 200 mm x 200 mm shear plate heads were required at a rate of 8 nails per lineal metre of embankment slope. A total of 275 nails were installed. Following nail installation, a coconut fibre biotechnical facing blanket was installed in order to promote new vegetation growth and assist in preventing erosion of embankment materials. All embankment faces within the stabilization area were seeded and fertilized (Figure 11).

At Site 2, the gully erosion at the toe of the embankment also needed to be addressed to regain all or most of the eroded toe, while retarding future erosion. This was accomplished by partially infilling the gully with rip rap and regrading to form a ditch within the rip rap.

Figure 12 – Site 2, installation of soil nails east limit of stabilization.

Figure 13 – Completion of embankment stabilization, soil nails and erosion protection installed.

Installation Equipment

Nails were driven into the embankment to specified tolerances with a unique mobile hydraulic walking excavator. The walking mechanism which allows for access up steep embankments consists of two wide base low pressure tires and two extendable out-riggers, all of which articulate and can be positioned or moved independently. The installation equipment is equipped with an articulating hydraulic hammer adapted and modified for rapid driving of nails. Impact from the hammer con be controlled and adjusted depending on the soil conditions and can provide an energy of up to 1400 J (Figure14).

Figure 14 – Installation of soil nails east limit of Site 2 stabilization.

SNET Economics

Options for grade stabilization of these two sites were put out for construction costing for slope flattening/widening solutions versus soil nailing. In both cases SNET costs were similar to those of traditional bank widening and slope flattening, although SNET had the advantage of requiring no environmental approvals and no need to relocate poles and signals lines.

Typical costs, excluding mobilization, are in the range of $15 to $50 (US) per square metre of slope face. At Site 1, a typical slope in granulart soil utilizing 3 m long nails with heads on a 1.2 m grid and including a biotechnical facing, the cost was $23 per square metre of slope treated. For a case of deeper slip planes, such as Site 2 in cohesive materials utilizing nails typically 6 to 8 m long nails with heads on a 1.2 m grid and a biotechnical facing, the cost was $40 per square metre of slope treated.

At both sites, the system was installed under a design-build contract, allowing fast-track construction at minimum risk to the owner.

Performance Monitoring

At both sites, the embankment was monitored for movements during the installation, and this confirmed that there was no measureable impact on the tracks.

After construction, some soil movement must take place to mobilize the added resistance provided by the nails. Accurately predicting the amount of straining required is complicated because of the several possible modes of failure and combinations of these modes, although it should be at a decreasing rate compared to pre-installation performance. Such movements (strains) are expected to be relatively small (less than 100 mm) near the slope surface and will stabilize as the soil nails become 'active'.

At Site 1, while no ongoing monitoring system is in place, railroad operations personnel have reported excellent performance with no ongoing movements or maintenance requirements (in spite of a long history of problems, maintenance and slow orders).

At Site 2, both the tracks and the nail heads were monitored for movements for several months following construction. These indicated movements of up to 50 mm on the slope (that is, the nail heads), which indicates that the nails were building up the resistance to slope movement. The monitoring also indicated essentially no movements for the tracks themselves. Railroad operations personnel reported that track maintenance over this section has been significantly reduced and all slow orders over this embankment have been removed.

Summary and Conclusions

A new engineered system for economical stabilization of existing embankments has been developed utilizing soil nails with a biotechnical facing. The nails incorporate special heads, and complementary techniques have been developed for stabilizing narrow shoulders at the crest.

Steep nailed embankment technology (SNET) has proved to be an economical method of stabilizing embankments that have slopes which are too steep or that are already slipping. SNET is particularly effective where traditional slope flattening is not practical, for example if there is insufficient right-of-way, if environmental impacts need to be avoided or if there are constraints to equipment and material access. In such cases SNET has proven to be a powerful tool that avoids the high costs associated with retaining systems and piling. The system brings together the proven technologies of soil nailing of excavation slopes and biotechnical slope stabilization, made possible with the development of new analytical methods and economical installation techniques that avoid drilling and grouting.

A research and development program as well as two case studies have revealed both advantages and drawbacks. The main drawback to the system is the difficulty with installing nails into fills containing boulders or frozen ground. Furthermore, the system needs to be designed on a site specific basis with the knowledge of geotechnical data.

On the other hand the advantages of SNET are considerable. The system basically requires little construction equipment/materials, is readily adaptable to different sites, is not restricted by difficult access, can be installed very quickly, is cost competitive with conventional bank widening or slope flattening, is much less costly than piling or retaining wall options, requires no digging or drilling, involves minimal ground disturbance and does not restrict traffic at the top of the embankment.

Acknowledgements

DST Technologies Inc. currently has a patent pending on the technology described herein. The authors wish to thank the firm for permission to publish results of their research and development work.

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MITIGATION OF A LANDFILL INTERSECTED BY THE SOUTHERN BELTWAY TRANSPORTATION PROJECT

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ABSTRACT

The Southern Beltway Transportation Project extends from Pennsylvania Route 60 near the Pittsburgh International Airport to the Mon-Fayette Expressway near Finleyville, Pennsylvania. The projects purpose is to provide needed transportation improvements, to support economic development, and to improve east-west mobility in the circumferencial corridor south of the city of Pittsburgh. The surrounding roadways have numerous horizontal and vertical geometric deficiencies. They are not capable of providing the needed vehicular capacity and are inadequate to serve the targeted growth area around the Pittsburgh International Airport. This area is favorable for growth due to the proximity of the airport and large acreage available for development. The proposed alignment will pass through a large percentage of land that was previously mined utilizing deep and strip mining techniques. The focus of this discussion will be the northern terminus of the project at the Pittsburgh International Airport. The terminal loop interchange of the proposed roadway will pass through a former strip mine that was subsequently used as a municipal landfill. The area of concern is a small remnant of the 174-acre Weiss & Lutz landfill. The majority of this landfill was excavated and regraded during construction of the Pittsburgh International Airport midfield terminal. Aerial photography review, a soil gas survey, and geophysical surveys (terrain conductivity and magnetometer) were employed to aid in the waste delineation. Subsequent subsurface drilling and sampling was performed to verify the geophysical findings. Site soils and groundwater were collected and submitted for analytical testing. The results of the environmental sampling program were compared to the Pennsylvania Department of Environmental Protections Clean Fill Standards and the Pennsylvania Land Recycling Program (Act 2) State Health Standards. Cost effective soil and groundwater exposure mitigation strategies were developed. Mitigation strategies included the removal of waste materials from the site, redistribution of waste materials on site, the reuse of on site materials, and groundwater control measures. The objective of the mitigation strategies recommended is to ensure the safety of future motorists, construction personnel, maintenance personnel, and to protect the environment. The site characterization approach, survey and sampling results, interpretations, and mitigation solutions are presented.

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INTRODUCTION

This investigation is one of several environmental and engineering studies being undertaken to develop three Southern Beltway Transportation Projects between PA Route 60 near the Pittsburgh International Airport and the Mon/Fayette Expressway near Finleyville, Pennsylvania (refer to Figure 1). The purpose of the projects in this 48-kilometer (30 mile) program is to provide needed transportation improvements, to support economic development and to improve east-west mobility in the circumferential corridor south of the City of Pittsburgh.

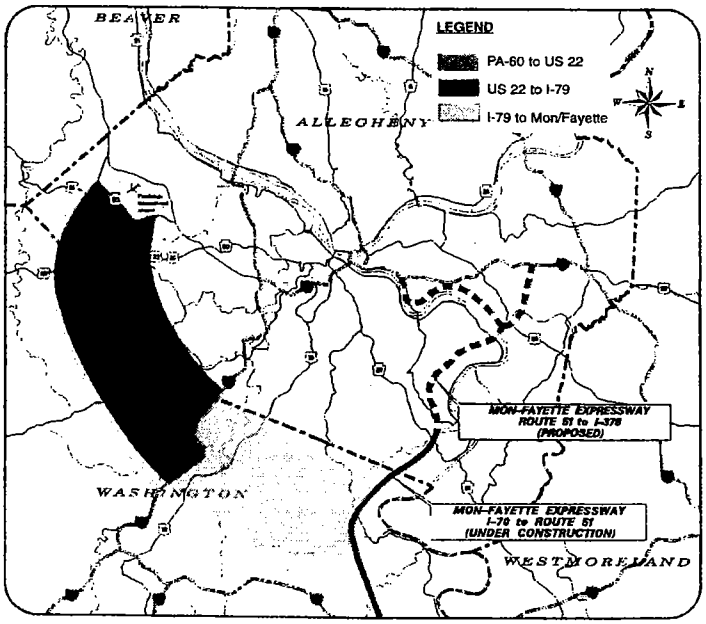

Figure 1. Location of the three Southern Beltway Transportation Project design sections.

The overall program of improvements has been designated as three separate projects for independent development and environmental review, based on: logical project end points; differing project needs; and differing scheduling and funding opportunities.

This investigation was conducted as a part of the Pre-Final Design activities associated with the Tan Alignment for the Findlay Connector of the Southern Beltway Transportation Project. This alignment alternative extends from U.S. Route 22 in Robinson Township, Washington County to the Southern Expressway (PA 60) in Findlay Township, Allegheny County. This document presents the findings and recommendations associated with the environmental media sampling performed for the Preliminary Site Investigation (PSI) conducted for Site 5 - Weiss & Lutz Landfills/PIA Site located within the Tan alignment of the Southern Beltway Transportation Project. Numeric site designations originate from the Initial Site Assessment (ISA) performed by Gannett Fleming in 1996. The locations of the project alignment and Site 5 are presented on Figure 2. Figure 2 also shows the locations of the various alignments studied and the associated potentially contaminated areas within the corridor studies. Figure 3 shows the subject area and the associated potentially contaminated areas within the vicinity of this site. The portion of the ISA Site 5, Weiss Lutz Landfills, studied for this project, comprises approximately the southern-most 22 acres (89,000 m^2) of the 77 acre site 5. The general term "site" will therefore refer to this

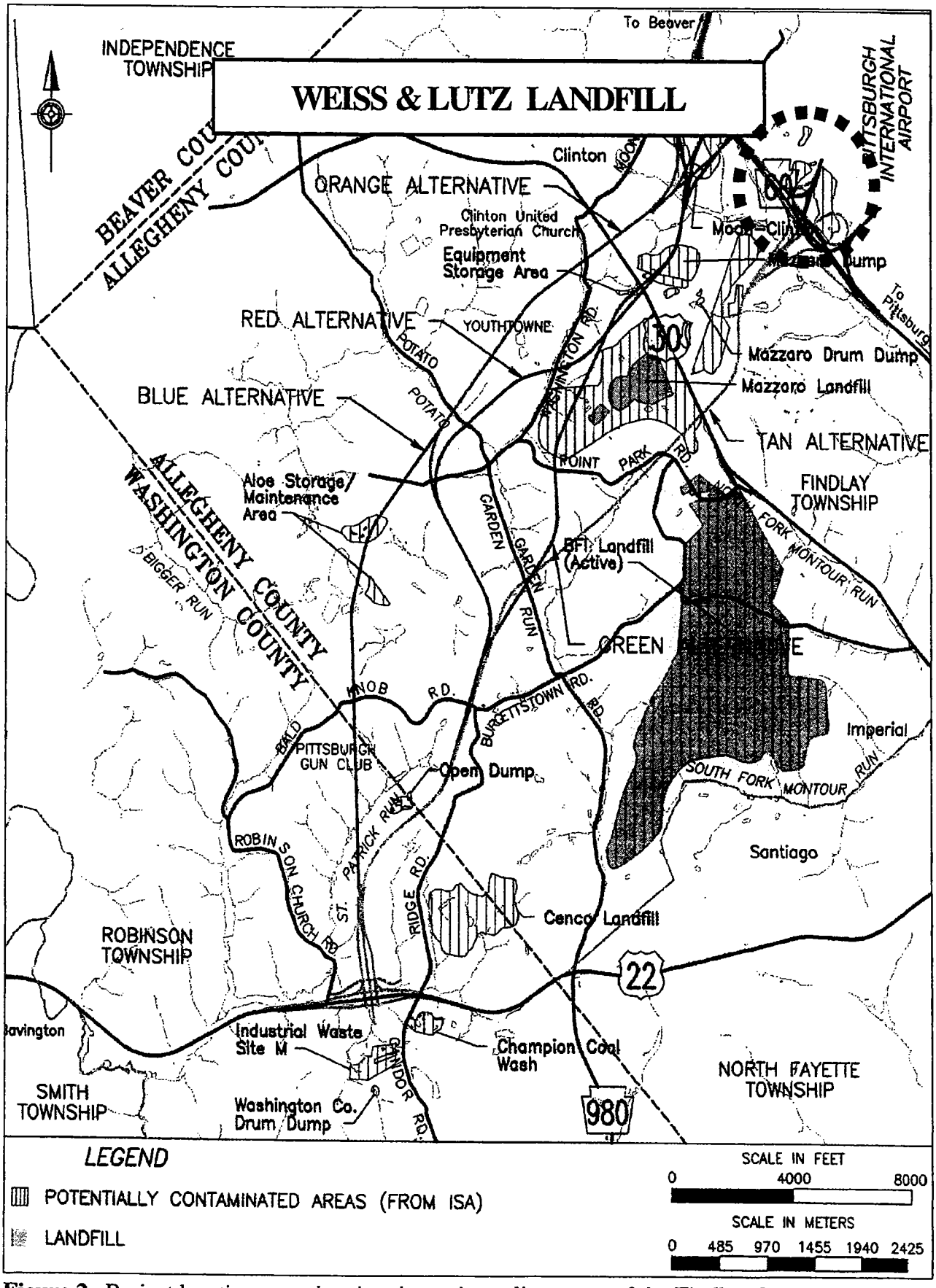

Figure 2. Project location map showing the various alignments of the Findlay Connector, and locations of potentially contaminated areas.

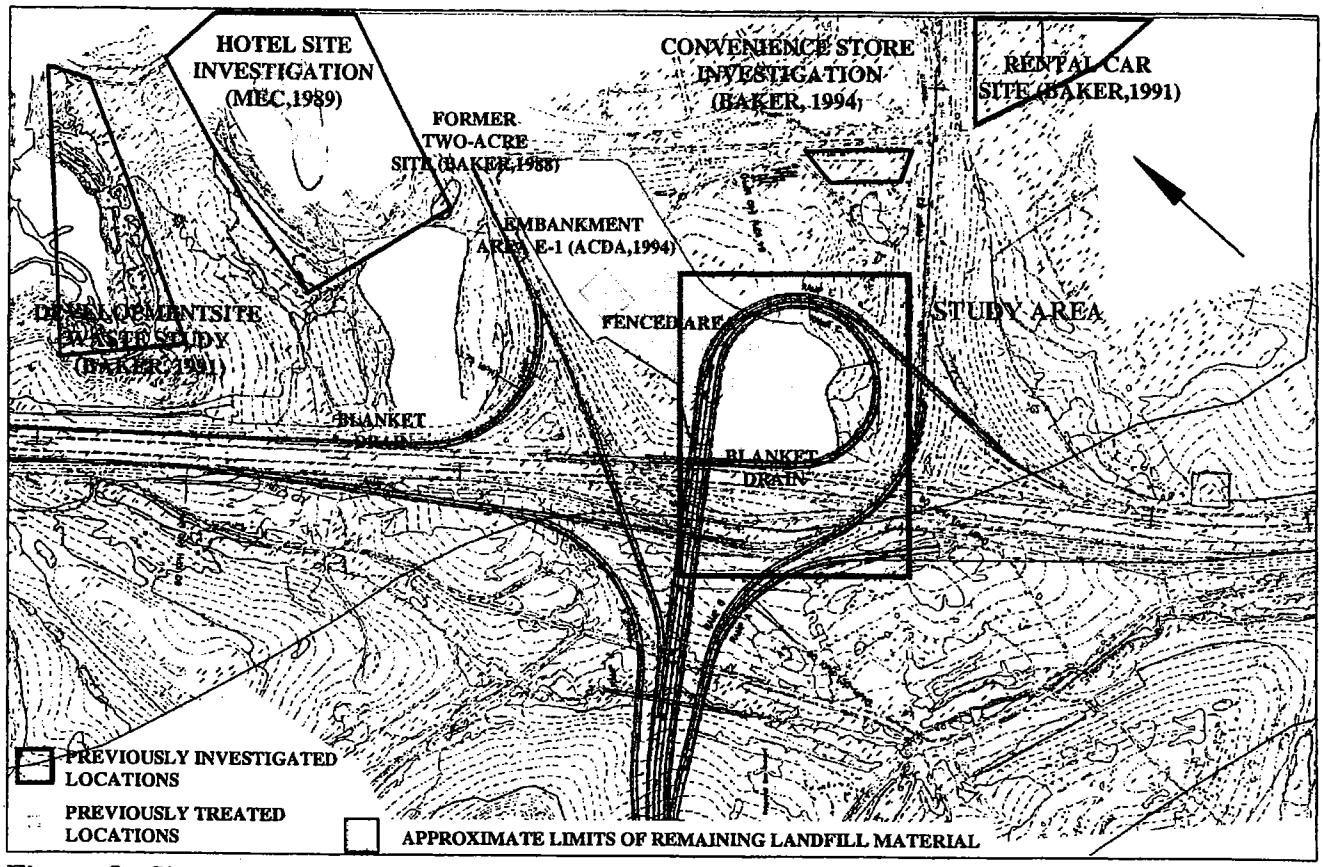

Figure 3. Site map showing the proposed interchange, the remaining portions of the former Weiss & Lutz landfill, locations of former investigations, and previously treated areas.

smaller parcel from this point forward.

SITE HISTORY AND LAND USE

The site has had four primary land uses during its history. These uses include agricultural, strip mining, municipal landfill activities and the airport facility. The site history is summarized in Table 1 below. The anticipated locations of potentially contaminated materials based upon the review of historic records is presented in Figure 8.

SITE GEOLOGY

The site lies within the Pittsburgh Low Plateau section of the Appalachian Plateau physiographic province. This province is characterized by generally horizontal strata subject to gentle folding. Stratigraphically, the site lies at the horizon of the Pennsylvanian age Pittsburgh Coal. This member serves to separate the Casselman Formation below from the Pittsburgh Formation above. Soils are dominantly related to the weathering and erosion of the parent bedrock, typically producing residual, colluvial, and alluvial soils. Borings advanced at the site indicate that the soils at the site are primarily strip mine overburden spoil. The material consists mainly of clayey soils with varying degrees of sand and silt sized particles.

Table 1. Summary of land use for the remaining portion of Site 5-Weiss & Lutz Landfills.

Time Period	Land Use	Source of Information
Prior to 1924	Agricultural	Aerial photographs and deed search
1924 to 1942	Strip Mining	Deed search indicates that several coal companies owned the property during this interval. The Pittsburgh Coal seam was extracted in vicinity of the site by strip mining. This is indicated by 1955 aerial photographs (Figure 4) that show a long linear trench aligned roughly north-south with disturbed ground to both the east and west. The trench appears to have collected water at the southern end, indicating a southerly component of dip of the coal seam.
1942 to 1972	Municipal Landfill	Deed search indicates the property was used as landfill as early as 1946. The 1955 aerial photographs for the now removed northern portions of the landfill support this. It is assumed that these activities ceased when Allegheny County purchased the property in 1972. The 1963 (Figure 5) aerial photographs show that the site is reworked and disturbed compared to the 1955 aerial photographs. Since the coal was previously extracted from this area, it is assumed that this disturbance is caused by landfill excavation and cover activities. In addition, the 1963 photos show several haulage roads and shallow excavations clustered around the trench shown in the 1955 photograph. Based on the 1965 aerial photograph, which shows the area to be reclaimed, waste disposal in the 1955 trench was apparently completed. The 1971(Figure 6) aerial photographs show a disturbed area along the western flank of the formerly stripped area. This appears to be an area of dumping. These areas persist into the 1975 aerial photographs.
1972 to Present		Since 1987, in the vicinity of this project, two major construction projects were completed. They were the construction of the Pittsburgh International Airport and the construction of S.R. 60 (refer to Figure 7)(Baker, 1988;Baker/TSA, 1988; PennDOT, 1989; R&R, 1990;BCM Engineers, 1990; ACDA, 1994;EG&G, 1995). Pursuant to economic development of this area, several environmental investigations took place on or adjacent to the current site (MEC, 1989;Baker,1991; Baker, 1994; Baker 1991) . During the construction of the airport several locations containing wastes were encountered. The wastes encountered were either removed from the site, placed into an on-site landfill, bioremediated at the site and placed in the on-site landfill, or left in place. Construction and development of these facilities has generated most of the existing data regarding the nature and extent of contamination in the vicinity of the site.

Angular rock fragments were encountered throughout the strip mine overburden spoil with occasional boulders. This material overlies the Pittsburgh Coal horizon underclay. This underclay, commonly left in place during strip mine operations, serves as the bedrock interface for most of the site. It should be noted that due to the thickness and low permeability of this unit, it is believed that the underclay is

Figure 4. 1955 Aerial photograph of site, note trench.

Figure 5. 1963 Aerial photograph of site, note reworking of trench.

Figure 6. 1971 aerial photograph of site, note trench is reclaimed and additional waste piles on western flank.

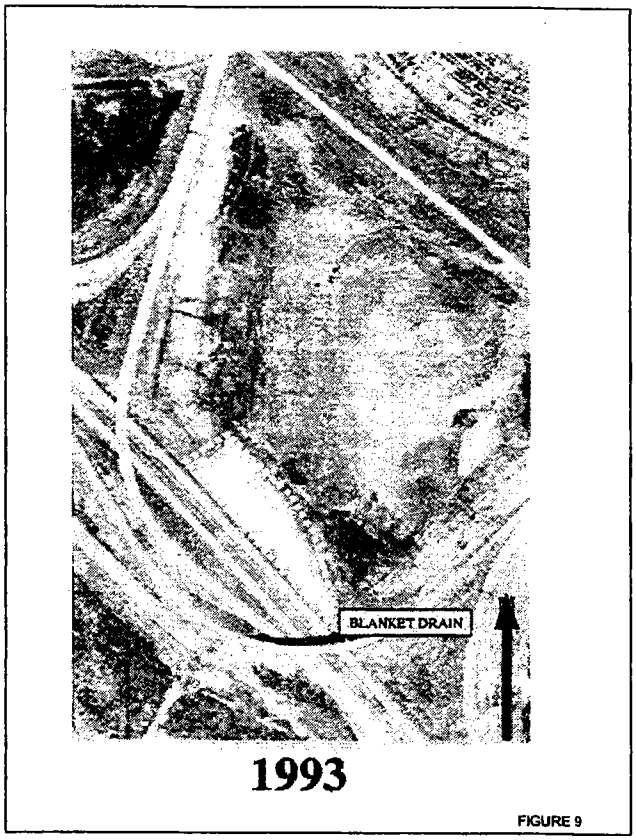

Figure 7. 1993 aerial photograph of site, note construction of SR 60 and airport ,and blanket drain.

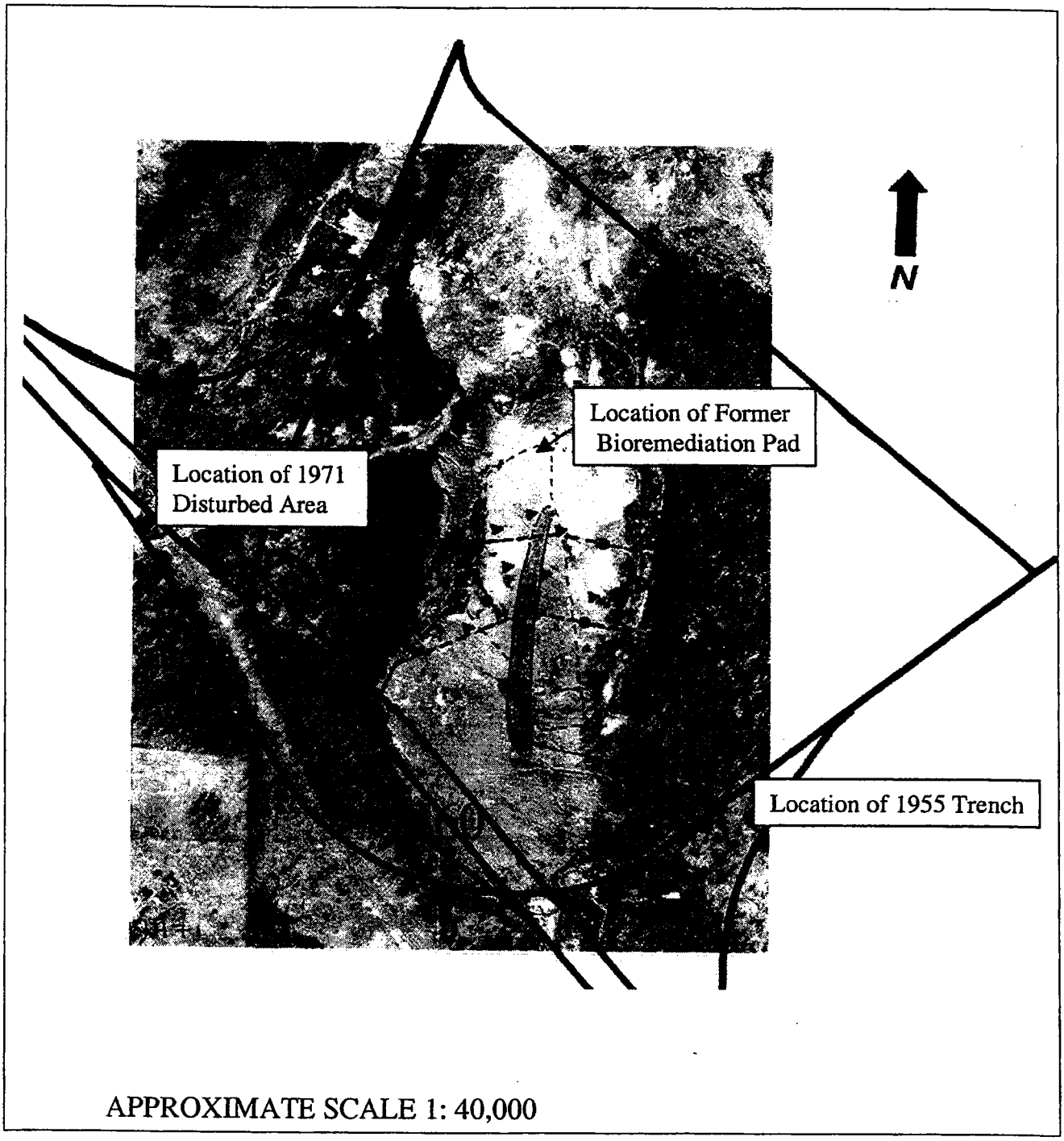

Figure 8. Anticipated potentially contaminated areas based on aerial photograph review. The location of the former bio-remediation pad was obtained from a 1990 aerial photograph not shown above.

serving as a barrier against the downward migration of landfill leachate. Below the underclay, interbedded sandstone, limestone, and shale of the Casselman Formation were encountered consistently across the site. Based on published sources and orthographic projections of boring data, bedrock dips generally to the southwest in the vicinity of the site. This compares favorably to the observations of the aerial photograph review.

Preliminary results indicate that the groundwater migrating through the vadose zone is traveling along the direction of the bedrock dip. In the vicinity of the site, ground water was encountered in the borings at the approximate elevations of El. 352 m and El. 338m. The ground water elevations near El. 338m corresponds closely with the stormwater facilities located at the site; the ground water levels near El. 352m were obtained in the borings drilled within the previously stripped mined Pittsburgh coal horizon. These water level readings indicate that a perched groundwater condition exists within the previously strip mined area. This perched condition of the groundwater may be due to the presumably lower permeability of the underclay. A conceptual model of the site geology is presented in Figure 9.

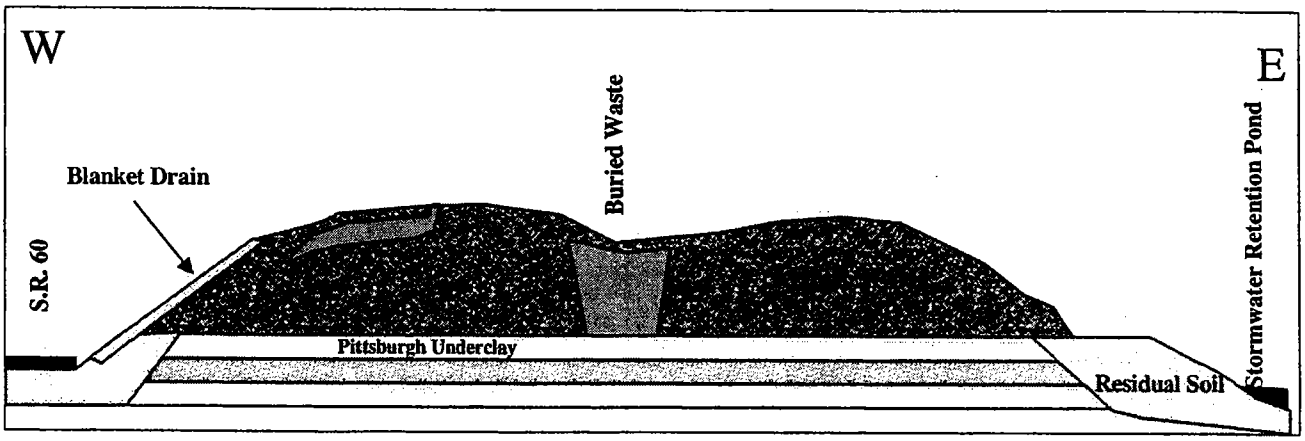

Figure 9. Conceptual model of site geology, based on boring program and aerial photography review

FIELD INVESTIGATION

The field investigations conducted for this site were conducted from May to August 1999. These investigations consisted of a field reconnaissance, a soil gas survey, geophysical survey utilizing electromagnetic and magnetometer techniques, geotechnical borings, auger borings, installation of groundwater monitoring wells, and other environmental media sampling. Each of these investigative elements are discussed below.

Field Reconnaissance

Several visits were made to the site prior to any sampling efforts. The site essentially consists of a promontory surrounded by steep slopes to the east, west, and south. The site is vegetated with high grasses, bound on the east by an eight foot chain link fence, and has one gully originating in the southern portion of the site and draining to the southeast. Traverses across the site also disclosed a swale trending north-south, and an irregular bluff inside the right of way fenceline. The swale

corresponds to the location of the trench observed in aerial photographs, whereas the bluff is in the vicinity of the disturbances noted in 1971 aerial photographs. The swale may have formed due to degradation and subsidence of the waste materials suspected to have been buried there.

Soil Gas Survey

The soil gas survey was used as a screening technique to aid in the delineation of pockets of concentrated organic vapors within the presumed boundaries of the Weiss & Lutz landfill. These pockets are presumably proximal to their source. Sampling was conducted over a regular grid 183 x 275 meters (as shown in Figures 12-14). Grid nodes were initially spaced at 30.48 meters. Intermediate sampling points were obtained in areas of elevated readings. Soil gas survey sampling took place at three feet below ground surface at each sampling point. Samples were obtained by driving a conduit into the ground to the appropriate depth. The driven conduit had openings at the base to allow the infiltration of soil gas into it.

Vapor samples were drawn from the conduit using a Perkin-Elmer Photovac Snapshot Gas Chromatograph (GC). This instrument has an internal pump used to apply a negative pressure to the inside of the conduit. The GC was equipped with a Benzene, Toluene, Ethylene, and Xylene (BTEX) module. After analysis, the instrument returned concentrations for benzene, toluene, and a combined reading for ethylbenzene, m-xylene, and o-xylene. Therefore three concentrations for each grid node were recorded.

Geophysical Surveys

The geophysical survey performed was used as a secondary screening tool for the site. The geophysical survey employed both electromagnetic and magnetic techniques. It was anticipated that the geophysical survey would complement the soil gas survey and aid in the selection of subsequent boring locations. It was anticipated that electromagnetic anomalies would reveal areas of low electromagnetic conductivity, indicating waste material and the magnetic survey would reveal significant magnetic anomalies, which may indicate buried ferrous objects such as drums. This correlation (or lack thereof) helped in determining the nature (metallic versus non-metallic) of the source of the organic vapor pockets. The geophysical surveys were conducted over the same grid area as the soil gas survey.

SUBSURFACE BORING AND SAMPLING PLAN

Geotechnical Borings

Drilling activities for the Preliminary Site Investigation were performed and executed in July and August 1999. There were a total of 11 borings drilled within the suspected contamination zone on the PIA property. These borings were advanced within this area to collect subsurface data required for the geotechnical design and construction of the roadway as well as to collect the necessary samples for analytical testing. Six monitoring wells were installed for the purpose of characterizing the condition of the groundwater and collecting groundwater samples for analytical testing. Split-spoon samples were collected and Standard Penetration Tests (SPT) were conducted continuously through the overburden soils. Split inner barrel rock coring (NQ2) was performed to the completion depths of the borings. Rock recovery and Rock Quality Designation (RQD) Index were recorded. Groundwater levels were measured when encountered and at least 24 hours afterward. The locations of the borings and wells are shown on Figure 10.

Auger Borings

In order to evaluate the magnetic anomalies revealed by the geophysical survey, as well as further delineate solid waste deposits, 13 auger borings were advanced across the site (Figure 10). Soil samples were collected every 1.52 meters in split spoon samplers to a maximum depth of 5 meters in these borings. Solid wastes were encountered in AB-1, AB-4, AB-10, AB-12, and AB-13, coincident with the disposal areas identified in the 1955 and 1971 aerial photos.

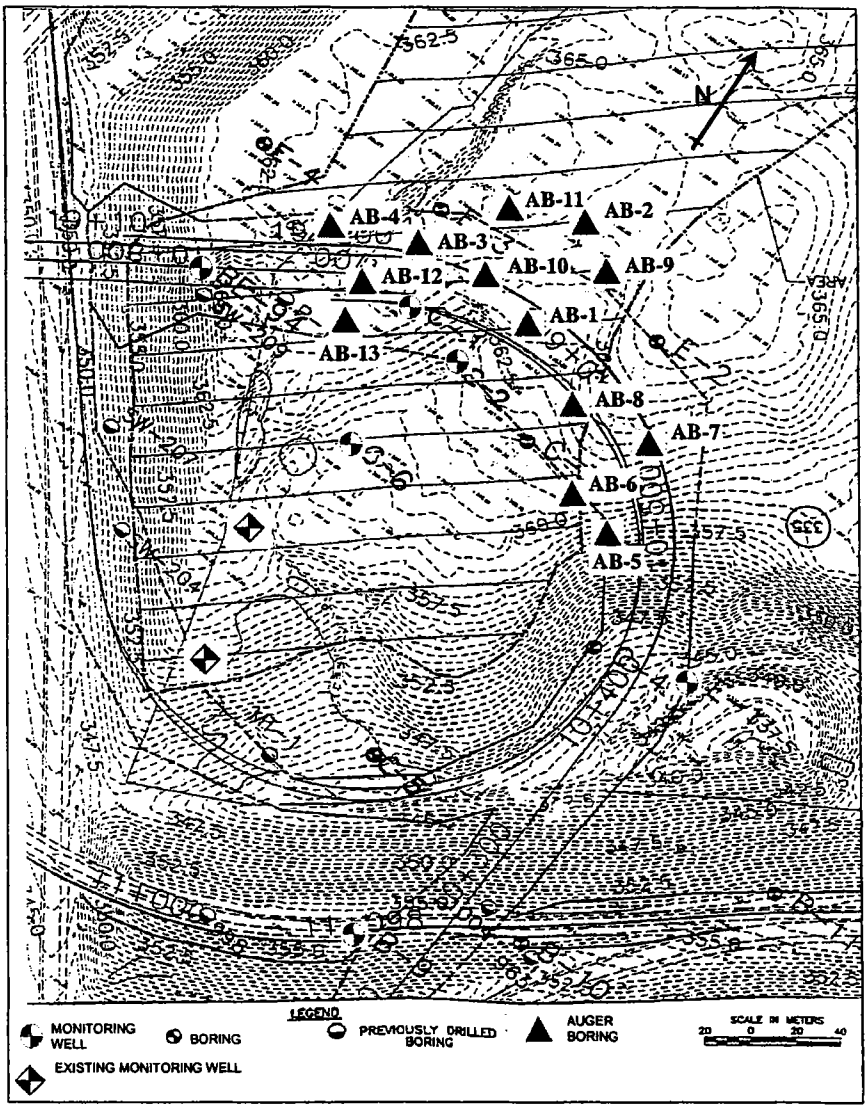

Figure 10. Locations of geotechnical and auger borings, and existing and new monitoring well installations

Representative subsurface soil samples collected in the split spoon sampler from auger borings AB-1, AB-2, AB-3, AB-4, AB-5, AB-6, AB-7, AB-8, and AB-13 and were subjected to analytical testing of EPA priority pollutant screening test series. The locations where environmental sampling was conducted are indicated on Figure11.

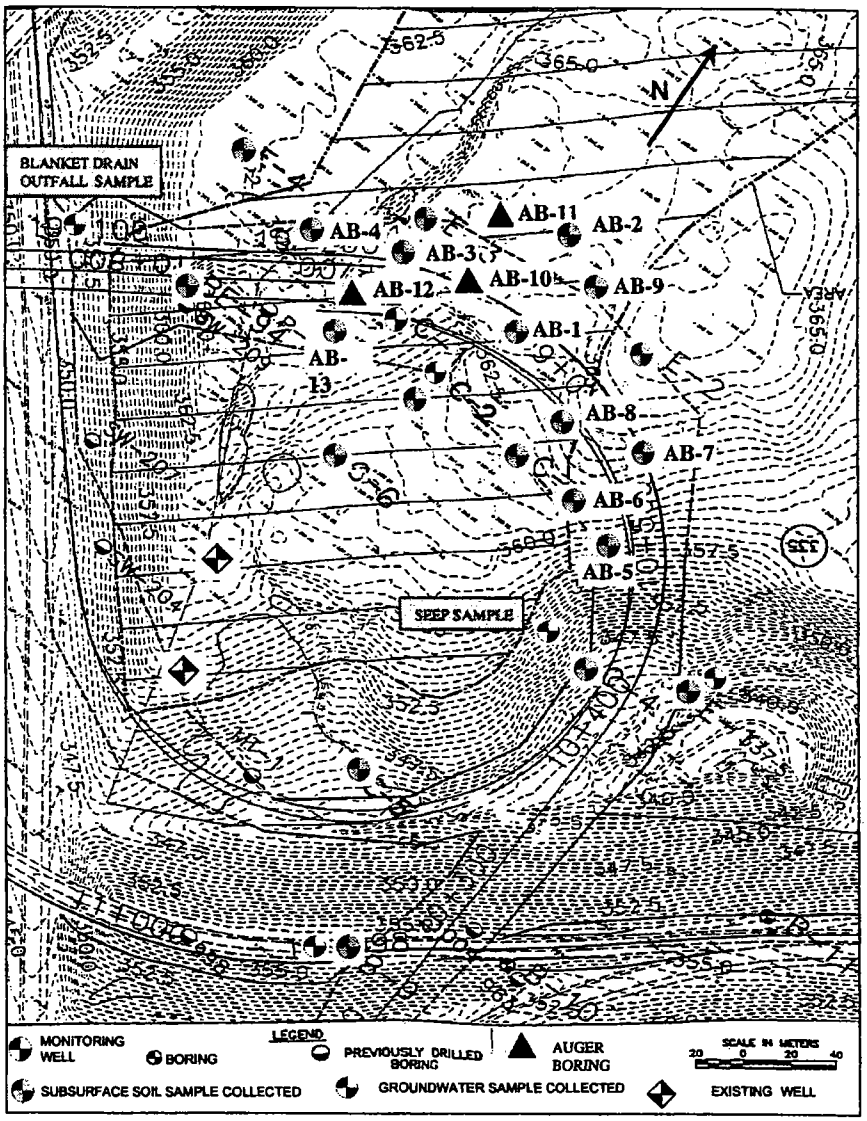

Figure 11. Location of environmental sampling points.

Monitoring Well Installations

In order to characterize the groundwater migrating across the site, several monitoring wells were installed in the geotechnical borings (Figure 6). Monitoring wells were installed in borings BE-94, B-9, C-1, C-2, C-6, and E-1. In addition to these new well installations, three existing well locations were also to be sampled. The existing monitoring wells were installed as a part of previous investigations for S.R. 60 and the Conveniences Store / Service Station work. Analytical samples were drawn from B-9, C-2, and E-1 monitoring wells (Figure 11). The remaining wells were dry during the sampling event.

INTERPRETATION OF RESULTS

Soil Gas Survey Result Interpretation

Results of the soil gas survey in this area are presented in Figures12-14. As shown in these figures, elevated readings of benzene and toluene were detected in the southwestern corner of the sampling grid. This area parallels the former cropline of the base of the Pittsburgh coal seam. The former Pittsburgh coal seam cropline was determined by review of historic borings. The base of the coal was taken to be the lowest elevation where mine spoil, soil, or waste was encountered. A review of

the historic test boring logs installed during design of SR 60 showed the lowest elevation at which waste material and / or mine spoil was observed was El. 348 meters. As can be seen on the figure, this elevation contour wraps around the southwestern and southeastern edge of the soil gas survey area, corresponding to the locations of the surveys positive detections.

The majority of the positive detections for ethylbenzene and xylene were along the southwestern edge of the sampling grid. In addition to paralleling the coal cropline, this area corresponds to and is aligned with the blanket drain installed to intercept drainage migrating towards SR 60. This suggests that contaminant migration by groundwater may have or is currently occurring. The blanket drain may be intercepting slope seepage and carrying it to the roadway drainage system. Soil gas readings directly above this drainage collection area would presumably be higher than less permeable materials behind the drain. The highest detections for these compounds were in the same approximate area as the benzene and toluene positive detection described above, further suggesting contaminant migration via groundwater in the southern direction from the landfill. These areas are closer to the water table and therefore soil gas sampled here is closer to the emitting dissolved phase, resulting in higher concentrations. The remaining detections for ethylbenzene and xylene occur in the footprint of the former disposal trench (based on aerial photographs) and support the observation that the swale may be due to subsidence of the waste materials.

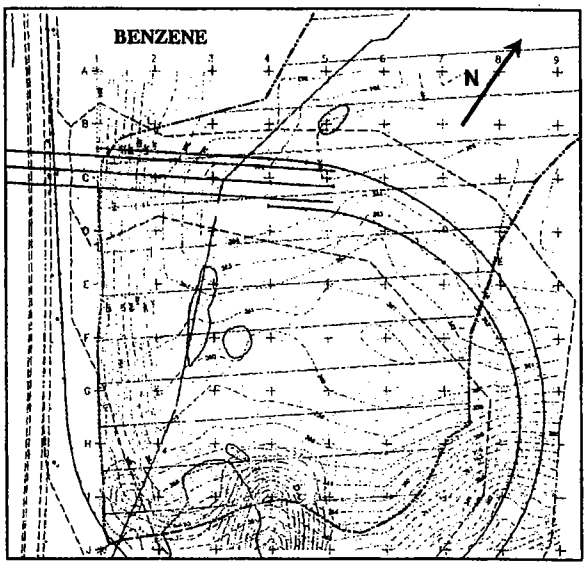

Figure 12. Soil gas survey results for benzene.

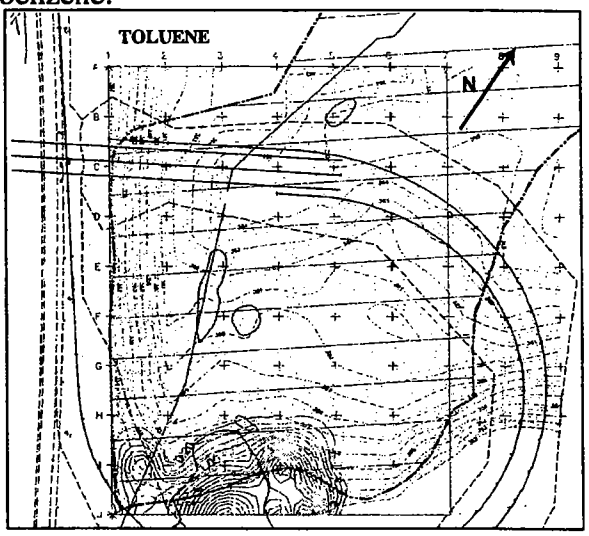

Figure 13. Soil gas survey results for toluene.

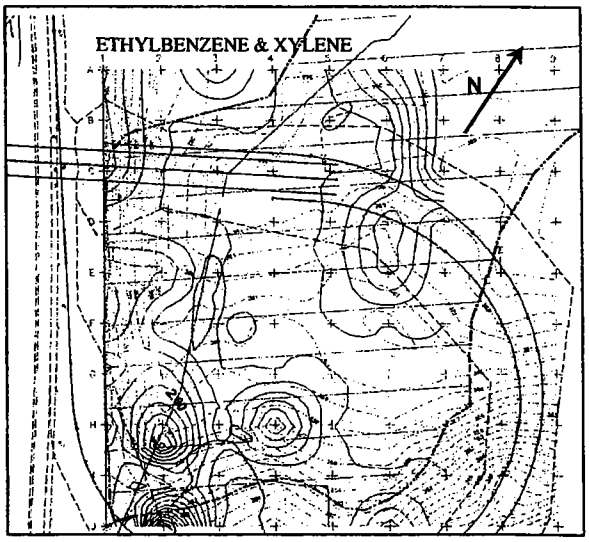

Figure 14. Soil gas survey results for ethylbenzene and xylene.

Electromagnetic Survey Results Interpretation

Terrain conductivity and magnetic susceptibility data were collected simultaneously through the use of a digital data logger. Terrain conductivity is a measure of the apparent conductivity of the subsurface materials and their pore fluids. Terrain conductivity data are collected in units of millimhos per meter (mmhos/m). Figure 15 shows the results of the terrain conductivity survey at the site. As this figure shows, the survey found one linear anomalous area trending north south through the center of the site. This area corresponds to the suspected burial trench observed in the 1955 aerial photograph (Figure 4). The electromagnetic response is consistent with what would be expected in a trench filled with garbage or a soil and garbage mixture. These buried materials would likely be more permeable, and therefore have a higher moisture content than the surrounding denser soil. This higher moisture content would be reflected by the EM instrument as higher material conductivity.

Magnetic Survey Result Interpretation

As previously stated, the purpose of the magnetic survey was to serve as a screening tool to identify strong magnetic anomalies which may indicate the presence of buried drums. The results of the magnetic survey, as shown in Figure 16, do not indicate the presence of any magnetic anomalies which are significantly greater than background values for the site and surrounding area. The magnetic data was compared with the electromagnetic data, aerial photographs, and soil gas survey results. Based on this comparison, the most significant

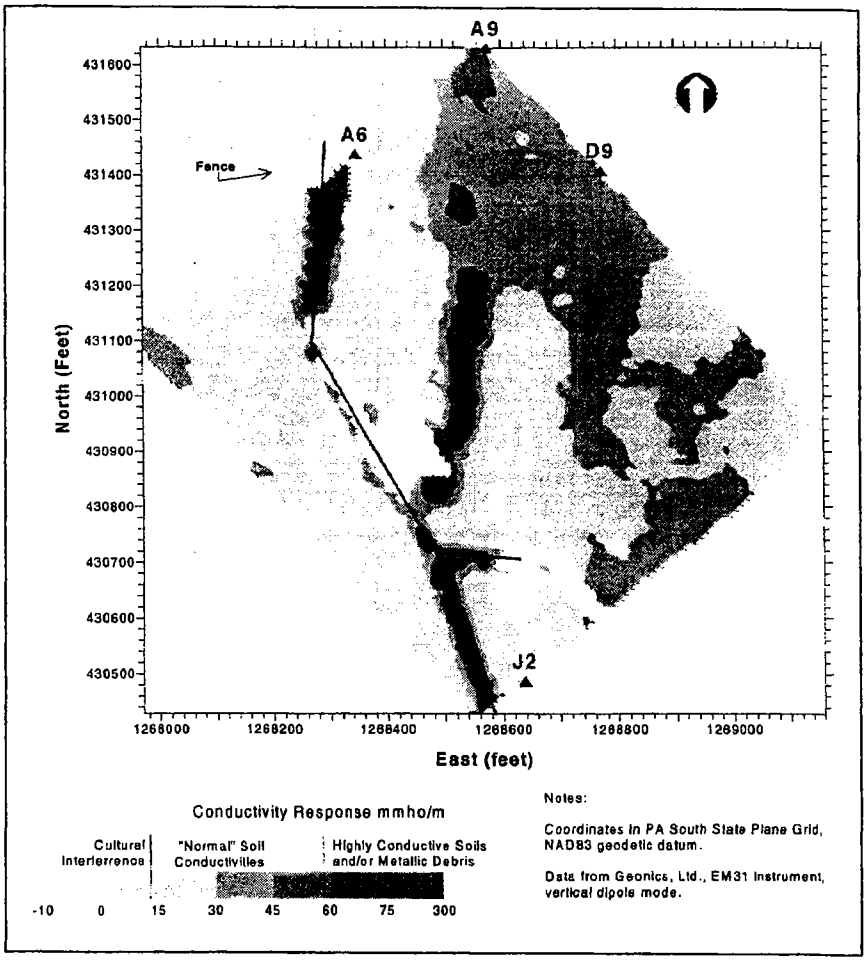

Figure 15. Results of terrain conductivity survey, note north-south trending anomaly corresponding to the location of the trench in 1955 aerial photographs.

magnetic anomaly locations were investigated with auger borings. Auger borings were advanced in an attempt to collect additional data regarding the nature of the anomalies. No evidence of drums nor higher levels of contaminates were noted in the auger boring samples. No definitive correlation between the magnetic survey and the EM survey could be concluded from this data other than a portion of the magnetic anomalies coincided roughly with the disturbed areas observed in 1971 aerial photos (Figure 6).

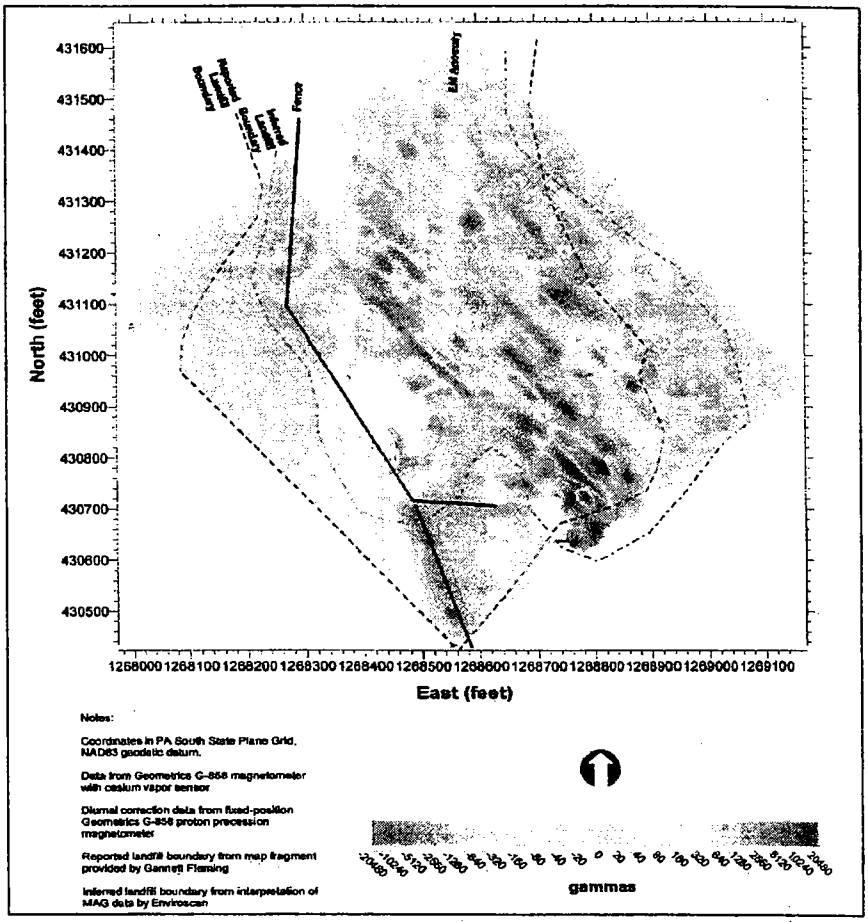

Figure 16. Results of megnetometer survey. The largest anomalies were investigated through the installation of auger borings.

Solid Waste Delineation

Based on the visual observations during the discussed field investigations, the nature of the solid waste material is consistent with decomposed municipal landfill wastes intermixed with strip mine overburden rock material. The occurrence of solid wastes in the geotechnical and auger borings correlate well with the historical aerial photographs. Therefore, it is anticipated that the limits of the solid waste deposition are as shown in Figure 17. It should be noted, however, that isolated pockets of waste may be encountered in other locations than those shown. Solid waste can be expected to be encountered along the Ramp E baseline between Station 10 + 610 and 10 + 645, and between Station 10 + 690 to 10 + 760. Within these areas, it is anticipated that the solid wastes extend from just below existing grade to the top of rock which is the depth of the strip mining operations.

Soil Contamination

A summary of the positive analytical soil detections for the soil samples collected is presented in Figure 18. This data is also summarized in Table 2. As the table indicates, the site soils contain several volatile and semi-volatile organic chemical constituents. The primary chemicals of concern

at this site are napthalene, bis (2-ethylhexyl) phthalate, and PCB 1254. These chemicals were found in site soils in concentrations exceeding Pennsylvania DEP clean fill standards (PaDEP, 1999). All samples were found to contain concentrations well below the governing state wide health standard, typically the Pennsylvania Land Recycling Act (Act 2) soil to groundwater value (Commonwealth of Pennsylvania, 1996), and therefore classify as non-hazardous.

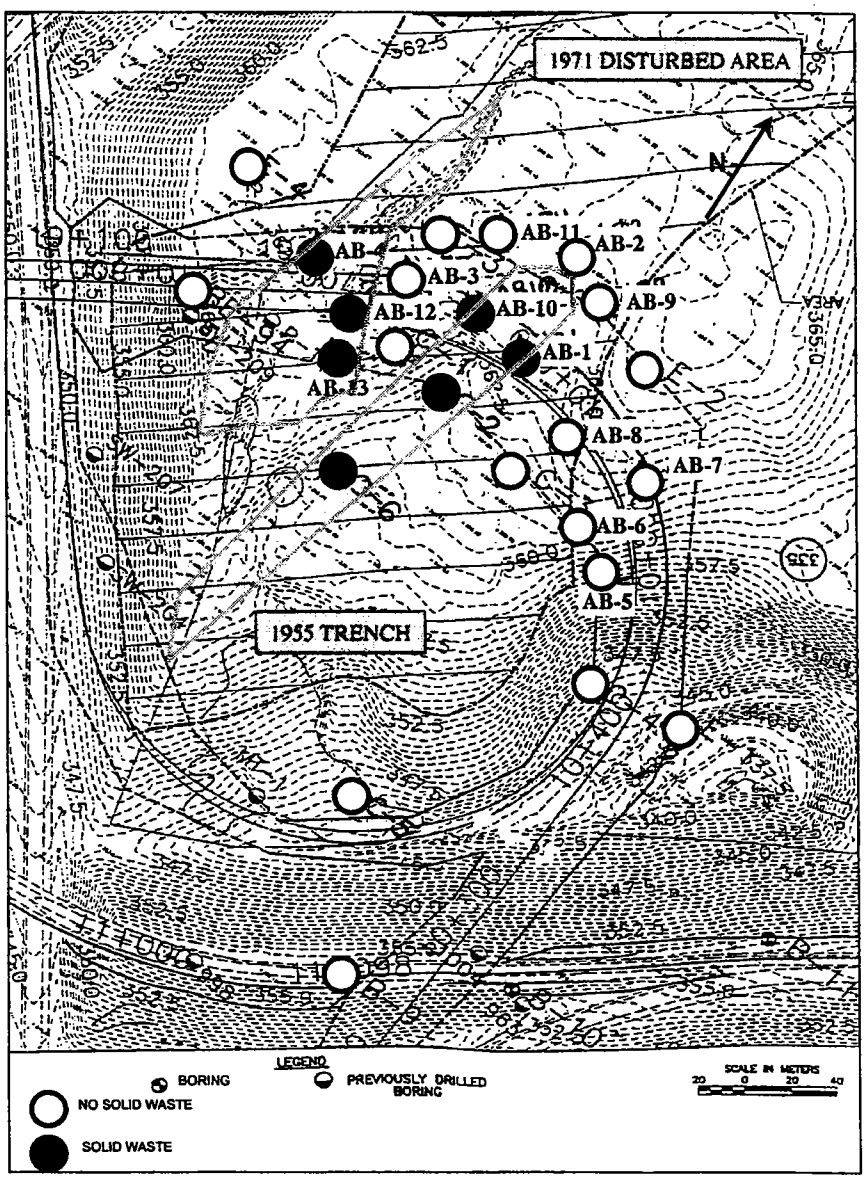

Figure 17. Delineation of solid waste materials based on the results of geophysical surveys, geotechnical borings, and auger borings.

Napthalene and bis (2-ethylhexyl) phthalate positive detections were encountered in samples taken in the suspected disposal areas. The samples with PCB concentrations are generally those collected beyond the limits of solid waste deposition (as defined herein). This suggests that the potential for sporadic, area-wide PCB 1254 contamination may exist. It can not be determined (based on the current data) whether the PCBs are related to the landfill activities.

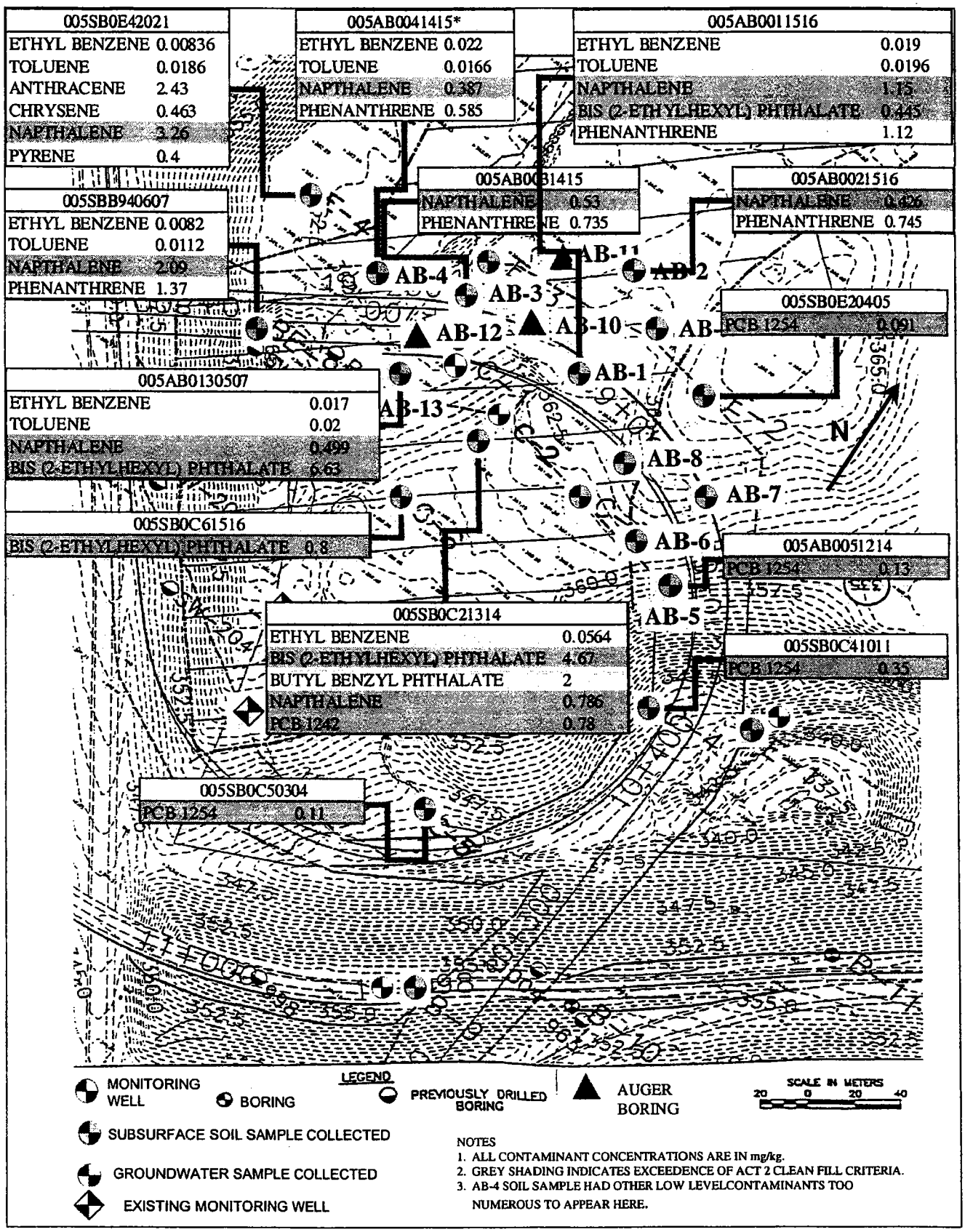

Figure 18. Positive analytical detections for the subsurface soil samples collected at the site.

Groundwater Contamination

Of the four groundwater samples obtained (from B-9, C-2, E-1, and Seep 1), only the C-2 sample returned a positive analytical detection. This sample contained benzene at a concentration of 5.4 μg/L. The state health standard threshold value for benzene in a non-use non-residential aquifer is 500 μg/L. Therefore, based on the data collected for this investigation, no groundwater remediation is warranted.

RECOMMENDATIONS

Based on the results of the field investigation, Gannet Fleming made several recommendations regarding roadway construction through the site. In order to prevent roadway subsidence, it has been recommended that the solid waste material not be used as back fill or be allowed to remain in place below the roadway. Therefore it has been recommended that the standard roadway template be over excavated to the top of the soil –rock interface (Figure 19). This over excavation will extend the entire length of the cut section in the Ramp C and E corridor. This will ensure that all waste below the roadway will be removed. The construction contractor will segregate the excavated material into waste and soil.

The soil from the excavation will be considered restricted use fill, which can be used in two applications. The first application for this material will be as compacted backfill used to bring the over excavation to within 1.52 meters of final grade (Figure 20). The second application will be to provide an encapsulation layer surrounding the waste material in a non-structural fill constructed

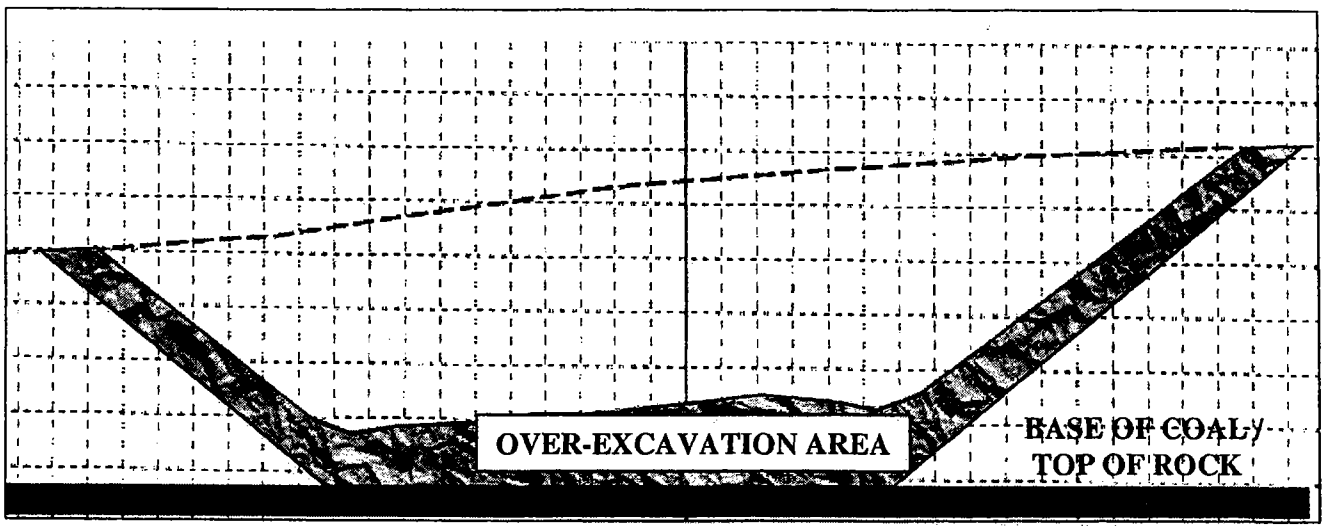

Figure 19. Schematic of over excavation recommendation.

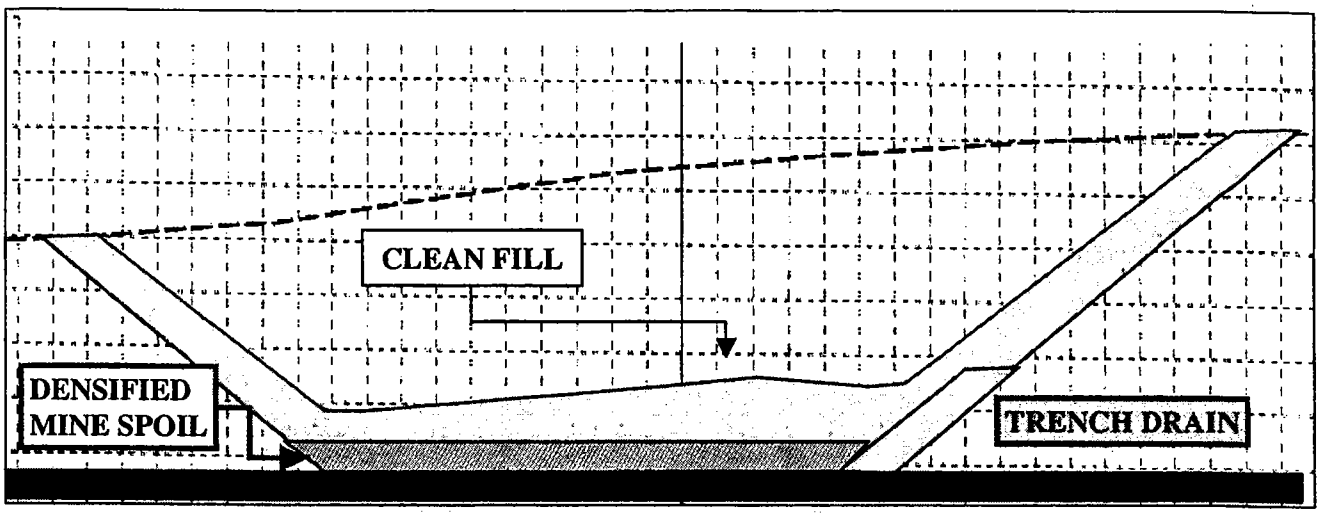

Figure 20. Schematic of recommendation showing application of restricted-use fill, trench drain, and clean fill.

adjacent to the roadway (Figure 21). The remaining distance between the restricted use fill and final roadway grade will be filled with clean fill (from off site). This will serve to provide a barrier between the solid waste and potentially contaminated soil and the finished roadway. Gannett Fleming, Inc. has also recommended that an interception drain be installed on the upgradient side of Ramp E in order to divert potentially contaminated groundwater from the roadway facility. This proposed drain would discharge into the existing airport stormwater facility.

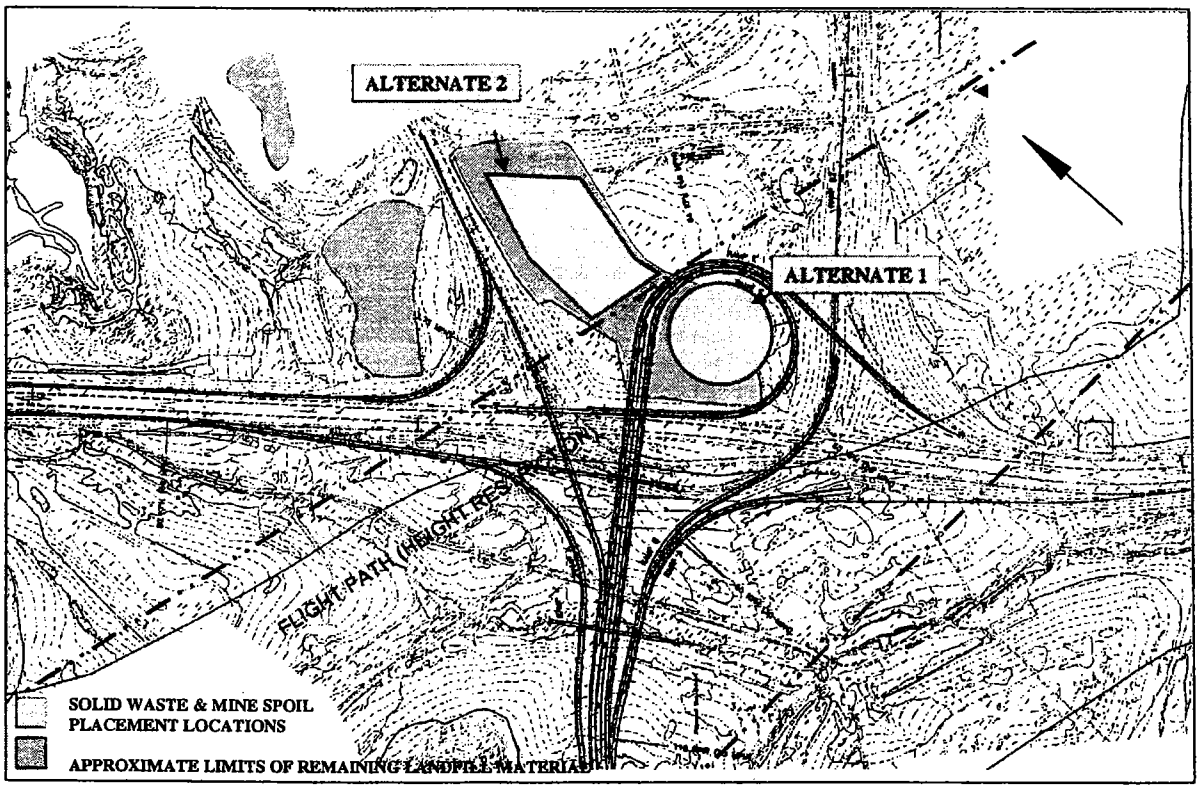

Figure 21. Locations of proposed waste encapsulation areas. Encapsulation at either alternate is pending approval from Allegheny County and PaDEP. The third disposal alternative would be offsite transport to a permitted landfill.

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Four Mile Bridge to North of Standard Tunnel

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Abstract

The final project that completed the upgrade of The West Virginia Turnpike to Interstate Standards was one of the most complex and largest earth moving projects that the WVDOT has ever constructed. The WV Turnpike was mostly a two lane road, which was constructed, in the early 1950's.

The project involved making a thru cut in Pennsylvanian aged Sandstone, Shales and Coal and filling the valley below the cut. The cut and fill were adjacent to the existing roadway, which contained the Memorial Tunnel and Bender Bridge. The elevation difference between the bottom of the valley to the top of cut was 900 ft. The project involved excavating approximately 10 million cubic yard of material and constructing a 385 ft. rock fill in the adjacent valley.

Four mile creek flows east through the valley and empties the watershed to the west of the site. Special design considerations were given to carrying Four Mile Creek beneath this huge fill. Either the creek had to be rerouted through a diversion tunnel cut in the sandstone bedrock, or routed through a cast in place concrete arch culvert.

Constructing the cut and fill took place while traffic was still traveling through the Memorial Tunnel and crossing over the Bender Bridge. To monitor the effects of blasting and excavation of the cut, Memorial Tunnel was outfitted with instrumentation at three stations. At each instrument station, three extensometers and a convergence meter were installed. In addition to the extensometers and convergence meters, the tunnel instrumentation program included three inclinometers and two seismographs. The instruments were monitored daily and after each blast to make sure the Tunnel was safe for the travelling public. The bridge piers were monitored with fixed survey points.

The Bender Bridge was demolished upon completion of the project and the road was opened to traffic. The opening was marked as a great engineering achievement for the completion of the WV Turnpike to Interstate Standards. An access road is maintained to the south portal of the tunnel. With removal of the bridge, the only reminder of the existing road is the South Portal Tunnel entrance perched on the side of the mountain which marks what was considered in 1954 as "an engineering achievement of heroic proportions."

Introduction (History of WV Turnpike)

The conception of the turnpike dates back to 1947 when the West Virginia Legislature created the West Virginia Turnpike Commission. The Commission envisioned a four-lane highway that would stretch from Charleston to Princeton. After organizing itself, the commission on October 25, 1949 assigned the consulting firm of Howard, Needles, Tammen and Bergendoff to do an extensive study to determine the best and most financially feasible route for such a roadway. In November 1951, after 16 months of intensive study and preparation, a location was designated for

the roadway. The original design concept was to construct a four-lane road. Bank interests who were financing the work refused to pay for four-lane construction when prospective revenues were judged to be too small to finance it.

After approximately six years in the planning, construction of the turnpike took less than two years to complete. It was hailed as "the greatest road building project West Virginia has ever seen" and "an engineering achievement of heroic proportions." With an original cost estimate for the Turnpike of 78 million, the commission issued 96 million, 3 ¾ percent revenue bonds, in April 1952. Cost over runs pushed the final figure to 133 million, which required issuing additional bonds.

As a part of the turnpike construction Memorial Tunnel was constructed in 1954 at a cost of 5 million dollars. The tunnel was a single bore tunnel and extended for a length of 2600 ft. The tunnel was constructed using the drill and blast technique. The permanent tunnel liner was constructed of 23" thick reinforced concrete with 8" steel ribs located 4 to 6 feet on centers. The 2600 feet of the Memorial Tunnel eliminated the need for the turnpike to go an additional 26 miles by existing routes. The Memorial Tunnel was the first tunnel constructed that was monitored by close circuit television.

The southern portal of the tunnel immediately entered onto the Bender Bridge, spanning Four Mile Creek, 280 feet below. Bender Bridge was named after Stanley Bender, a native of Fayette County, WV and WWII Medal of Honor winner. When constructed, this bridge was the highest span east of the Mississippi.

In 1958 the West Virginia Turnpike, along with those of Ohio and Pennsylvania, was incorporated into the nationwide Interstate and Defense Highway System established two years earlier. The incorporation did not provide funds for upgrading. The Federal Highway Act of 1966 set interstate design standards that required at least four lanes of traffic. The 1968 Highway Act included a provision permitting the state to use Interstate funding for acquisition and reconstruction of the Turnpike beginning in fiscal 1970. After disputes of land acquisition and an agreement to retain tolls until outstanding bonds were liquidated, the first contract for the upgrade was awarded in January 1973. Upgrading of the turnpike continued through the 70's and 80's. The last project to complete the upgrade, was the area between Four Mile Creek Bridge (Bender Bridge) and Standard Tunnel (Memorial Tunnel). Alternative designs to complete this upgrade consisted of two broad alternatives. The existing tunnel and bridge would be improved, a new bridge and tunnel would be constructed, or the mountain would be cut and the adjacent valley filled.

Although constructs cost would be lower for the tunnel and bridge alternative, it was viewed that in the long term, maintenance cost for the bridges and tunnels would be higher than the one time cost of constructing the cut and fill. The alternative to construct a new tunnel and bridge never received serious consideration.

The amount of material required to be cut and filled was one of the largest single projects contemplated by WVDOT. Constructing this huge cut and fill presented several areas of concern that would have to be addressed, by the Department, Consultant Designer (Sverdrup Parcel), and the Contractor that was awarded the contract. The elevation difference from the top of the

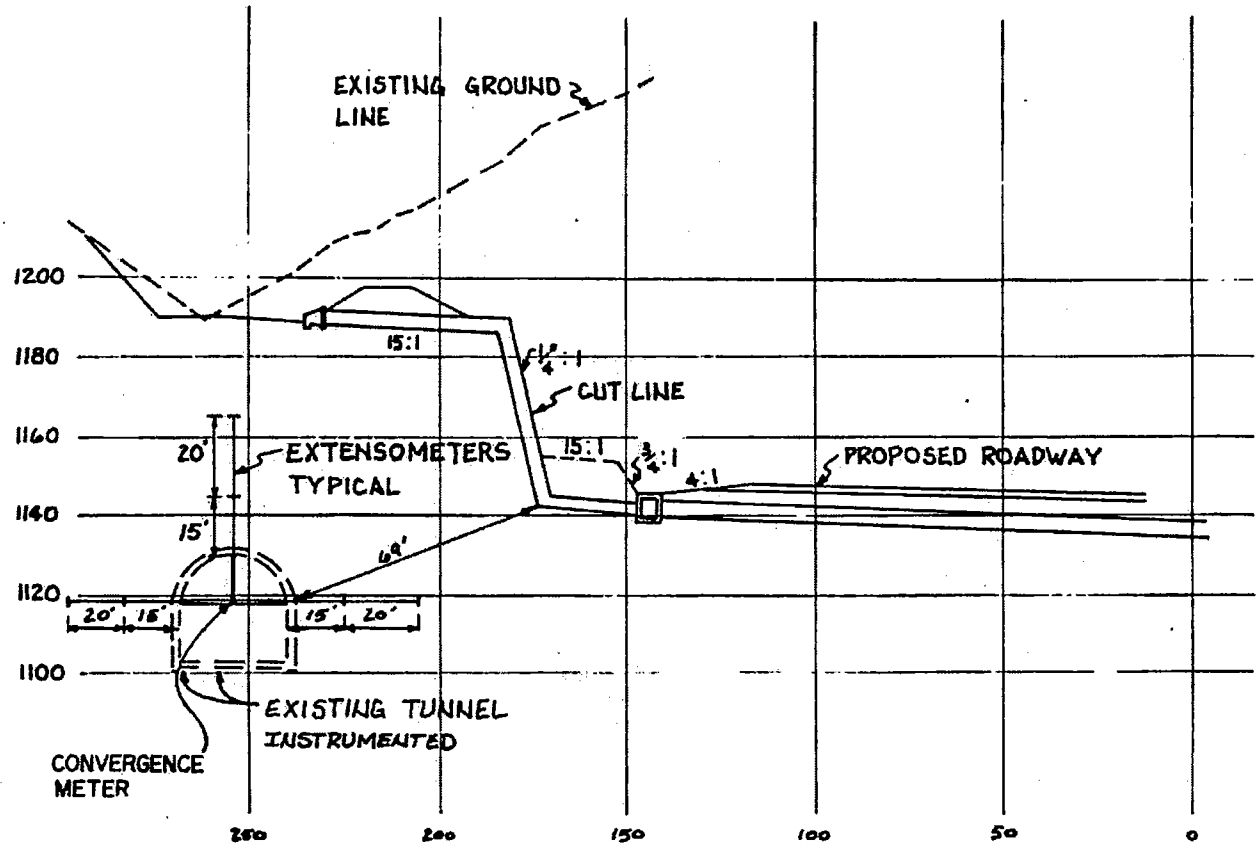
mountain to the valley bottom was 900 feet. The difference in elevation would test men and equipment as the cut and fill progressed. The location of the cut was adjacent to the Memorial Tunnel. The tunnel would have to remain open until the project was completed. Blasting techniques would have to be monitored closely. The piers that support Bender Bridge were located on the sides of the gorge that would be filled. Like the tunnel the bridge would have to remain open until the project was completed. Four-mile creek, which flowed beneath Bender Bridge, would have to be routed beneath the fill. Special design considerations would have to be given for constructing a culvert that could support the weight of a 355-ft. high fill. A coal mining operation was located to the west of the project; access would have to be maintained during construction and a permanent access constructed.

Instrumentation of Memorial Tunnel & Bender Bridge

Before any construction began on the project, Sverdrup Parcel was contracted to plan a strategy to monitor the effects of blasting and construction activities adjacent to the Memorial Tunnel. Sverdrup Parcel researched the as built plans of the tunnel, and established deflection criteria and what effect the deflection would have on the tunnel. The first (northern) 400 feet of the tunnel liner shows some distress. The ceiling hangers in the exhaust duct are bowed up to 7 ½ inches out of plumb. Fine line horizontal cracks were found in the air duct and walls especially where the ceiling hangers are bowed. The tunnel roadway ceiling at the north portal is cracked and has about 2-inch vertical displacement. Instrumentation was installed at three locations within the tunnel, near the north portal. The northern portion of the tunnel came into the closest proximity of the cut. At each location a total of three sets of extensometers were installed. One set was installed in the crown line and two on each side of the spring line. The extensometer instrumentation was intended to measure rock movement horizontally and vertically 35 ft around tunnel. The rock and tunnel liner movements in the horizontal direction can be obtained by assuming anchors C1 are fixed (See Figure 1). At each location a convergence meter was installed. The convergence meter measured the inward or outward deflection of the tunnel. At two sites seismographs were installed to measure the peak particle velocity (ppv) of the blasts. All instrumentation was monitored electronically and data was collected in a room at the north portal of the tunnel. All instrumentation was in place and being monitored for base line readings 6 months prior to any blasting activity.

Deflection Criteria established by Sverdrup Parcel

Springline less than 1" Crown less than ½"	1. Inspect Ceiling If o.k. open to traffic if broken fix ceiling and open to traffic.
Springline between 1" & 2" Crown between ½" & 1"	1. Call in experts. 2. Inspect ceiling and liner If both o.k. open to traffic If broken ceiling only---fix ceiling and open to traffic. If cracked liner close tunnel until experts say ok
Springline greater than 2" Crown greater than 1" Or a sudden jump in readings.	1. Close to traffic & open when experts say o.k. 2. Inspect ceiling & liner 3. Call in experts.



SECTION STA. 1458+50
PROPOSED ROADWAY STATION
TUNNEL ROADWAY RELATIONSHIP
Figure 1

Fill was to be placed around the concrete piers of Bender Bridge. Horizontal and vertical tilt plates were initially installed on the concrete piers to measure possible deflections of the piers. Due to the difficulty in reading the tilt plates, readings were never initiated. Fixed survey targets that could be read from fixed ground points replaced the tilt plates.

Design & Construction

The engineering estimate to construct this project was 50 million. Centerline cuts measured up to 385 feet through a sequence of Pennsylvanian age lower Allegheny and upper Kanawha Formation sandstone, siltstone, coal and shales. Several coals of commercial thickness would be exposed. It was not certain how much coal remained in the mountain and was not removed by mining. The following list contains the name, thickness, and elevation for the location of the coal.

Coal Name	Thickness	Elevation
5 Block	5.0'	1422
Little 5 Block	2.0'	1400
Stockton 'A'		
Split 1	2.0'	1373
Split 2	2.5'	1358
Split 3	1.5'	1354
Stockton	5.0'	1327
Coalburg	6.5'	1237
Winifrede	4.5'	1196

The fill width measures one-quarter mile across the base, and had to be constructed around the bridge piers. Sand cushions were to be placed around the piers, as earthmoving equipment would come within 5 feet of them. The fill would reaches 385 feet in height with excavation amounts of 10 million cubic yards. .

One of the most intriguing aspects of the design concerned the routing of Four-Mile Creek beneath the fill. Two design alternatives were developed. The first alternative was to construct an arch culvert that closely matched the existing channel location. Due to the weight and the amount of fill that was to be placed on the arch culvert, the design required the culvert to be founded on bedrock. The second alternative was to move the stream and relocate the channel through a diversion tunnel. The diversion tunnel was designed to be cut through bedrock on the south side of the gorge. Based on the subsurface investigation the tunnel would cut through fine grain hard sandstone. The length of the diversion tunnel was 1,885 lineal feet. Mashuda Corporation of Evans City, Pa. was awarded the contract for 34.8 million. Mashuda's bid on the contract included the alternative to construct the diversion tunnel.

The first stages of the project consisted of constructing a haul road for the fill and constructing the diversion tunnel, while maintaining access across the fill for the coal mining operation. Mashuda's biggest problem was starting the project. Vic Mashuda stated "Our bigest problem was getting that first load of dirt to the bottom of the fill a thousand feet below. Our Company had to spend $1.5 million at the beginning of the project just to build a haul road to get the material off."

The haul road was constructed on a 24 percent grade with six switchbacks. The width of the haul road was 60 feet wide. Two ten-hour shifts began working on the project. Both shifts combined moved approximately 19,000 cubic yards of material. As the haul roads were being constructed work was started on the diversion tunnel. Cowin & Company Inc. Birmingham, Alabama was sub contracted to construct the diversion tunnel. To begin construction a tunnel-cutting machine was brought to the site from England to cut the 9ft. diameter tunnel. The tunneling machine was effective for cutting the hard sandstone, but to slow. The tunneling machine progressed only 10 feet in 24 hours. Use of the tunneling machine was abandoned and traditional drilling and blasting techniques were used to complete the tunnel. Using the drilling and blast method, an average of 25 ft. could be cleared during one shift.

Work continued year round on the cut. The contract allowed blasting on even hours. The timing of the blast was limited so the public could anticipate when the delays would occur. Usually the blast took place at 12:00 noon and 6:00 p.m. Non electric caps were used for the shots with each bast going off individually. This type of blasting was felt to cause the least potential for damage to Memorial Tunnel. Most shots broke up 10,000 –15,000 cubic yards of material. Before each shot, traffic was stopped before Bender Bridge to the south, and just before the north portal of Memorial Tunnel. After each shot was put off, readings from the instrumentation was obtained and traffic was released after verification the tunnel and bridge were safe. Normal delays for each blast was 10-15 minutes.

As the fill progressed access was maintained for the working coal operation to the west of the project. A high level access was constructed through the fill near the top. The high level access provided access to the coal operation and the south portal of Memorial Tunnel. The high level access was constructed by National Engineering Strongville, Ohio. Upon completion of the project Bender Bridge would be removed and the only access to the south portal would be through the High Level Access. During construction it was simply not certain what fate was in store for the soon to be abandoned tunnel.

Photo 1 View of Bender Bridge and South Portal of Memorial Tunnel prior to construction of the open cut and fill.

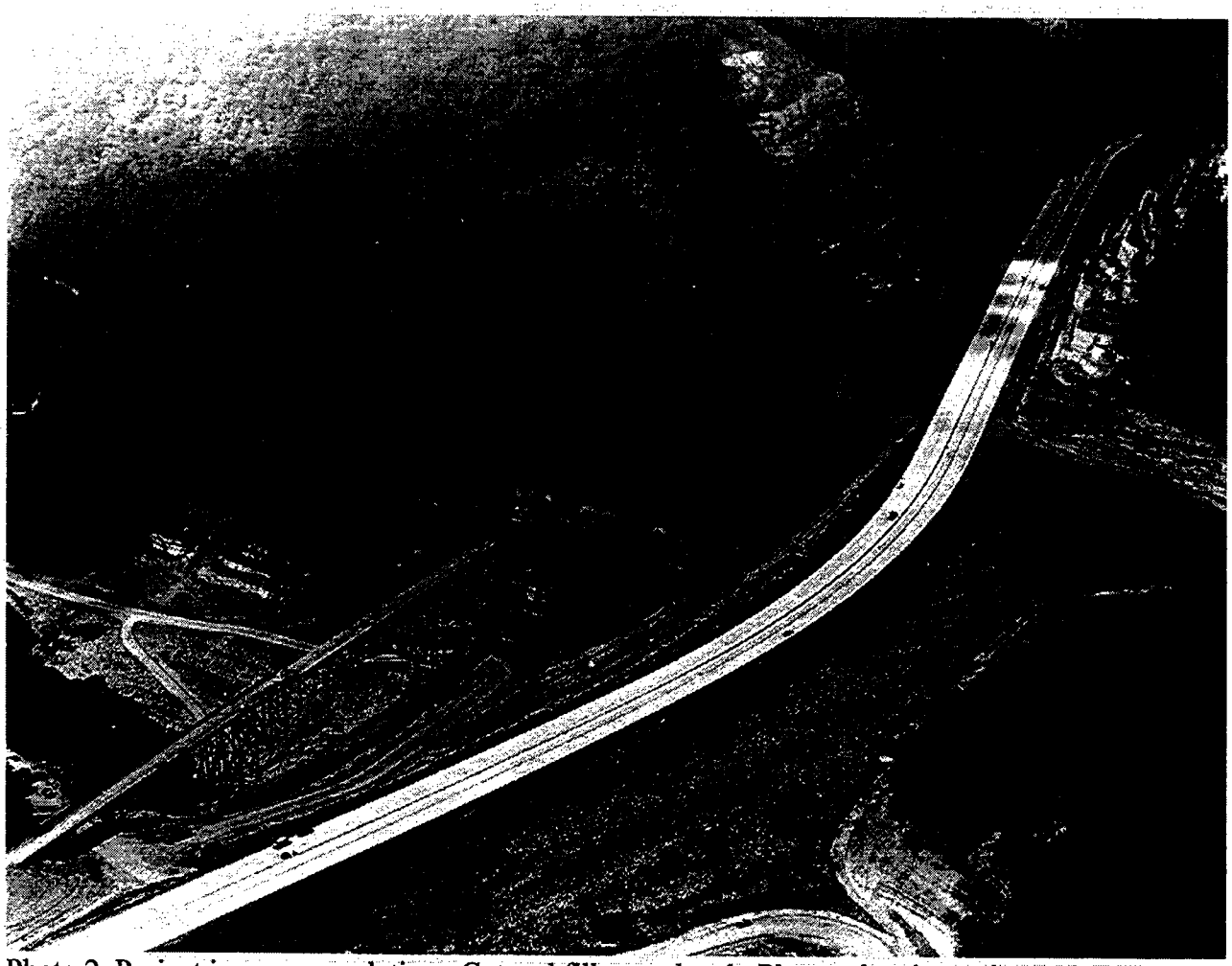


Photo 2 Project is near completion. Cut and fill completed. Photo taken just prior to demolition of Bender Bridge.

Summary

The instrumentation monitoring was maintained through the project completion. The extensometers and convergence meters did not measure any significant movement of the bedrock or the tunnel liner. The project was completed and traffic was maintained through the tunnel and Bender Bridge with only slight delays. However, longer traffic delays occurred when several blast were recorded that were greater than 2 inches ppv. Each time of this occurrence a thorough visual inspection of the tunnel took place and Mashuda was notified to modify their blasting techniques.

Mashuda sold approximately 300,000 tons of coal that was removed from the cut. This amount was much larger than was anticipated.

The existing crack in the ceiling liner opened wider during construction. The opening of the crack was judged to have occurred due to the accumulation of all, not one specific blast event near the north portal of the tunnel. Loose pieces of concrete fell onto the roadway, fortunately no one was injured and no vehicles sustained damage. Loose pieces of concrete were removed from the

ceiling and wire mesh was tacked to the ceiling to prevent any further pieces of concrete falling onto the roadway. The tacking of the loose concrete to the ceiling was successful, and remained in place until the projects completion.

The diversion tunnel proved an effective way to carry Four Mile Creek beneath the fill. All bids received on the contract include the diversion tunnel alternative. The arch culvert was costly and would have required a massive amount of excavation to found the culvert on bedrock.

The project took 3 years to complete and was finished and opened two months ahead of schedule. The opening marked the completion of the West Virginia Turnpike to Interstate Standards. The completed upgrade took 11 years to complete using 19 contractors from 10 states. The final price tag for upgrade of the 89 miles was 622.8 million.

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REVIEW, ROCKFALL HAZARD RATING AND REPAIR OF AGING INTERSTATE ROCK CUTS IN NORTHEASTERN VERMONT

By - Thomas D. Eliassen - VTrans
Peter C. Ingraham - Golder Associates

ABSTRACT

Like many departments of transportation, the Vermont Agency of Transportation (VTrans) is addressing maintenance and repair issues on aging interstate highways. As part of a program of Interstate Safety Projects being completed with FHWA, VTrans is evaluating the condition of the rock slopes along I-89, I-91 and I-93, which were constructed 18 to 44 years ago. These slopes have undergone significant weathering and dilation and are experiencing increasing numbers of rockfalls. VTrans policy for justifying rock slope removal on Interstate Safety Projects considers accident related areas, areas of historical rockslides, high maintenance areas and areas of insufficient clear zone. The FHWA Rock Hazard Rating System (RHRS) was used as the basis for evaluating and rating the slopes. However, it was necessary to adapt the RHRS for the weather and traffic conditions in northeastern Vermont. The initial work involved inspecting, rating and ranking a total of 175 slopes. Based on a combination of quantitative and qualitative evaluation of the results, 52 slopes were recommended for maintenance and/or repair. For each slope alternatives were identified that provided a range of reduced risks and costs. The alternative methods of remediating the slopes and reducing potential rockfall hazards were reviewed with FHWA. Recommended alternatives were reviewed by Vermont's Act 250 Environmental Commission for aesthetic and other potential environmental impacts that could be anticipated during and following construction. Recommended alternatives include scaling, trim blasting, recutting slopes, rock bolting, buttressing and installation of rockfall catch fences and wire rope net drapes. Typical examples of past repairs and planned repairs to the slopes are also presented.

Introduction

Vermont has approximately three hundred and twenty miles of Interstate highways. The Interstate system consists of I-89, I-91, I-93 and I-189 (Figure 1). All of these highways are four lane (two in each direction) limited access highways.

Interstate 91 is a north-south artery that parallels the Connecticut River Valley from the Vermont-Massachusetts border north to St. Johnsbury, Vermont. Here I-91 departs the Connecticut River Valley and progresses north to the Vermont-Canada border at Derby Line, Vermont. The total length of I-91 is approximately one hundred and seventy eight miles.

Interstate 89 enters Vermont from New Hampshire at White River Junction, Vermont and progresses northwesterly to Burlington, Vermont at which point it turns to the north and continues in a northerly direction to the border between Vermont and Canada.

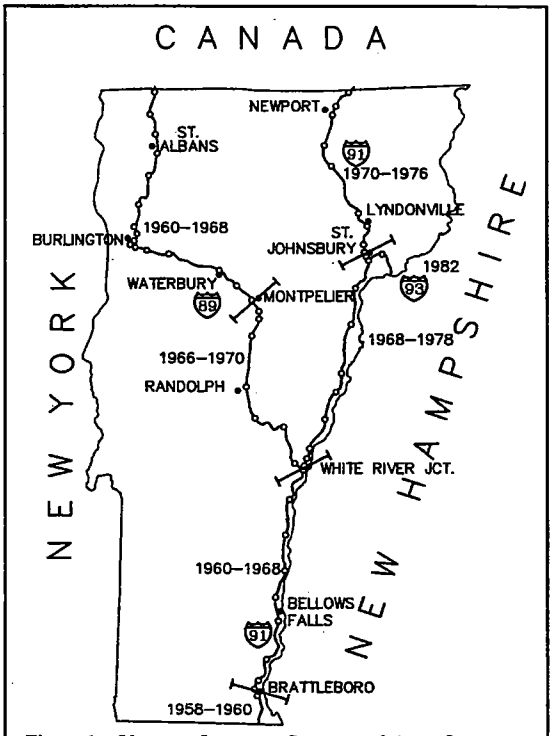

Figure 1 – Vermont Interstate System and Age of Segments

I-89 is approximately one hundred and thirty miles in length.

Two short connector highways link traffic to I-91 and I-89. Interstate 189, one mile in length, is located in South Burlington and connects traffic between U. S. 7 and I-89. Interstate 93 enters Vermont from New Hampshire at Waterford, Vermont and runs eleven miles west to intersect I-91 just south of St. Johnsbury, Vermont.

Construction on Vermont's Interstate System was started in 1956 with a section of I-91 beginning at the Vermont-Massachusetts border and was completed in 1982 with the construction of I-93 from the border with New Hampshire to it's intersection with I-91 just south of St. Johnsbury. Interstate 89 was completed in 1970, Interstate 91 in 1978 and Interstate 93 in 1982. Figure 1 shows the dates when each of the phases of interstate construction were completed.

The design and construction of rock cuts during the time period between 1956 and 1982 underwent a number of changes that are reflected in the characteristics of individual rock cuts along the Interstates. Generally, early design and construction procedures were based primarily on economic considerations and little consideration was given to rock structure in the design phases of the project. Standard rock slope design at that time adhered to a 4 vertical to 1 horizontal slope. As construction progressed northward, cut stability conditions necessitated investigations by the highway geologist and where warranted, some rock cuts were cut at angles favorable to the geology of the area. Early rockfall catchment areas were designed so the toe of the slope was 23½ feet from the travel lane. This was extended to 30 feet beginning in 1971. Pre-splitting was introduced in 1972. Also in 1972 an earthen cushion was added at the toe of new rock cuts. Approximately 210 miles were designed by state engineers/geologists while consultants designed about 110 miles.

This paper presents a review of previous rock fall studies and remediation and a discussion of interstate safety projects investigated in 1997 and 1998.

Previous Rockfall Investigations, Remediation and Regulatory Considerations

In 1978 the Vermont State Geologist (Charles Ratte) and Chief Geologist of the Vermont Highway Department (Frank Lanza) performed a reconnaissance of all rock cuts greater than twenty feet in height along I-89 and I-91. This reconnaissance consisted of rating each cut for the following seven criteria:

	Criterion	Maximum Rating Points
1.	Height of roadcut	
	Less than 20 feet	0
	20 to 30 feet	1
	30 to 40 feet	2
	Greater than 40 feet	3
2.	Distance from toe of cut to pavement (if less than 20 feet)	1
3.	Planar geologic structures dipping towards roadway at angles of:	
	Less than 30 degrees	1
	30 to 70 degrees	3
	70 to 90 degrees	1
4.	Topographic slope directed toward roadway	1
5.	Presence of groundwater or surface water	1
6.	Past history of rockfalls/rockslides	1
7.	Other recognizable enhancing conditions	1

Total scores from 1 to 3 indicate low rockfall potential, 4 to 6 indicates moderate potential and 7 to 10 indicates high potential.

Based on their evaluation, forty rock cuts were identified as high potential sites for rockfalls and were recommended for detailed geologic analysis (Phase II Assessment). Locations identified as low to moderate risk potential were recommended for regular maintenance (light scaling and routine clean up of fallen rock in ditches). It does not appear that Phase II assessments were performed at that time, perhaps due to a lack of funding. Regardless, it is interesting that numerical rating systems for evaluating rock cuts were being used over twenty years ago.

Other ranking systems have been developed in the last 20 years. The New York Thruway implemented a rockfall hazard rating system based on a 1986 paper by Duncan Wyllie on rock fall hazard evaluation. The Thruway RHRS was adapted from the FHWA RHRS in 1997 to include provisions for the high volumes of traffic experienced on their system. The Thruway also developed a rock slope database as part of their systemwide database of structures and other design elements. In 1989, the Oregon State Department of Transportation developed a comprehensive Rockfall Hazard Rating System (RHRS) under an HPR pooled-fund study. This system was later adopted by FHWA and was used in the rating of slopes for this study. The New York State DOT implemented a ranking system in 1993 and modified the system in 1996. The Ontario Ministry of Transportation has also recently (Draft 1999) developed a system that builds upon the Oregon/FHWA system by adding eight more criteria in the ranking system. There are undoubtedly other systems in place and under development in other states that are similar to those mentioned above.

Between 1989 and 1994, VTrans completed four interstate safety projects which included removal of loose rock, removal of median ledge, laying slopes back to shallower angles and excavating slopes back to provide a wider clear zone. In preparation for these projects, slopes identified in the 1978 Lanza and Ratte study were reviewed and where necessary, detailed evaluations were completed. Two of these safety projects spanned the section of interstate 91 from the Massachusetts border to White River Junction, and two were completed along I-89 from Royalton to Waterbury, Vermont.

During the later portions of the above mentioned safety projects public pressure was placed upon VTrans regarding the removal of rock from cuts in the area of Montpelier (state capital). A coalition formed which opposed the excavation of rock. The popular sentiment was that rock exposures represent one of the signature visual attributes of Vermont and as such, blasting would destroy the existing beauty of the cuts. Years of weathering and deterioration of the rocks have left some cuts in a ragged condition. Part of the opposition was concerned that half-barrel casts left on the finished slope face is aesthetically unpleasing. Others questioned the Agency's justification for excavating rock along highways.

As a result of the scrutiny placed on the Agency, in April of 1994 VTrans revised their "Ledge Removal" policy. The revised policy identified four conditions that would justify the excavation of rock ledges. Those areas are:

- Accident-related areas (>3 accidents involving vehicles striking ledge within a 0.3 mile zone)
- Rockslide areas (Areas where there is a clear and present danger of rock falling onto the roadway)

- High maintenance areas (Areas that have required repeated clean-up actions because of falling rock)
- Insufficient clear zone, reduced sight distance, ice build-up problems or other areas that have been identified in accordance with appropriate engineering standards.

Excavation of rock (called "ledge" in New England) under any of these conditions require the approval of the VTrans Secretary of Transportation. In addition, the policy states that "where ledge is to be removed, it shall be done as much as possible by mechanical means".

In 1994 the Vermont Environmental Board issued an Advisory Opinion that VTrans would be required to file for an Act 250 permit for any remaining interstate safety projects. Act 250 is Vermont's land use permit that is required for certain construction/development activities. Act 250 evaluates the project's impact on ten criteria. These criteria address various economic, historic, environmental, aesthetic and municipal concerns. A considerable amount of effort and time were expended investigating the possible impact to the ten Act 250 criteria. In the end the Environmental Board determined that the remaining interstate projects (those addressed in this paper) represent maintenance of existing infrastructure and therefore do not require Act 250 permits.

Geologic Setting

Vermont's geology is characterized as a sequence of folded and faulted sedimentary rocks (limestone, dolomites and shales) in the Champlain Valley (western side of the state); Precambrian schist and gneisses in the core of the Green Mountains which occupies the highlands in the middle of the state; and schist, phyllites and limestone of the Connecticut Valley sequence in the eastern portion of the state (Figure 2). The rocks of Vermont have undergone significant deformation during the Taconic and Acadian orogenies resulting in intensely folded and faulted strata. The overall trend of bedding and foliation throughout the state is north-northeast/south-southwest.

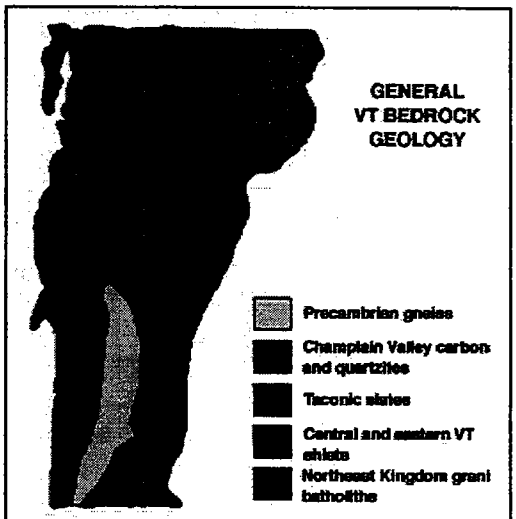

Figure 2 Generalized Geologic Map

The structural geology along I-91 reflects significant tectonic activity and occupies the eastern margin of the Connecticut Valley-Gaspe Synclinorum. The interstate follows the trend of the Ammonoosuc Thrust fault and crosses the fault above Norwich and again in Thetford. The orientation of bedrock bedding and foliation generally parallels these major structures and strikes roughly north 5 degrees east (N5°E) to north 20 degrees east (N20°E) or parallel to the overall alignment of the nearby river and the roadway. Bedding and foliation generally are steeply dipping to near vertical along much of the alignment.

The predominant rock types along I-91 and I-93 consist of phyllitic schist with quartzite and graywacke interbeds and greenstones and felsic chloritic schist. The phyllitic and chloritic schist near the Ammonoosuc Fault is highly folded and exhibits curvilinear fabric that is in contrast to orthogonal jointing in the rocks. Recent slope repairs at Fairlee were complicated by such conditions.

The twenty two-mile section of I-89 investigated during the 1997-1998 safety project program trends parallel to sub-parallel to the rock fabric. Rock along this alignment consists of limestone, schist and phyllites of the Waits River and Gile Mountain formations.

In engineering terms the rocks in Vermont are mainly strong, hard and durable. There are only a few places where the rock has weathered or slaked appreciably since the highway was constructed. The stability of the rock slopes is controlled by the discontinuities in the rock (the joints, bedding planes, faults, blasting fractures, etc.) rather than by the strength of the rock mass, (i.e., the strength of the rock mass is far greater than the stresses produced by the construction of the slopes).

The rock falls that do occur are a result of individual blocks and semi-intact masses of rock sliding along discontinuities or being released to topple or fall by the discontinuities. None of the rock falls occur as a result of failure through intact pieces of rock.

In the nearly 20 to 45 years since the rock cuts were excavated, many of the discontinuities have opened up and a number of blocks of rock have worked loose due to ice jacking, water penetration, soils infilling, vegetation growth and vibrations from heavy traffic. Some of these loosened blocks now pose a risk to the highway.

Investigation Program

The objectives of the 1997-1998 Safety Project investigations were to:

- identify areas where rock slopes present a risk to the motoring public;
- develop a rating for each slope of the relative rockfall risk to the public;
- identify potential corrective measures and relative costs to reduce risks presented by rockfalls from the slopes;
- review the corrective measures for potential impacts relative to ACT 250 criteria; and
- review the corrective measures relative to the VAOT policy on ledge removal.

The 1997–1998 study included a review of existing information and past remedial measures constructed by VTrans, and completion of a rockfall hazard ranking of slopes along I-89, I-91 and I-93. Specific problem slopes were then mapped and potential remedial measures to reduce rockfall risk were identified. The problem slopes were analyzed for kinematic stability and rockfall bounce simulations were run to assess the probability of rockfalls reaching the roadway. Input from VTrans was sought during the field program to ensure sufficient information was being collected for Act 250 hearings. Following completion of the mapping, the identified remedial alternatives were field reviewed by the VTrans Transportation Geologist and Golder Personnel.

The investigation included a review of existing information on rock slopes collected by the Transportation Geologist, Mr. Thomas Eliassen and his predecessor Mr. Alan McBean. These data included mapping of specific rock cuts that had experienced rockfalls and data on recent remedial measures completed in Fairlee in 1996. The Fairlee project is located on a segment of one of the 1997-1998 safety projects, and consisted of a 300-foot high slope on Interstate 91 that was experiencing raveling failures and significant rockfalls that necessitated a rapid remedial response. Remediation consisted of extensive rock scaling, installation of ten spot rock bolts and draped wire mesh (Figure 3). The early study of rock slope hazards completed by Lanza and

Ratte in 1978 was also reviewed. This early study provided insight into past areas of rockfall risk and a benchmark of relative slope risks for comparison of 1997 slope conditions.

As part of the Rockfall Hazard Rating, maintenance personnel in the VTrans Districts were contacted to provide information on chronic rockfall areas that have required repeated maintenance. Although detailed records were not available, maintenance personnel contacted were able to identify problem slopes. Finally, remedial actions under consideration by VTrans for all the Interstate Safety Projects systemwide, and those completed for the Fairlee area in 1996 were also reviewed to ensure similar measures and levels of risk reduction were considered for the ongoing projects.

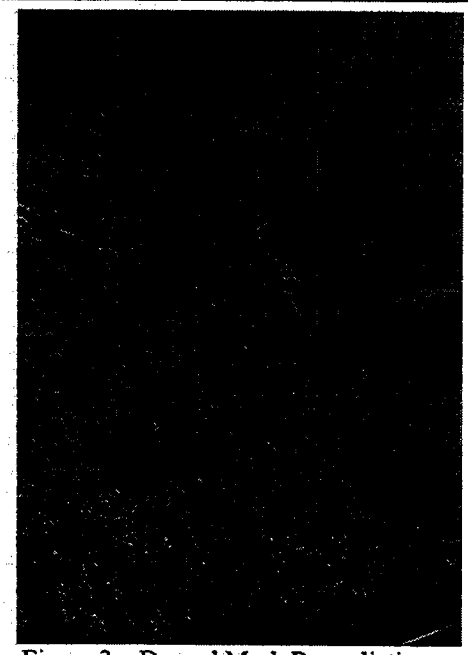

Figure 3 – Draped Mesh Remediation at Fairlee

At the onset of the study, the VTRANS Policy on Ledge Removal was reviewed to identify likely conditions where rock removal/slope modifications could be justified. The policy was further reviewed with the VTrans Project team and the Attorney General's office. Also reviewed were Act 250 criteria and potential environmental concerns relative to prospective rock slope treatments and aesthetic measures that could be implemented to reduce visual impacts associated with rockfall mitigation measures.

Rockfall Hazard Rating

The slopes in the project area were evaluated using the Rockfall Hazard Rating System (RHRS) described in the Federal Highway Administration (FHWA) Publication Number SA-93-057. The RHRS described in the FHWA publication formed the basis for the quantitative rating of rock slopes along the I-91, I-89 and I-93. As recommended in the RHRS user manual, the system was modified to address conditions in Vermont. Specifically, climate was ignored because of the size and similar conditions statewide, and consideration of traffic exposure was reduced because, while traffic volumes in Vermont are lower than for many parts of the country, the occurrence of rockfalls is rare enough that they are virtually unanticipated by the motoring public.

The rockfall hazard rating of the project slopes was conducted in two stages. The first stage included a preliminary rating to group rock slopes into categories of low, moderate or high rockfall history or potential for rockfalls based on a visual assessment. The second stage consisted of a detailed rating of the high and moderate risk slopes. When the RHRS exercise was complete, the results were reviewed to evaluate which slopes might require stabilization measures to reduce the rockfall hazard. These slopes were mapped as Stage 3 of the fieldwork. Photographs of the critical slopes were copied and used during the mapping, Figure 4.

Mapping

The objective of the mapping was to identify, map and quantify the most critical blocks, wedges and areas of the slopes that should be scaled, trim blasted, supported, draped with mesh, laid-

back, have rockfall catch fences or otherwise treated to reduce the rockfall hazard. The mapping

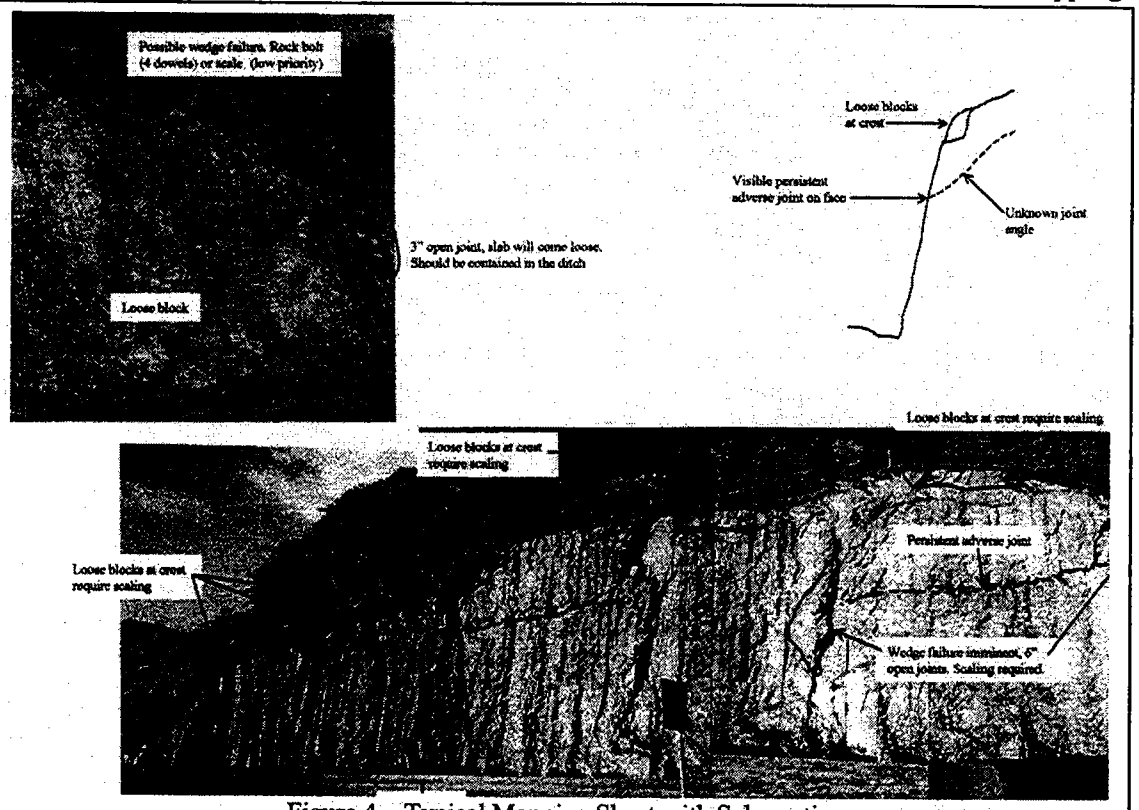

Figure 4 – Typical Mapping Sheet with Schematic

was documented on panoramic photographs of the slopes that are intended to form the basis for construction drawings for the remedial work. Information on access for construction equipment and other requirements for the construction activities was also collected during the mapping.

The mapping process involved photographing the problem slopes, compiling panoramic views of the problem slope areas to form mapping sheets and annotating the color copies of the mapping sheets in the field. Mile markers along the roadway were used to locate the photographed sections, and were noted on the mapping sheets in the field. Field measurements of the slope geometry including slope height, cut angle, azimuth of the rock cut, ditch configuration, and offset from the paved roadway were made. Geologic data were also collected including lithology, visible structural features such as shear zones, bedding and joint orientations (dip and dip direction), and the size and shape of rock blocks and unstable rock masses was noted. Joints and other discontinuities in the rock slopes were observed for orientation with respect to the cut slope geometry, spacing, roughness, dilation, seepage, infilling, persistence, frequency and spacing. The orientation of the line of intersection between planes of conjugate joint sets was observed and the size and shape of scars on the rock face, together with joint and rock cut orientations were used to identify past and potential modes of failure.

Data Analyses

In keeping with the VTrans Ledge Removal Policy, slopes mapped as warranting remedial action were analyzed using the Colorado Rockfall Simulation program (CRSP). The CRSP program was used to verify potential for rockfall debris reaching the roadway, and to estimate the energy,

bounce height and velocity of falling rocks at various locations on the modeled slopes to allow design of catch fences and evaluation of changing catch ditch geometry to improve catchment.

Detailed surveys of slope and ditch cross section were made for those slopes where rockfall catch fences were identified as a remedial option. Idealized cross sections were developed for the remaining slopes based on field measurements of ditch width and depth, slope angle and height.

A kinematic review of the mapping data was used to confirm the modes of failure identified in the field and relative risk of certain block configurations. The analyses involved evaluation of the orientation of bedding planes, joints, fractures, and foliation, and the spacing and friction along these surfaces, to identify the size and shape of rock blocks that can fall from a rock face. Major failure modes germane to the Vermont slopes include toppling failures, planar failures and wedge failures. Circular failures generally occur in weak rock formations and are not likely to occur in the Vermont rock slopes.

Study Findings and Recommendations

Based on the field mapping program, the predominant issues with slopes having elevated risk of rockfall center on the presence of loose rocks or rock masses, rather than other rating criteria such as traffic volumes, poor ditch capacity or site distance/clear zone problems. The presence of loose rocks was generally caused by the following:

- there was insufficient scaling during the initial construction;
- over time, ice-jacking, water infiltration or vibrations have caused the joints to dilate and blocks to detach from the remainder of the rock mass;
- tree roots have pried the blocks loose;
- the rock has weathered locally;
- the slopes were originally cut steeper than the dominant geologic discontinuities;
- the rock fabric and geologic structures are irregular or curvilinear and create unstable blocks in particular locations; and
- blasting was not effective during the original construction of the slopes.

Of particular interest were the last three issues where the rock was cut more steeply than the discontinuities, curvilinear fabric was present, and where presplit lines were lost and blocks were left at the crest of the slopes.

The structural geology of the region presents some challenges to rock slope design. Most of the rock has discontinuities that dip at 50 degrees to 65 degrees and strike nearly parallel to the roadway alignment. There are also areas of rock fabric, which present planes of weakness in some areas that appears as curving planar features in exposed rock faces (Figure 5).

The original construction of the slopes at 4V:1H or 75 degrees was satisfactory in most locations. However, there are local areas that have experienced post construction difficulties over time. By cutting the slopes

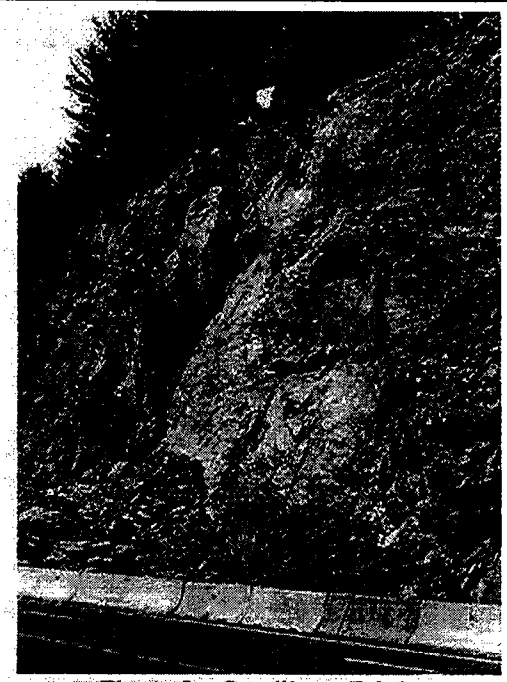

Figure 5 – Curvilinear Fabric

at angles steeper than the bedding, joints and fractures, steep planar blocks were allowed to form (Figure 6).

Some standard 4V:1H slopes in areas of folded rock with curved fabric have experienced rockfalls. The Fairlee slopes recently remediated by VTrans have a section of curved fabric immediately north of the zone draped with mesh to control rockfalls. Rockfall risk in this area is reduced by a large ditch and rockfall runout area.

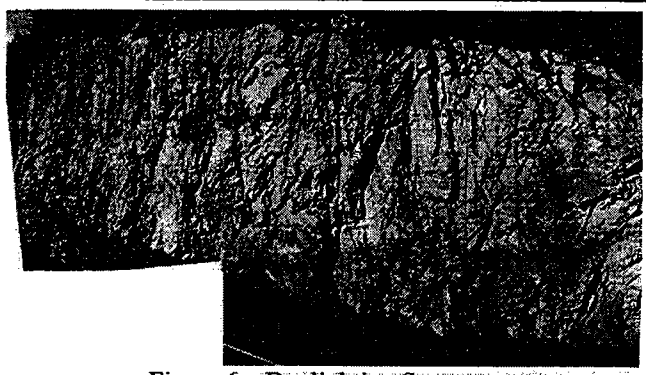
Figure 6 – Daylighting Structure

The remnants of construction blasting that are visible on the I-91 and I-89 rock slopes consist of half-barrel casts or the remains of holes made by rock drills. The alignment and presence of half-barrel casts were used in this study to identify blocks that had moved following construction, and to assess the effectiveness and control of construction blasting. Most slopes have areas where the blast had followed the pre-split line and controlled blasting had been effective. In some instances, it was apparent that not all the rock was removed and not all the rock along the pre-split line had cracked. In these areas, humps of rock were left at the crest, at the toe or on the slope. The high rock masses that were left in place have subsequently weathered and the presplit line and joints in the rock have dilated. The low masses represent humps at the toe of slope that could "kick" a falling rock onto the roadway (Figure 7).

A total of 175 slopes were reviewed as part of the safety projects. Of these, the preliminary hazard ratings indicated that 95 slopes warranted additional study due to their height, narrow catch ditch, adverse geology, poor condition or combinations of these parameters. These slopes were rated using the Rockfall Hazard Rating System and over 60 slopes were mapped as part of this study.

Figure 7 – Hanging Rock Mass

As a result of this mapping and rating, 52 slopes are recommended for repairs and remedial treatment to reduce the risk of rockfall to the travelling public. The methods of reducing the rockfall risk evaluated for each slope included:

- Hand and Machine scaling;
- Trim blasting;
- Slope angle reduction (or laying the slope back);
- Increasing ditch width;
- Rock bolting;
- Rock lashing; and
- Installation of rockfall catch fences and draped mesh/wire rope net.

For some of the slopes, a single, inexpensive method of repair was sufficient and recommended. However, for the remainder of the slopes, more than one option for reducing the rockfall hazard was considered as shown in Table 1.

SLOPE (Direction/Location /MM-MM)	Current Risk to Travelling Public	Option	Estimated Cost	After Repair Risk	Comments
NB/Median/71.55-71.65	Moderate	1. Scaling and Trim Blasting	$25,000	Low	Removal of 75 cubic meters (100 yd3) of rock
NB/Outside/71.55-71.70	Moderate to Light	1. Machine Scaling 2. Lay Slope Back to 45o	$65,000 $450,000	Low to Mod Very low	Scaling of +300 cubic meters (400 yd3) of rock 58,000 cubic meters (76,000 yd3) of rock
NB/Outside/95.65-95.75	Low to Moderate	1. Machine Scaling 2. Medium Capacity Rockfall Catch Fence	$15,000 $27,000	Low Very low	110 cubic meters (140 yd3) of rock
NB/Median/95.73-95.95	Moderate	1. Lay Slope Back to 60o 2. Medium Capacity Rockfall Catch Fence	$102,000 $55,000	Very low Low	6,500 cubic meters (8,500 yd3) of rock
SB/Outside/92.75-92.67	Moderate	1. Rockfall Catch Fence 2. 6' High Gabion Wall	$60,000 $20,000	Low Low to Mod	May be less effective

Table 1 – Typical Options for Slope Repairs

The Vermont Environmental Board reviewed the selected alternatives and decided that the remedial actions constituted maintenance of existing slopes and further review and Act 250 permitting activities were not required for the project. Review within VTrans during the design was completed to ensure that the requirements of the Ledge Policy on rock removal were met. Similarly, the work was reviewed with the State Attorney General's office to ensure that the requirements of Act 250 were fully considered and the Environmental Board would understand that alternatives addressing aesthetic and impact mitigation during and after construction were fully considered.

Conclusions

The Oregon/FHWA RHRS is a good system for addressing rockfall risk on aging interstate slopes. As outlined in that system, consideration of unique regional characteristics must be factored into the system. Similarly, judgement and objectivity are important elements of rating slopes. The benefits of taking a proactive stance in identifying several alternatives for each slope needing remediation were twofold:

- cost/benefit consideration was provided to management to assist in the decision making process; and
- discussion of alternatives, including construction and long-term aesthetic and natural impacts, ensured environmental review personnel were satisfied that such impacts/considerations were part of the decision-making process.

The alternatives developed for the slopes ranged from low cost remedial methods such as scaling and vegetation removal to higher cost alternatives of rock stabilization and slope reconstruction.

Discussions of the efficacy of the alternatives in reducing risks and provision of relative costs allowed VTrans to weight the relative benefits of each alternative and identify the level of risk they were willing to accept following remediation.

At the present time, the first of the 1997-1998 Safety Project Program designs is complete and through the permitting stage. Three remaining Interstate segment designs are currently in the preliminary through contract plans stages of design. Construction should be substantially complete within the next three to five years.

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Fracturing Within Anticlines and Resultant Modes of Slope Instability

by Tom Badger
Washington State Department of Transportation

Folding strata into an anticline creates shear stresses parallel to bedding within the fold crest and proximal limbs and tensional stresses due to flexure that are radial to the crest. These stresses commonly produce two predominant joint sets, S_1 coincident with bedding (S_0) and S_2 perpendicular to bedding. For slopes that strike parallel to the fold axis, these two joint sets can produce three structurally-controlled modes of slope failure.

Dip slope failures along bedding (S_0) and joint set S_1 are common and can be very large. An example of this failure mode exists within a very strong, dacite tuff of the Oligocene age volcanic rocks of Huckleberry Mountain located near Snoqualmie Pass, Washington. Here, S_1 dips towards the southwest at around 35°. In 1957 during the construction of Interstate 90, excavation at the base of a high slope undercut this structure, resulting in its catastrophic failure which killed several construction workers.

Where bedding (S_0) dips favorably into the slope, failures along joint set S_2 and across bedding can also occur. This mode of slope failure is commonly observed in southern Whatcom County, Washington in a fold belt of bedded sandstone and siltstone of the Eocene age, Chuckanut Formation.

Lastly near the fold crest, flat or shallowly dipping (even favorably so) S_0/S_1 joints and intersecting S_2 joints define blocks that can be mobilized by high groundwater pressures. A dramatic cut slope failure occurred in Aberdeen, Washington in 1996 in the Miocene age, Montesano Formation. This failure involved a block approximate 150 feet high, 600 feet long, and 80 feet thick resting on a favorably oriented bedding surface (S_0/S_1) dipping into the cut slope at about 10°. The block failed during an extreme storm event, presumably driven by very high water pressures behind the block, along S_2, and beneath the block along S_0/S_1.

Rock Slope Remediation on the Pennsylvania Turnpike - Bedford, Pennsylvania

by Corry Goumans and Ron Woolf
Pacific Blasting and Demolition, Ltd., Burnaby, B.C.

This paper describes the rock slope remediation performed along America's first Superhighway, the Pennsylvania Turnpike during an 8-month project in 1999. This work was performed on an existing rock cut slope within the Bedford Narrows, a water gap formed by the Juniata River through Evitts Mountain. These rocks are in the folded Juniata and Tuscarora Formations within the Valley and Ridge Province and strike northeast with near vertical overturned bedding.

Public safety and unimpeded traffic flow throughout the project area of this busy highway was of prime importance. Protection from falling rock was provided with an in-house modification of a proven rock fall containment net that was first tried by Washington State Department of Transportation on the I-90, Snoqualmie Pass in Washington State. The protection system allowed for safe vehicular travel on the Pennsylvania Turnpike while rock excavation and scaling activities were performed. Specialized drilling equipment was designed and fabricated for the project that permitted pattern bolting along the rock face.

Highway Rock Cut Stability Assessment in Rock Masses Not Conducive to Stability Calculations

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Key Terms: *Rock Cuts, Slope Stability, Discontinuities.*

Abstract

Ensuring the stability of highway rock cuts requires an evaluation of the structure of the rock. Failures usually initiate and follow pre-existing discontinuities rather than break through intact rock. Rock determined to be loose or with the potential for failure must be removed or restrained in some way. Analytical calculations of potential instability can range from limiting equilibrium analysis to numerical modeling.

In many terrains the discontinuities are oriented in such a way that they contribute to create wedge, planar, or toppling failures. These are then relatively easy to analyze. In other terrains, most notably flat lying sedimentary rock with vertical jointing, the predominant failure mechanism tends to be raveling, which is typically not conducive to calculation.

This paper describes some of the techniques currently used for stability assessment, as well as speculating on future trends in analysis and data acquisition.

Introduction

The safety and convenience of the motoring public demands that highway rock cuts be made safe. Catastrophic failures of rock cuts can result in property damage, injury, and even death. Highways impeded by even small spills of rock material are in addition an inconvenience for motorists (Figure 1).

Ensuring the stability of highway rock cuts, whether new or old, requires an evaluation of the structure of the rock. Failures usually initiate and follow pre-existing discontinuities rather than break through intact rock. Thus it is the nature of the discontinuities (joints, fractures, bedding planes, faults, and other breaks in the continuity of the rock) and not of the intact rock that governs the mechanical and hydrological behavior of the rock mass (Figure 2). With a few exceptions, most of the rock masses that engineers deal with are influenced to some extent or another by discontinuities. While the understanding of the mechanical properties of intact, solid rock is well advanced in rock mechanics, the understanding of the fundamental behavior of discontinuous rock is significantly less well developed.

Planar and wedge sliding and toppling analysis and remediation

In many terrains the discontinuities are oriented in such a way that they contribute to create wedge, planar sliding, or toppling failures (Figure 3). Franklin and Senior (1997b) report that of

Figure 1. Example of rock failures causing obstructions in roadways

Figure 2. The discontinuous nature of rock masses

Figure 3. Example of wedge, planar, and toppling failures along road cuts.

415 analyzed cases of failure in Northern Ontario, only 33% of failures involved these mechanisms (23% toppling, 8% planar sliding, 2% wedge sliding). These are failures are however easy to analyze, and can range from limiting equilibrium analysis to numerical modeling (Hoek and Bray, 1981; Piteau, 1979c; Piteau, 1979d). The mapping of discontinuity orientations is a requirement, before or after the cut has been exposed (Piteau, 1979a; Piteau, 1979b; Piteau 1979g). Rock determined to be loose with the potential for failure must be removed or restrained in some way (Piteau, 1979e). Prescribed designs for remediation and/or mitigation are easy to find (Brawner, 1994; Konya and Walter, 1991; Piteau, 1979f; Franklin and Senior, 1997a).

Raveling type failure modes

In the Northern Ontario study, 65% of the failures were of the "raveling" type (Figure 4). These included raveling (25%), overhang/undercutting failure (15%), ice jacking (14%), and rolling blocks (11%). In other terrains, most notably flat lying sedimentary rock with vertical jointing, where planar and wedge slides are unusually not found, the predominant failure mechanism being of the raveling type is even greater.

These raveling failures, whether slow, time-dependent or fast and catastrophic are much more difficult to analyze. Analytical techniques for prediction are non-effective, and remediation judgments are typically made with on-site engineering judgment of an experienced specialist, who must then balance the risk in terms of probability of failure and consequence of failure, against the cost of effective remediation. The use of empirical design and rock mass classification become important (Franklin and Maerz, 1996).

Even though no analytical tools are available for this task, other tools are available for the practitioner. One example of this is the use of empirical design and rock mass classification, as for an example the Oregon RHR (Rock fall hazard rating)) system. The Colorado rock fall simulation program (CSRP) can be used to assess the risk of falling, rolling and bouncing rocks, and to design slope angles, breaks, and ditch dimensions to mitigate the consequence of falling, bouncing and rolling rocks.

Other new technologies are available to assist in the analysis and design. Imaging software such as WipJoint can be used to analyze the structure and block size of the rock. Computer analysis, using instrumented cameras, using oblique images will in future be able to measure or estimate slope heights, slope angles, loose rock quantities and ditch capacities to help identify optimum remedial measures and help with construction specification quantities.

Empirical Design and Rock Mass Characterization

Empirical design is a design methodology that does not use formal design methods, and calculations or analytical equations or modeling or such. Instead it relies on experience and judgment of the engineer. When applied to rock cuts, an experienced engineer with considerable expertise in a particular geological terrain, will be able to use his experience to drive the design

But what of the inexperienced engineer, or the experienced engineer in an unfamiliar geological terrain? Clearly the engineer needs to rely on more than his own experience. This is where rock mass classification comes in.

Classification is defined as the formal arrangement of attributes in a hierarchy. In the case of rock engineering this means collecting data and classifying the outcrop in some meaningful way, based on parameters that are both easy to measure or classify, and are useful as predictors of rock behavior. Examples of such parameters would include block size and rock strength.

The realization of empirical design that uses not only individual experience, but also the cumulative experiences of many comes from the following principles.

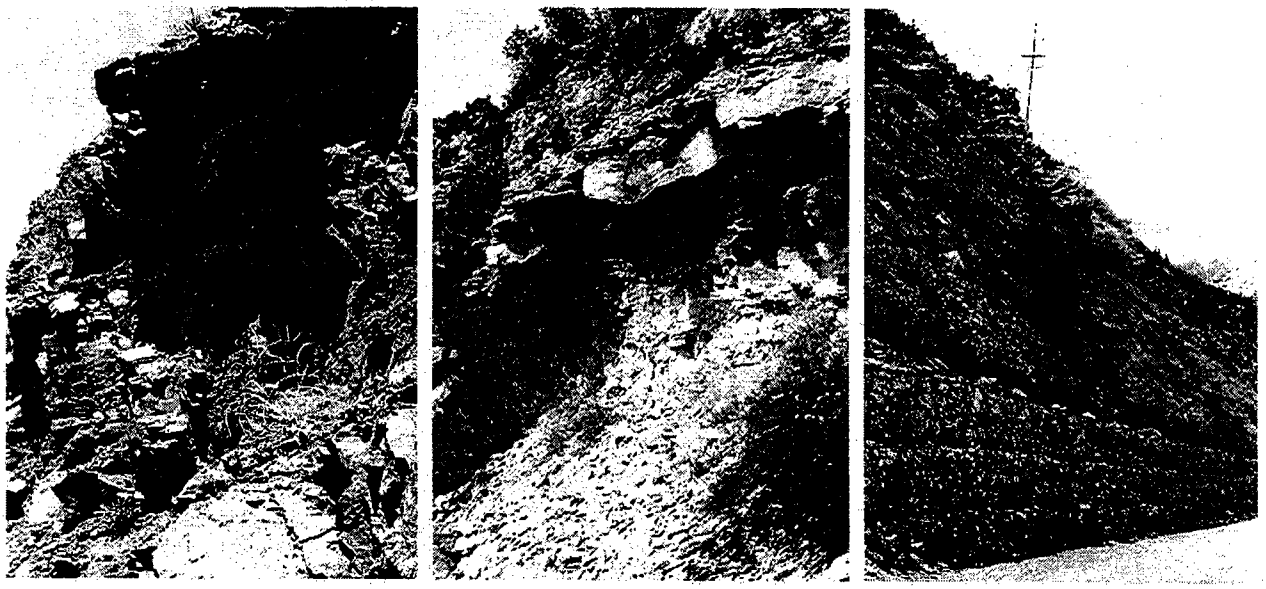

Figure 4. Raveling, undercutting, and rolling failures along road cuts.

1. Description of **ground quality** by a quantitative classification system, based on parameters that are easily and universally measured.

2. Description of **ground performance** by a formal set of parameters (unsupported stand time, support requirements, bearing capacity, ease of excavation, etc.).

3. Correlation of the above 2 based on a broad spectrum of case histories, based on local or global experience.

Design schemes like this are common in the mining and tunneling industries, and are described in Singh and Goel (1999). Examples of such classification systems that include elements of design includes Deere's RQD (rock quality designation) system (Deere et al., 1969), Franklin's Size-Strength system (Franklin, 1986), Franklin's Shale Rating System (Franklin, 1983), Bieniawski's RMR (rock mass rating) system (Bieniawski, 1984), and Barton Q system (Barton et al., 1974).

In addition there are several schemes for slopes. Romana's SMR system is for rock slopes, based on Bieniawski's RMR system (Romana, 1985). The Oregon RHR (rock hazard rating) system is designed specifically for highways cuts ((Pierson and Van Vickle, 1993). The Ontario RHRON (Rock Hazard Rating ONtario) system is a modification of the Oregon system (Franklin and Senior, 1997b).

Basic Rock Mass Classification - RMR

Bieniawski's rock mass rating (RMR) was originally developed for tunneling (Singh and Goel, 1999; Bieniawski, 1998). In this system 5 basic parameters are measured or estimated in the field:

1. Uniaxial compressive strength of the intact rock,
2. RQD (rock quality designation),
3. Discontinuity spacing,
4. Condition (weathering) of the discontinuities,
5. Groundwater conditions.

Each of these parameters is rated with a value typically between 0 to 20, 0 to 15, or 0 to 30 depending on the case. The sum of these 5 ratings is the basic RMR rating, which is an evaluation of rock quality without taking into affect the orientation of the discontinuities. This sum is then modified by a negative rating for discontinuity orientation. Discontinuities are rated as favorable or unfavorable in terms of strike parallel/perpendicular to tunneling direction, dip with/against direction of tunneling, and dip angle. Ratings range between 0 to –12 for tunnels.

The final RMR rating is then used to determine both unsupported stand-up time and support requirements, as a function of opening span, based on an empirical database.

Rock Mass Classification Applied to Slopes - SMR

Romana's slope mass rating (SMR) builds on the RMR system by incorporating adjustments to the basic RMR rating (Singh and Goel, 1999):

$$SMR = RMR_{basic} - (F_1 \bullet F_2 \bullet F_3) + F_4$$

where:

$$F_1 = (1 - \sin(\alpha_s - \alpha_j))^2$$

where α_s is the strike of the slope face and α_j is the strike of the critical joint (discontinuity) plane, and:

$$F_2 = \tan \beta_j$$

where β_j is the joint dip angle with respect to planar failure mode, and F_3 is a rating between 0 and –60 based on the relationship between the slope face and the dip of the discontinuity.

While these three parameters deal primarily with sliding and toppling failure mechanisms, the F_4 parameter is an adjustment factor dealing with the method of excavation (poor blasting –8, normal blasting or mechanical excavation 0, smooth blasting 8, pre-split blasting 10, and natural slopes 11).

As with RMR, the SMR value ranges between 0 and 100, and design charts can be used to infer stability, type of failure potential, and a crude estimate of the probability of failure. Suggested support designs are also available.

Rock Mass Classification For Rock Falls – RHR

The rock fall hazard rating system (RHR) was designed to proactively address the issue of rock falls for road cuts and rail lines (Pierson and Van Vickle, 1993). The system goes much further than other classification systems, in addition to looking at material properties, taking into account such diverse aspects as rock fall history, volumes of material that might fail, and the capacity of existing mitigation measures to contain that volume of rock, in a quasi-probabilistic manner. In addition, the system is useful as a screening technique, allowing high-risk slopes to be quickly identified by a preliminary rating.

The preliminary rating consists of an assessment of the slope into one of three categories, A, B, C (high, moderate, low) for two considerations: 1) Estimated potential for rock fall on roadways, and; 2) Historical rock fall activity. Typically only slopes with preliminary ratings of A are given detailed ratings.

The detailed rating system uses 10 categories with 4 nominal rating criteria and scores, although interpolations of scores between criteria are allowed. The scoring is a power progression, with score

$y=3^x$ where x is a rating between 1 and 4 and allows scores of 3, 9, 27, and 81 respectively. The following are the categories:

1. Slope height (25, 50, 75, or 100 feet),
2. Ditch effectiveness (good, moderate, limited, or no catchment),
3. Average vehicle risk (vehicle present in rock fall section 25, 50, 75, or 100% of the time),
4. Sight distance (100, 80, 60, or 40% of stopping distance when viewing a 6" object),
5. Roadway width (44, 36, 28, or 20 feet including shoulders),
6. Structural condition *discontinuous rock* (discontinuous joints- favorable orientation, discontinuous joints – random orientation, discontinuous joints – adverse orientation, or continuous joint – adverse orientation),
7. Rock friction (rough –irregular, undulating, planar, clay infilling or slickensided),

or

6. Structural conditions *eroded rock* (few differential erosion features, occasional erosion features, many erosion features, or major erosion features),
7. Difference in erosion rates (small, moderate, large – favorable structure, or large – unfavorable structure),
8. Block size/volume of rock fall event (1/3, 2/6, 3/9, or 4/12 ft/cubic yards),
9. Climate and presence of water on slope (low to moderate precipitation; no freezing periods; no water on slope, moderate precipitation or short freezing periods or intermittent water on slope, high precipitation or freezing periods or continual water on slope, or high precipitation and long freezing periods or continual water on slope and long freezing periods,
10. Rock fall history (few, occasional, many, or constant falls).

All the categories are then added up, and the highest scores are deemed to have the highest priority in terms of remediation.

Rock Mass Classification For Rock Falls – RHRON

The rock fall hazard rating system, Ontario, (RHRON) is a modified version of the RHR system. It attempts to address the overemphasis on high slopes and large volumes that occur as a result of the power relationship between rating and score (Franklin and Senior, 1997a). In Ontario, with its relatively lower slopes than locations like Oregon, the RHR system was just not sensitive enough. In addition, five new parameters were added, and several parameters were re-defined.

The basic formulation for RHRON is:

$$RHRON(\%) = (F1 + F2 + F3 + F4) * \frac{100}{36}$$

where:

F1=Magnitude: "How much rock is unstable?",
F2=Instability: "How soon or often is it likely to come down?",
F3=Reach: "What are the chances of this rock reaching the highway?",
F4=Consequence: "How serious are the consequences of the blockage?"

For the *preliminary screening*, each of these F factors is directly rated on a scale of 0 to 9. For the *detailed rating*, each of these F factors are calculated from a number of individual ratings also on a scale of 0-9:

$$F1 = \frac{(R2 + R3 + R4))}{3}$$

$$F2_{ravelling} = \frac{(R1 + R9 + R11 + R4 + R5 + R6))}{6}$$

$$F2_{sliding} = \frac{(R1 + R9 + R11 + R5 + R6 + R8))}{6}$$

$$F2_{erosion} = \frac{(R1 + R9 + R11 + R4 + R7 + R10))}{6}$$

$$F3 = \frac{(R4 + R13 + R16 + R19)}{4}$$

$$F4 = \frac{(R17 + R18 + R19))}{3}$$

where:

R1=History of rock falls
R2=Volume of the largest potential rock fall
R3=Volume of total potential rock fall
R4=Face irregularity
R5=Face looseness
R6=Joint orientation/persistence
R7=Rock intact strength
R8=Rock joint shear strength
R9=Block size
R10=Slake durability
R11=Water table height
R12= Slope height
R13=Crest angle
R14=Ditch and shoulder width
R15=Ditch capacity
R16=Overspill potential
R17=Average vehicle risk
R18=Decision sight distance
R19=Available paved width
R20=Remediation cost

Not all ratings (R) are used in all cases and some ratings are used for different purposes entirely such as calculating cost-benefit ratios.

Rolling and Bouncing Rock - CRSP

Having identified slopes with significant failure potential, the consequences of the failure must be evaluated. One aspect of this evaluation has to be an analysis of the kinetics of the bouncing and rolling rock. Will the falling rock reach the highway? What effect will slope angles have on this? Will the rock bounce or roll into the catchment ditch and out the other side?

One way to assess this is to do some tests to evaluate this. If the roadway below a slope is closed, it is a simple matter to select 100 typical broken rock fragments and roll them from the top of the slope and record how far towards the road they travel. Of course it is not always possible to close the road below.

An interesting alternative is the Colorado rock fall simulation program (CSRP) (Figure 5). This program uses slope and rock geometry and material properties, calculating falling rock bounce height, velocity and travel distance.

While this can be used to predict the amount of road material that will reach the road, its most useful application is probably to help with design of slope angles, break in slope angles and, ditch geometry. Because rounded rocks rolling gather a great deal more energy than angular blocks sliding, the results, in terms of predicting the amount of rock reaching the roadway, should be used with caution. In addition, CRSP requires the input of a number of coefficients, which are best input empirically, based on calibrating with actual bouncing and rolling tests.

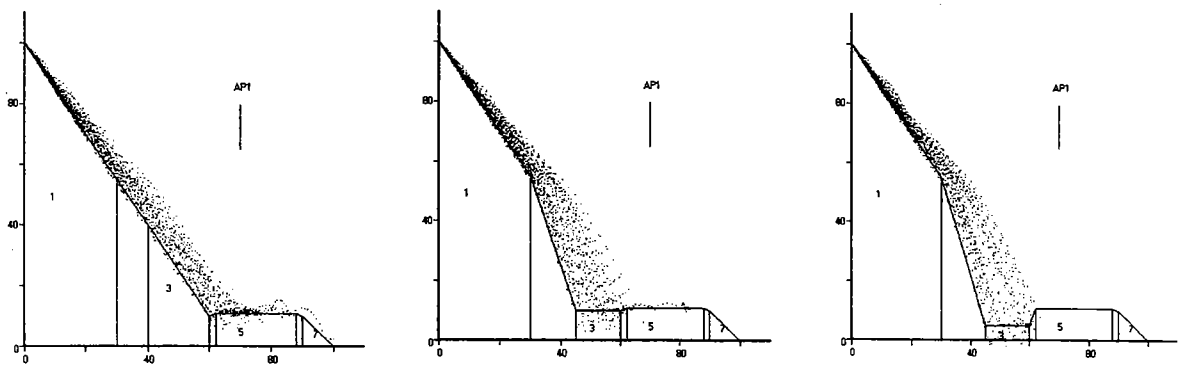

Figure 5. CRSP simulation for 100 blocks 0.4' in diameter bouncing and rolling down a slope to a roadbed. The simulation reveals that for the original geometry (left) 14 blocks reach the road surface; if the slope is cut back (center), 2 blocks reach the road surface, and if a ditch is put in place (right), no blocks reach the road surface.

Automated Measurements - WipJoint

A cursory survey of the different ratings of RHRON shows that a significant number of the parameters upon which these ratings are based are items where quantities are measured or estimated. Examples of these include slip heights, ditch capacities, block sizes, block volumes, and face irregularities. Some of these items are conducive to automated measurements such as can be done using image analysis (Franklin and Maerz, 1988; Maerz et al., 1997).

An example of an image analysis package that does exactly this is WipJoint, designed to measure blocks size and joint fabric images of rock cuts (Figure 6).

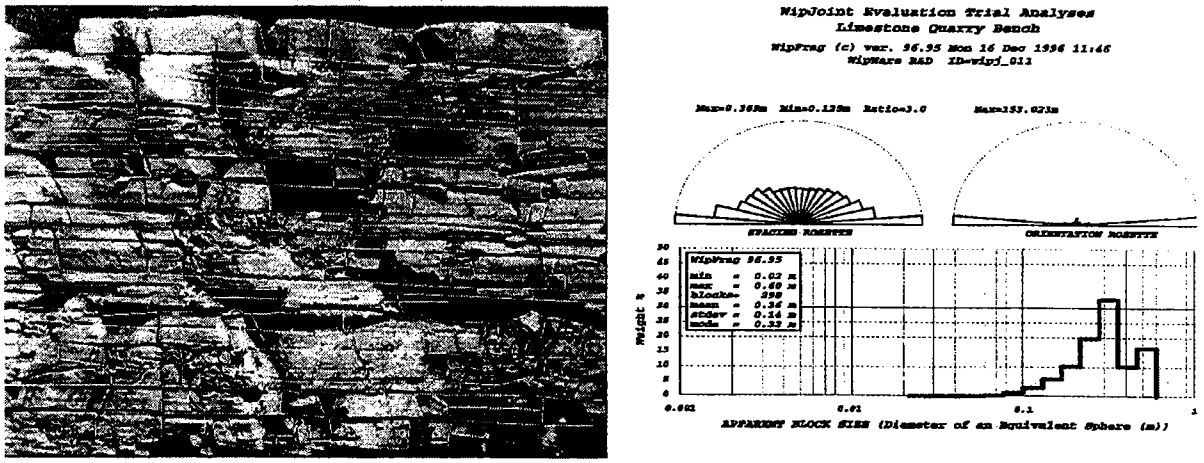

Figure 6. WipJoint image analysis of a rock cut.

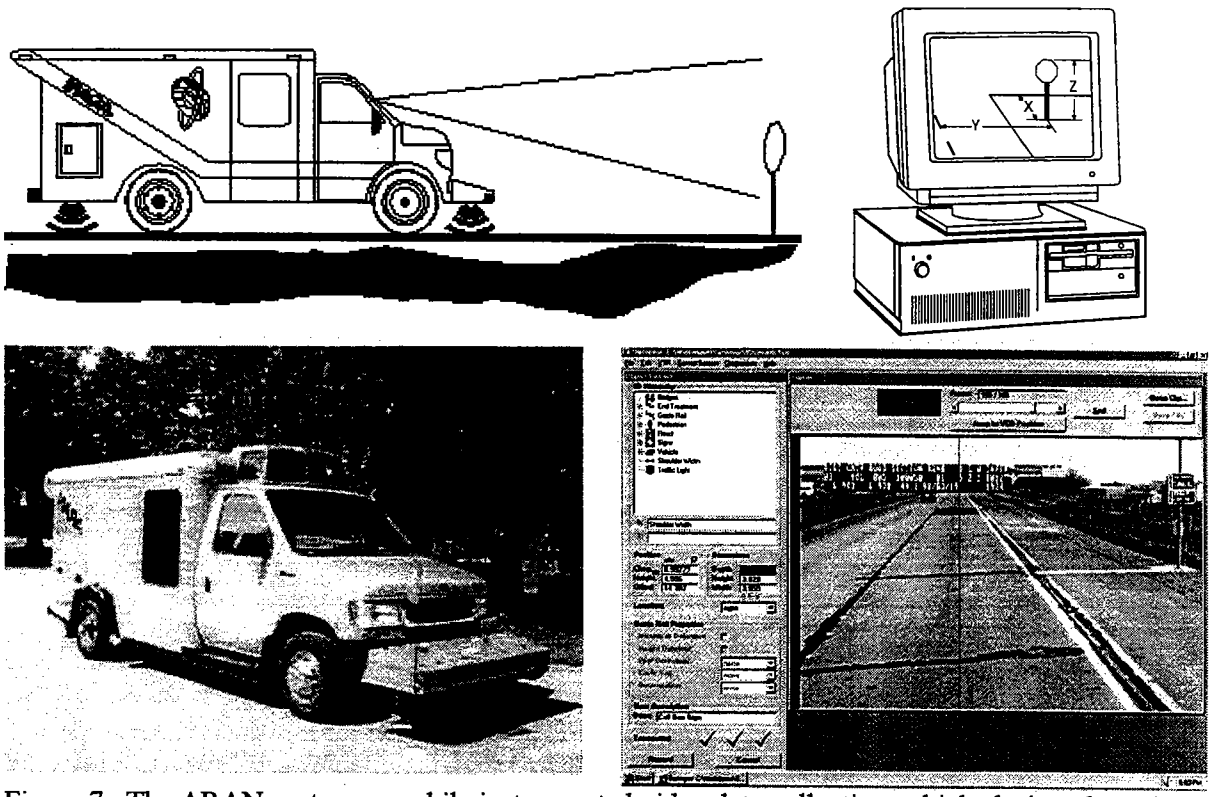

Figure 7. The ARAN system, a mobile instrumented video data collection vehicle designed to take video images at highway speeds and allow users to make measurements and create roadside inventories (Maerz and McKenna, 1999).

Future Trends - Rapid Screening and Mobile Automated Measurements

Future trends in automated measurements of rock cuts may come in the form of a mobile data acquisition system. An example of such a system is the ARAN (Automatic Road Analyzer) Surveyor System (Figure 7) (Maerz and McKenna, 1999). This system was designed to allow measurements of object dimensions, and inventorying of those dimensions. A user identifies objects and distances to be measured using mouse clicks on a computer monitor showing the video taped images. The computer then makes all necessary calculations, possible because each video frame has an associated data set that corresponds to the camera position, attitude, speed, and orientation.

This type of data acquisition would open entirely new possibilities with respect to collecting data for rock cuts.

In the first place, although video logging is not entirely new, it allows initial screening of the cuts. Often engineers spend all too much time screening road cuts, deciding which need to be further evaluated and which do not, rather than focusing on those that are clearly in need of work A further benefit is that in addition to being a screening tool, this would provide a permanent record of the condition of the road cuts at a given point in time.

Second, the measurement capabilities can be used to measure estimated volumes of rock, slope heights, slope angles, ditch capacities and land and shoulder widths.

Finally hard copies of the video images can form "working diagrams" (Figure 8), complete with GPS (global positioning system) coordinates for establishing correct locations.

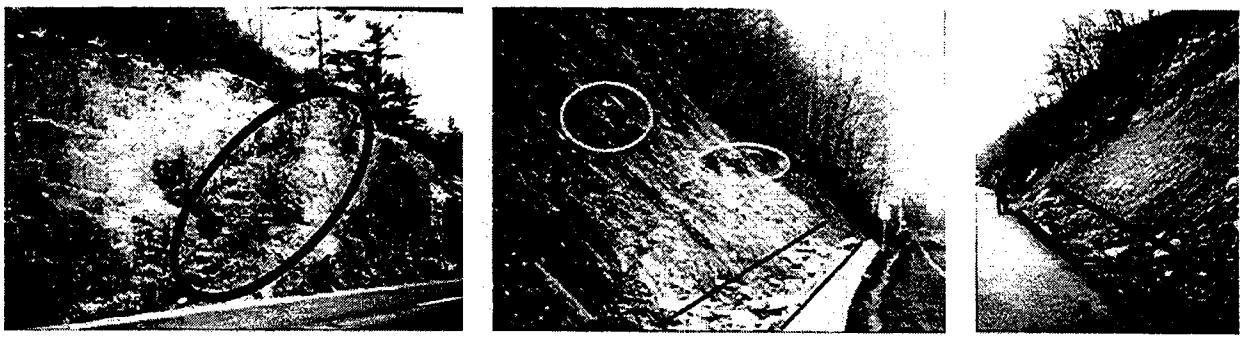

Figure 8. Simulated images demonstrating the potential to develop mobile vision based data acquisition systems for rudimentary measures such as slope heights, slope angles, loose rock quantities and ditch capacities

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Monitoring an Unstable Sandstone Cliff And Removing Rock Materials Utilizing Underground Mining Excavation Techniques

State Route 264, Tuba City – Window Rock Highway
Coconino County, Arizona

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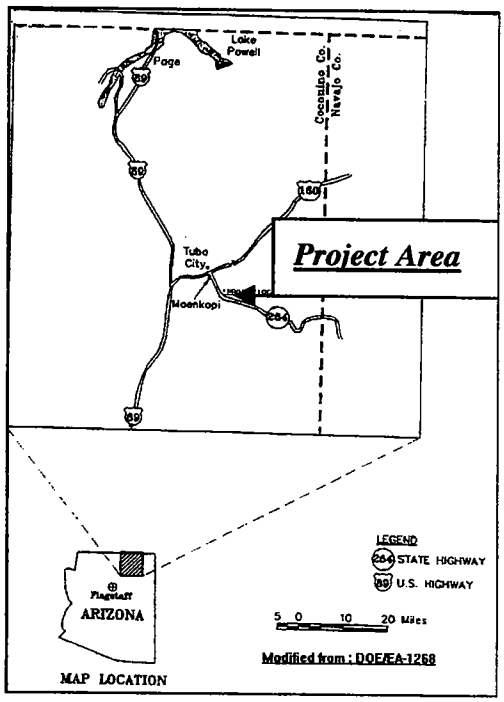
Frank Montonati P.E. *(Mining Engineer, MCO Mine Services, Prescott AZ)*

BACKGROUND



Location

The project area is locally referred to as the Moenkopi Wash Cliffs, and is located on the north east side of State Route 264 in the vicinity of milepost 324. This site is approximately 70 miles northeast of the town of Flagstaff Arizona, and 3 miles southeast of Tuba City. Generally the project area borders the agricultural lands of the Hopi Indian community's Moenkopi Village.

Physiographically the area is located on the Colorado Plateau (Figure 1) which is " predominately characterized by horizontal stratified sedimentary rocks that have been eroded into numerous canyons, plateaus and scarps along which are exposed many colorful rocks ranging in age from Precambrian to Cenozoic." (Nations and Stump , 1981). "The plateau country landscape is characterized by low relief, wide mature valleys, and scarps that develop on the beds of contrasting resistance and retreat down structural slopes."(Lucchitta I., 1990)

Climate and Vegetation

The climate in the vicinity can be described as arid. Normal annual precipitation is about 8 inches per year, with the average annual pan evaporation ranging between 85 and 90 inches per year. Temperatures range from, near 100 degrees in the summer to 20 degrees in the winter. The area has very little natural vegetation over most of the area. However seeps (low volume spring) that flow from the base of the Navajo Sandstone support a luxuriant growth of shrubs., This green band makes an excellent stratigraphic marker. The springs are tapped by the local inhabitants to irrigate their traditional seasonal crops.

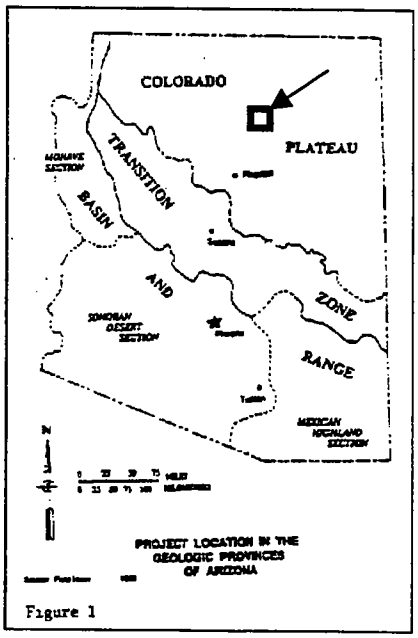


Figure 1

Highway Corridor and Geometry

Archival information indicates that the Bureau of Indian Affairs constructed the present alignment as an improvement to a pre-existing route in 1956. Generally, the route cuts through and descends against the topographic contours that describe the Moenkopi Cliffs. The elevation changes is approximately 180 feet within the 2.1 miles of highway This stretch of sinuous, steep graded, two lane highway is located between the intersection of SR 264 and SR 160 and the bridge crossing at Moenkopi Wash. This route is the Hopi Indian community's principal all-weather access to the hospital, high school and fire station in Tuba City, on the Navajo Indian Reservation, 3 miles northwest.

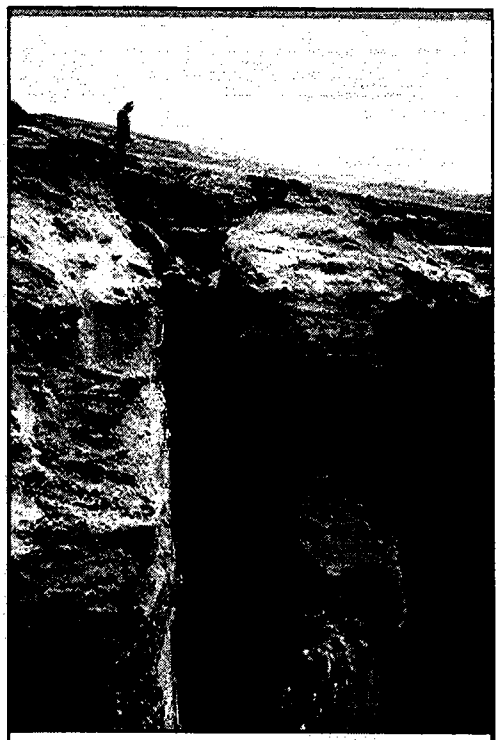
Highway road cuts to accommodate construction of the improvements of the alignment were generally ½ to 1 in rock and 3 to 1 in soils. Embankments were constructed of existing site materials, (principally blow sand and waste rock) with slope rates that range form 1 to 1, to 4 to 1. The 1956 construction effort improved the alignment, however some of the steepest cuts have narrow shoulders, which typically are approximately two (2) feet wide.

Aerial Photo displaying SR 264 alignment, and part of the Moenkopi Cliffs

PROJECT HISTORY

This project began as a response the district maintenance foreman and the Hopi Chairman concerns over a precariously positioned slab of sandstone. The slab appeared likely to detach itself from the cliff face and fall onto the road shoulder and paved section. Upon climbing the slope and to inspect the slab, additional concerns were observed a short distance from the original problem. Sub-parallel to the cliff face and along a 300-foot length, many en-echelon joints had effectively detached 15 foot thick, 35-foot long, (approximate dimensions), sandstone blocks. Joint spacing occurred at intervals approximately 10 to 15 feet behind the existing slope. Initially the joint spacing between the first block and the second was approximately 12 to 48 inches. However the first block had a definite inclination to topple onto the highway below. Differential erosion of the lower portion of the cliff appeared to have been accelerated by the recent change in a natural drainage channel above the cliffs.

Up to this time the district maintenance forces reported no unusual amount of rock fall. However there was



Gaping en-echelon joint and adjacent potentially unstable block. State Route 264 approximately 125 feet below.

recent mud debris removal work at the base of the potentially unstable blocks. Staining on the cliff and residual materials in the cut ditch reinforced the idea that the sandstone blocks had recently been exposed to increased amount of deterioration from rain runoff.

A tape measure was used to set some swing ties between the joints to get an approximate area and volume of the area of concern. The site was immediately photographed from re-locatable vantagepoints, which were not susceptible to movement. Aerial photography was ordered to establish a baseline of rock movement. A joint-monitoring program was implemented using triangular sets of swing ties and tell tales crossing susceptible joint intersections. A geologic site characterization was initiated.

After examining aerial photography from 1964, it was observed that the jointing behind the unstable blocks had been in place for some time. These features were probably in existence during the 1956 construction efforts. Although the slopes appeared to be deteriorating, the rate of deterioration appeared to be gradual. It was decided to continue monitoring the slope so additional analysis could be performed.

The site was continuously monitored with a variety of low-tech methods from 1994 through 1997. In the fall of 1997, suspected movement of several large sandstone blocks suggested that a large rock fall was imminent. At that time a rock removal contract was awarded and the apparent hazard removed in a controlled manner.

GEOLOGIC INVESTIGATION

PREVIOUS STUDIES

No previous engineering investigation was known to exist for the site. The area had generally been studied by the US Geological Survey, (Harshbarger, et all 1957, Cooley et all, 1969). Blakey & Middleton (1986) had described the highway cuts in a GSA field trip guidebook. Kiersch et all, 1995, mentions the terrace deposits adjacent to the site and described the quarrying characteristics of the Navajo Sandstone. The U.S. Department of Energy (1998) has conducted an environmental assessment of a Uranium Tailings Site near Tuba City.

General Rock Formation Descriptions
Abstracted From: **Harshbarger et all 1957**

> ***Kayenta Formation***
>
> *In the Tuba City area the Kayenta Formation is represented by a silty facies. (This material) differs markedly from in outcrop character from its (sandstone) type section. In the Tuba City area the Kayenta formation conformably overlies the MoenaveFormation. It inter- tongues with the basal part of the Navajo sandstone.*
>
> *At Moenave (5.2 miles NW of Moenkopi) the silty facies of the Kayenta Formation consists of 495 feet of fine-grained sandstone and mudstone. Most of the sand grains are subrounded to subangular. The bedding is lenticular, and crossbedding is common in the sandy units. Much of the mudstone occurs as flat, thin-to very thin-bedded units. Locally thin limestone ledges occur in the upper*

part of the member. The entire unit is banded pale red and grayish red, but the overall color more nearly approximates pale red. The silty facies weathers into irregular badlands. The outcrop area is marked by rounded hills and earth pillars (demoiselles) having resistant caprocks.

Navajo Sandstone

The Navajo sandstone ranges from very pale orange to pale reddish brown. It is composed of medium- to fine grained, subrounded quartz grains and in most places is bounded with weak calcareous cement. It contains many small lenticular beds of cherty limestone. The beds form conspicuous ledges (in the area) between Tuba City and Hopi Buttes.

Eolian crossbedding in the Navajo Sandstone is one of its most conspicuous features. It is typically large scale, high angle and of the trough type. Characteristically it weathers into rounded hills and domes in which the conspicuous crossbedding is etched out by erosion. Across vast areas of the Navajo country it weathers into a very rough surface and is dissected by deep, sheer-walled canyons. Much of the outcrop area is completely devoid of soils and vegetation. The formation has a remarkable consistent lithology throughout the Navajo country

The field relationships of the site and the abstracted geologic reference material information suggests that the eolian dune sands form the Navajo Sandstone intercalate (inter-tongue) with the fluvial deposits of the Kayenta Formation. Jurassic age sand dune encroachments may have created temporal dams and backwaters in which the fine gained fluvial materials were deposited. According to Blakey & Middelton, (1986) "fluvial/eolian cycling" may have occurred "4 to 5 times" locally.

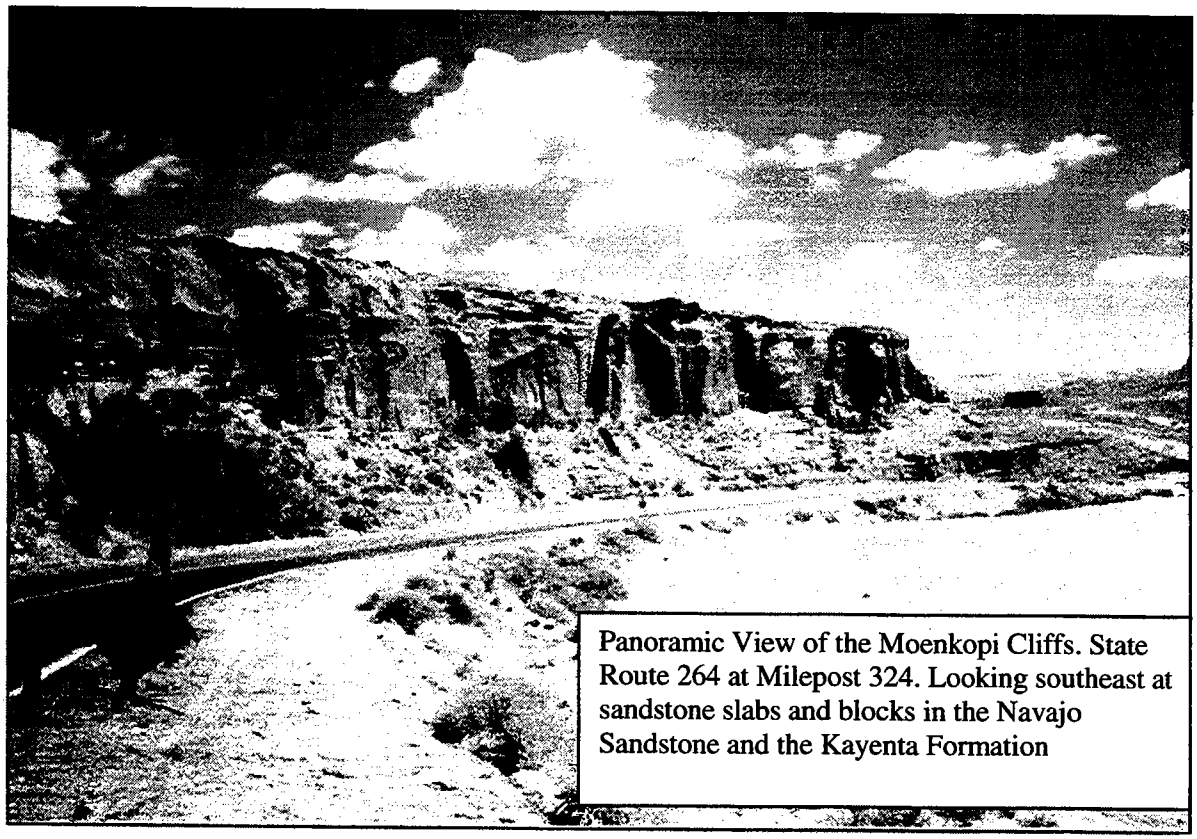
In the project area State Route 264 descends through a sequence of *en-echelon* jointed, eolian Navajo Sandstone with dramatic high angle cosets texture. This unit is inter-tongued /intercalated with fluvial mudstone and silty sandstone, of the Kayenta Formation. At the site a thin-bedded clay-stone mudstone separates the two units.

Hydrogeology

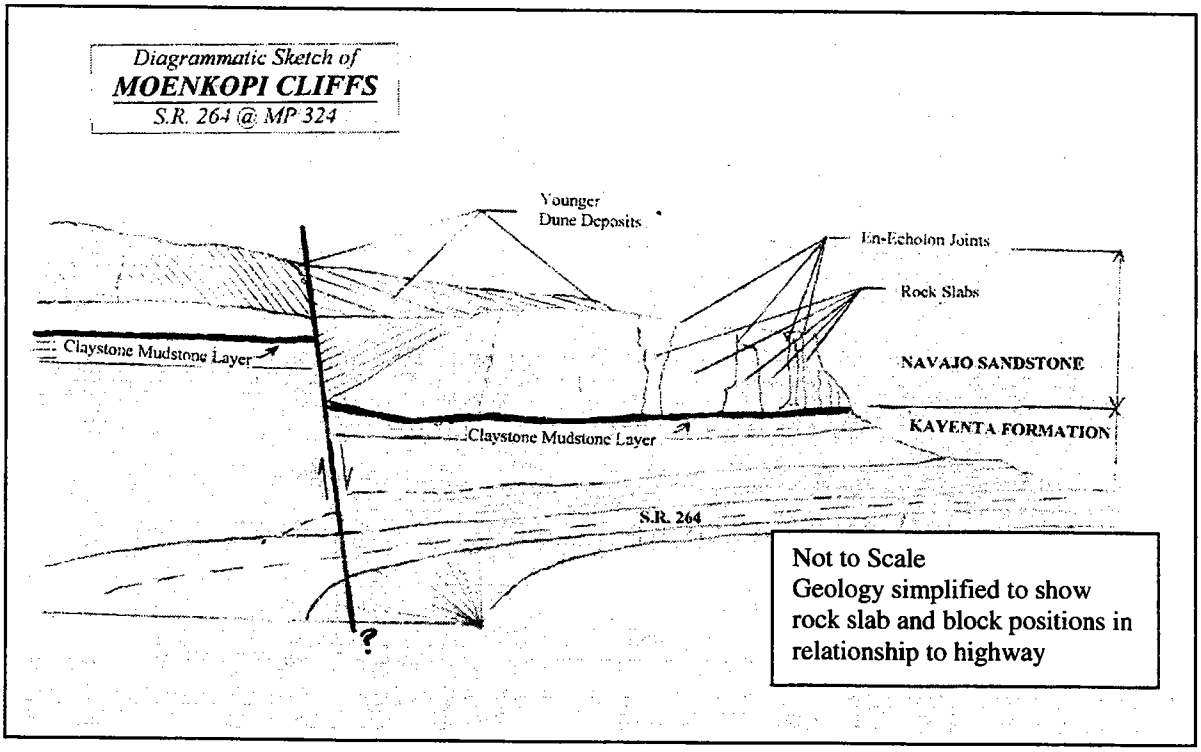
The intercalated/inter-tongued zone has been described by Cooley et all 1969, and assigned to be part of the "N-aquifer" one of the "chief aquifers in the region" Notably this aquifer system "yields small to medium amounts of water to springs and shallow water wells in the Tuba City area".

The U.S. Department Of Energy (DOE1998) states that, " The N-aquifer, regionally obtains its recharge from rainfall and snowmelt throughout the 1,400 square-mile area where the Navajo Sandstone is exposed." Koski (1996) referencing Museum Notes, (1931) indicates that the "large mass of sandstone exposed on the Kaibito Plateau" is the recharge area for the aquifer. Groundwater is then conveyed by a system of "clay layers" down gradient to the Tuba City area.

"The Valley of Moenkopi Wash is an important regional discharge location for the N-Aquifer. The discharge occurs as evapotranspiration, direct discharge to springs and seeps along the axis of Moenkopi Wash, and discharge to Moenkopi alluvium. Annual discharge to Moenkopi Wash from the N-Aquifer is estimated to be approximately 3,800 acre feet"(DOE 1998).



Panoramic View of the Moenkopi Cliffs. State Route 264 at Milepost 324. Looking southeast at sandstone slabs and blocks in the Navajo Sandstone and the Kayenta Formation



As previously mentioned the seeps allow dense vegetation to grow out of the side of the near vertical Moenkopi Cliffs.

In the project area, a distinctive green band marks the position of the lower portion of the Navajo sandstone and part of the N-aquifer system. It can be traced with confidence to within 500 feet of the sandstone blocks. At this location, seeps (springs) issue water from the Moenkopi Cliffs. The vegetative growth continues towards the project area, diminishing in intensity. This decrease in phreatophyte numbers suggests a smaller quantity of water from the regional aquifer is present in the project area, although some ground water may make its way to the site.

Joint Orientations and Site Stratigraphy.
Most of the bedrock in the project area are filled and covered with recent eolian sand. This fact strongly suggests that these joints may have originated from events that occurred long ago.

Macro joint and lineament trends are observable in the immediate area. By using low altitude aerial photography features with strong linear trends can be recognized. Two sets of dominant trends were noted in the inter-tongued sandstone units exposed in the project area.

The most prominent trend was exhibited by joints that were oriented between a narrow range of N 41°- 48° W. These joints appeared to principally occur sub-parallel to the existing cliff face. The second most prominent joint trends were orientated approximately perpendicular to the dominant set and ranged between N 48°– 52° E. These orientations appear to be similar to those reported to occur in the southern Kaibito Saddle region by Kelly & Clinton (1960).

Dip and dip direction for most of the joints were roughly perpendicular to strike and vertical however some the exposed dips varied in a wavy circuitous direction within a few feet of any observation point. The overall effect was to produce joint surfaces that exhibited the exfoliation and arching texture that is typical for sandstones of the Colorado Plateau. However, some of the exposed cliff surfaces appeared to exhibit fracture surfaces roughly suggestive of hackel and rib marks. Many of the joint surfaces behind the cliff face did not appear to be fresh. In fact, the tops of some of these discontinuity surfaces appeared to be eroded back from their original plane of failure.

Most of the joint surfaces terminated abruptly approximately 35 feet below the top of the sandstone cliff. At this location, a thin, shallow dipping, differentially weathering, clay-mudstone (shale) is present. Some of the joints persist approximately 10 more feet into another durable sandstone. At the base of this ledge, all observable joints appear to terminate in weak silty clayey sandstone. This material forms a sloping bench below the steepest part of the cut and supports intermittent vegetation growth. The bench is approximately 25 feet in thickness and overlies another more durable sandstone bed near highway grade. This intercalated assemblage of fluvial and eolian sandstone is believed to be part of the "N-aquifer previously described by Cooley et all (1969).

The intercalated materials and geologic history of the site has produced a rock cliff that has reacted differentially to stress, water movement, and erosion. The net result is an upper sandstone

unit that consists of adversely oriented, rock slabs and blocks. These blocks are delineated by wide spaced vertical joint sets and bedding plane discontinuities. These conditions are conducive to the formation of toppling, block sliding and wedge rock fall failures above the highway.

SITE MONITORING

Aerial photo investigation revealed that the site conditions, although very alarming to observe had been deteriorating in place for over 30 years. Consequently, it was decided to continue to monitor the site until sufficient resources were found to mitigate the apparent hazard. Several simple techniques with a monthly inspection program were initiated.

METHODS

1.Triangular Joint Intersection Measurements

Susceptible joint intersections were identified in the field. At the most critical joint intersections, triangular sets of swing ties were measured with a steel tape between benchmarks, the tape held horizontal so those true distances were measured. The triangle was set so each leg crossed, (roughly perpendicular) to the trend of the joint. Each horizontal measurement was recorded with the date on which the observations were made. Additionally, the angle that sub-tended the two measured lengths was calculated. The three angles of the included triangle should add up to approximately 180 degrees if the measurements are correct The highest (most stable) corner of the triangle became the vertical benchmark for the three point set. In order to confirm relative movement, a hand level could be used to measure the relative change in vertical elevation between outcrop apices. The joint intersections were photographed (from stable vantagepoints) every time a joint movement was suspected.

This method presented some practicable problems. The hazardous appearing rock slabs had the effect of intimidating some of the personnel sent to take the measurement. Initially resulting in erroneous measurements

Another problem, which plagued efforts throughout our investigation, was vandalism. Many of the benchmarks were disturbed or removed and the benchmarks had to be re-established.

2.Remote Sensing Using Aerial Photography

An attempt was made to establish tell tales with (12 inch wide) rigid plastic panels. A "T" panel was constructed across a suspected joint. The top of the T was oriented parallel to the joint opening and secured with PK nails. The bottom leg was installed perpendicular to the joint surface and secured, only on one side. (The joint side opposite of the first panel.) This bottom leg bridged over the joint opening and overlapped the center of the first panel. A PK nail marked the center of the top,"T" panel, and the bottom leg panel was trimmed to point to the nail. Four additional panel points were established along the centerline of SR 264 at 100-foot intervals. These points established a scale to which subsequent aerial photos could be compared to.

Scheduled aerial photography was not deemed a cost-effective method. However ADOT aerial services routinely traversed the area. Initially the concept was to have a flight crew in the vicinity take low altitude photographs using the tell tales as targets.

Relative (on-site) movement would still have to be observed at the center of each "T" target, every month. However, we would develop an incremental, scalable, photographic record of the entire site. When instability was suspected, these photographs could be compared with photographs previously taken. In this way, rock slabs and blocks, not easily accessible from the top of the slope could be inspected safely.

Unfortunately the plastic T panels were not durable enough to survive many seasons, and the time between aerial flights extended from 6 months to 1 year, thus very few photographs were obtained

3.Joint Gap Measuring Devices (JGMD)

The majority of the anticipated rock failures involved an initial component of movement away from the cliff. This component was also the easiest and safest movement to track with a static device. Many of the joint spaces between blocks were 12"or more apart. Therefore, a joint gap-measuring device (JGMD) was fabricated to measure apices between 1-4 foot widths. The extended lengths would allow measurements from a point of relative safety when movement along the joints occurred.

Joint Gap Measuring Device
Target and Thread bar point on fabricated base

A standard thread bar was pointed and mounted on a fabricated steel frame to support the bar. The frame was installed perpendicular to the axis of the joint being monitored. A target was fabricated with an adjustable base so it could be mounted across the joint and parallel to it. The point of the thread bar was screwed into the target so that a mild pressure was applied to it. (approximately 5psi) A soft, impressionable surface was applied to the target plate so that any minor movement in the plane of the joint could be recorded, by a scratch mark on the target. Any movement away from the target could be measured with a tape or feeler gauge. Measurement across the joint could also be made by counting the number of threads of additional lengthening necessary to close the gape to the relocated target. This device does have limitations in that not all components of movement can be observed.

One case of vandalism of the JGMD did occur, however the device was recovered and is reusable. Painting the JGMD with a reddish brown primer paint obscured its presence at Moenkopi Cliffs.

Monitoring Results

The site was intermittently monitored from 1994 through 1997. During most of this time, little (if any) movement was observed. At one point, observations were re-scheduled to occur every other month, and consideration was given to observations every six months. No correlation had been found between rainfall events, the N- aquifer, and joint gap widening. However, it was suspected that rainfall was accelerating deterioration of the highest mudstone layer (at the top of the Kayenta Formation) and eroding the blow sand in the joint spaces.

In the fall of 1997 subtle changes in the sediments adjacent to joint intersection were apparent. The level of sediment filling the joints had lowered and surface materials adjacent to the joints had been disturbed or removed. A check on the orientation of the suspected block indicated that it might have leaned out an additional 5 to 10 degrees from its original 9-degree tilt (towards the highway) position. Movement of this large sandstone block suggested that a significant rock fall might be imminent. Movement had not been observed at all the monitored joints; however, removal of the unstable block would probably accelerate failure of the adjacent slabs. The decision was made to remove the unstable blocks.

ROCK MATERIAL REMOVAL

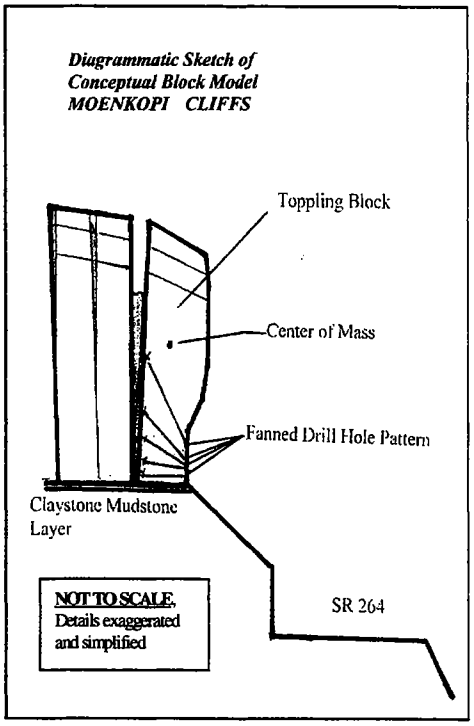
The excavation contractor arrived at the site and his initial impression strongly suggested that slope failure was incipient. To bring down the unstable blocks in a controlled manner would require a great deal of site engineering. It was mutually agreed that as much of the unstable rock material would be removed as possible, hopefully eliminating additional mitigation actions.

The most effective means of handling the site was to break it down into individual problems. In that way, each unstable block would be individually evaluated and a separate blasting plan developed.

A thin shale (claystone –mudstone) bed approximately 8 – 12 inches thick underlies the upper sandstone blocks. The existing joints dominantly terminated at this thin layer. This layer was accepted as the base of the unstable materials. The first en-echelon joint would mark the vertical limit of the material to be removed.

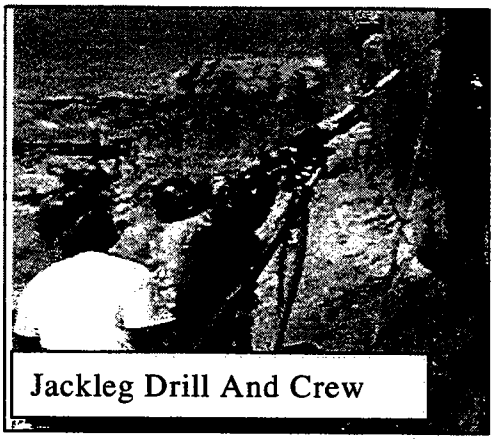
The geometry of each individual block was measured to calculate its rough shape, mass, and its center of gravity. Simple trigonometry was used to create a conservative conceptual model for each block.

After the models were established, a series of individual shot patterns was designed to remove the base of each block. Ideally, charges were positioned below and just past the center of mass for each block to be removed. Using timed delays within each block would cause sequential fragmentation from the center out to the edges of each bock base. The charges were placed to destabilize the block from the cliff, and would fracture it into discreet triangular facets. The faceted blocks would then topple, and further fragment the rock mass, as it rotated off the slope and onto the sand cushion constructed on the paved highway section. The same process was anticipated for the remainder of the blocks, but they would be timed with successive delays so each block would fall consecutively. Additionally, there would be no amplification of the shock wave from the preceding blast.



Access to the site was quite limited. The unstable blocks were located atop a bench approximately 60 feet

above the highway. This steep slope prevented the use of mechanized power equipment. A decision was made to perform the drilling and blasting activities with hand labor. Access was gained by using a ladder to climb the steepest part of the bench. A rope could be used to traverse up the remaining talus slope to reach the shale layer horizon. A Gardner-Denver 83, percussion jackleg drill, was utilized, with a three-man crew, to fan drill the 1.5" diameter blast holes at the calculated angles and depths underneath each block.


Jackleg Drill And Crew

The blasting plan was designed, executed and the debris removed within a ten-day period. The initial blast dislodged over 2,000 cubic yards of material. The debris was wholly confined in the existing, 200 foot wide, right of way. A travel lane was cleared within 45 minutes and controlled traffic flow was available shortly thereafter.

Measuring Block Size And Joint Gap

SUMMARY

Although the site initially displayed characteristics that suggest imminent failure, rock materials at the site had been experiencing deterioration for some time. The wide open joint had in fact been filled with eolian sand suggesting that its formation was not a recent event. Additionally all of the joints did not appear fresh and some were partly eroded.

A regional ground water aquifer is present in the area, but its effects on this portion of the rock cliffs appear to be intermittent. The nearest known seep is approximately 500 feet away.

Differential weathering of the inter-tongued Navajo Sandstone and Kayenta Formation appears to have initiated retreat of the Moenkopi cliffs above highway 264. The Kayenta formation erodes faster and undermines the overlying, more resilient Navajo Sandstone

Surface drainage from the top of the slope appears to have infiltrated the sand filled joint piping the material to the base of the cut. The claystone/mudstone layer appears to have acted to concentrate deterioration of the strata immediately underlying the Navajo Sandstone.

These conditions favor the destabilization of existing, en-echelon-jointed blocks, over a long period of time

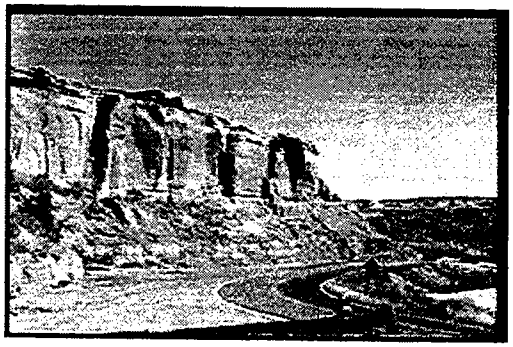
The apparent instability was monitored for four years with triangular joint intersection measurements, low altitude aerial photography, fixed vantage point photography, and joint gap measuring devices. Little movement was observed during this time

The rock materials were removed with underground mining techniques. The contractor used the inherent weakness of the undermined sandstone block to selectively blast it into discrete triangular facets which collapsed onto themselves.

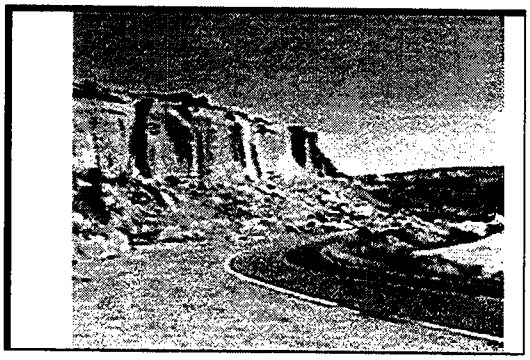
Because of limited and difficult access, most of the drilling was accomplished with hand labor and underground mining equipment.

Over 2,000 yards of potentially unstable materials was removed form a 300-foot section of the Moenkopi Cliffs adjacent to Highway 264, for a cost of roughly $100,000.00.

Pre-Blast Site Conditions



Post- Blast Site Conditions



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The Use of Ring Nets for Flexible Rockfall Catch Barriers

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ABSTRACT

The primary and most significant component of any flexible rockfall protection barrier is the netting material, due to the flexibility and strength requirements. The nets are most often the first component to be impacted, and thus must efficiently transfer the forces to the infrastructure of support ropes, suspensions, posts, and finally to the anchors while incurring a minimum of distortion or damage. Ring nets configured like Olympic rings, and made from robust high grade steel and relatively large diameter wire have proven to be the most effective type of netting for fulfilling these requirements for energy absorbing capacity.

Ring nets have long been available for such purposes in the form of World War II surplus anti-torpedo nets, and until recently, have been reserved for use with very high energy barriers due to the finite supply of the torpedo nets. In recent years, Brugg Cable Products has developed the capability to produce new ring nets, made from high tensile strength steel wire. This type of net has a very high internal elastic energy absorbing capacity and therefore is maintenance-free throughout a large range of impact energies. Performance of these nets has been documented in extensive field testing, supervised and analyzed by independent agencies in controlled field tests, and through a great number of field installations worldwide in the past several years.

Beyond the greatly improved energy absorbing capacity of the nets, other important advantages include far superior corrosion resistance, the ability of the nets to conform to irregular terrain and elevation changes, ease of installation, and ease of repair in the unlikely event of net damage.

A review of system performance and testing will be presented, in addition to support for the described advantages, and several case histories describing specific field applications of these new ring nets.

INTRODUCTION

Since the 1950's, rockfall barriers utilizing flexible yet strong wire rope netting have been developed and have proven to be an effective and efficient alternative for rockfall mitigation. The systems have continually evolved during this time, with improvements in overall design and in individual components, such that ever increasing capacity and decreasing maintenance requirements have been achieved.

Figure 1 shows a typical flexible rockfall catch barrier design, with woven wire rope nets. Each of the system components is designed to emphasize flexibility and energy dissipation, with the netting material usually being the primary point of impact. During a

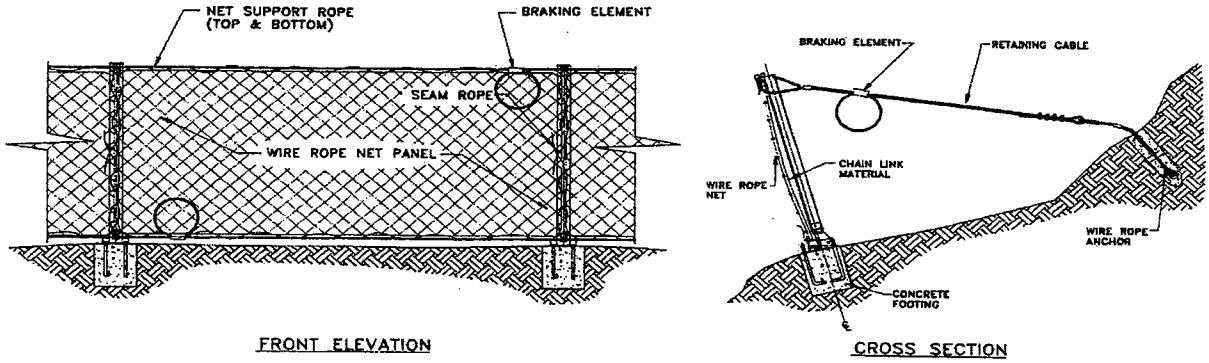


Figure 1. Typical Cable Net Barrier Design

rock impact, kinetic energy must be efficiently transferred from the nets to the support cables and finally to the anchors. In the case of energies approaching system design limits, the friction-braking elements will engage, further dissipating kinetic energy.

Because the netting is the primary point of impact, the nets must be both strong and flexible, and must efficiently transfer forces to the support infrastructure. It is through this net flexibility that much energy is dissipated through increasing the deceleration time of the falling rock. The netting must also have sufficient strength in order to minimize maintenance requirements after impacts.

TYPES OF NETS

Until recently, nets that are hand woven from galvanized aircraft cable have been the primary type of netting used. To date, these nets have performed well for the intended purpose.

For barriers with very high energy requirements, World War II surplus anti-torpedo nets have been used very

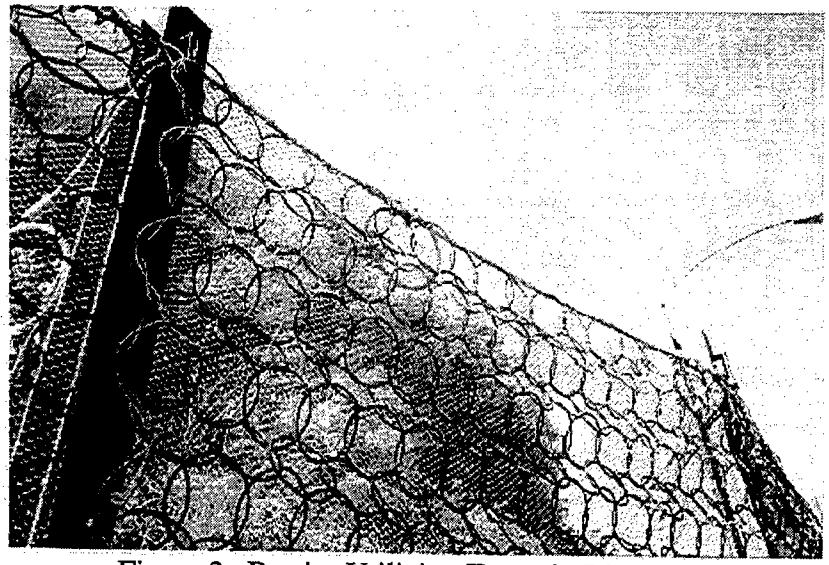


Figure 2. Barrier Utilizing Torpedo Ring Net

successfully (see Figure 2). These nets are ideal for flexible rockfall catch barriers because the interlocking rings are very flexible and are highly robust in strength. In addition to the superior flexibility and strength of these ring nets, there are other advantages for use of these nets over woven cable nets, as discussed later in this paper. However, because these nets are World War II surplus, the supply is finite, with the prospect of manufacturing identical nets being uneconomical in modern times.

In order to take full advantage of the superior performance characteristics of ring nets, in the mid 1990's, Brugg Cable Products began research and development of machinery that could economically produce an acceptable facsimile to the torpedo nets. Soon thereafter, the first ring nets produced since World War II were produced by Brugg Cable Products in Switzerland, under the trade name of "ROCCO" nets.

ROCCO NETS

ROCCO ring nets are manufactured on a fully automated machine in Brugg's Santa Fe, NM factory for the North American market, as well as in facilities in Switzerland and Japan. As this machine was independently developed by Brugg Cable Products, the machine and all ROCCO nets are patented in the U.S., Canada, and elsewhere.

ROCCO Net Characteristics

Nets are composed of interlocking wire rings, each ring having a diameter of 12

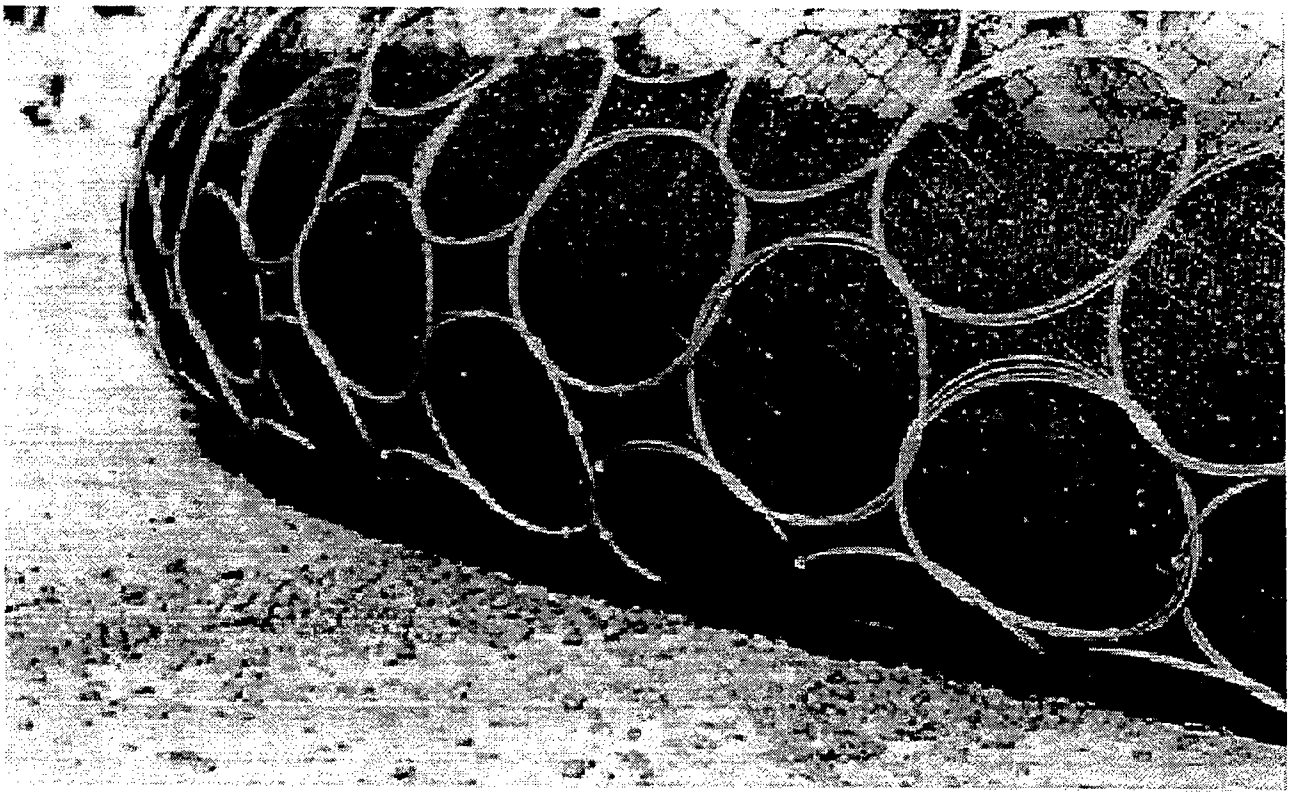


Figure 3. ROCCO Ring Net With Retained Debris

inches and with each ring interlocked with 4 adjacent rings. Rings are composed of several spirals or coils of 3.0 millimeter diameter wire, with the number of spirals depending on the kinetic energy rating of the barrier in which the net will be used. Typically 5, 7, 12, or 19 spirals of wire are used. A minimum of 3 steel clips are mechanically pressed around the spirals to securely hold the ring together. Figure 3 illustrates a good close-up view of the nets, full of retained debris during testing of the debris flow protection barrier system.

The special wire used for the rings is composed of high-tensile carbon steel, alloyed, with low contamination, and with a minimum mechanical strength of 1770 N/mm^2 (210,000 lbs/in^2). While the extremely high tensile strength of this wire is very hard and resistant to mechanical strain, it is also sufficiently tough and does not become brittle. Two types of coating are available for the wire: heavy zinc galvanization and zinc/aluminum supercoating.

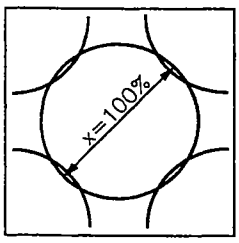
Testing

In the last decade Brugg Cable Products has conducted hundreds of tests of ring net rockfall barriers at various test sites in Switzerland as well as in Japan. Since 1991, the Group for Prevention Engineering at the WSL/FNP Swiss Federal Institute for Forest, Snow and Landscape Research, Department for Natural Hazards has been supporting and analyzing field tests of various Geobrugg rockfall protection barriers at various permanent test facilities in Switzerland.

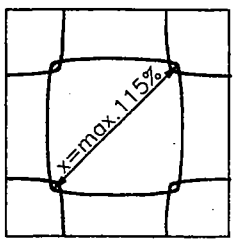
Through this testing, rockfall impact energies were analyzed and the response and behavior of the systems studied. This detailed analysis of rock deceleration in the barrier allows the calculation of the distribution of impact forces in the individual system components. The result has been an improved and advanced generation of ring net barriers capable of stopping rockfall energies ranging from 40 kJ to 2,000 kJ.

ADVANTAGES

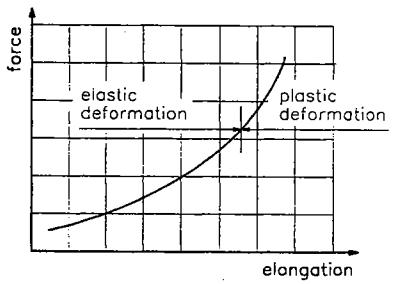
There are a number of clear advantages to using ring nets instead of woven nets, including superior performance during rock impacts, superior corrosion resistance, better adaptability and conformance to irregular terrain, ease of installation, reduced maintenance, and aesthetics. For these reasons, ring nets are quickly becoming the rockfall barrier netting standard in most recent installations worldwide.



shape of ring before impact



shape of ring during impact



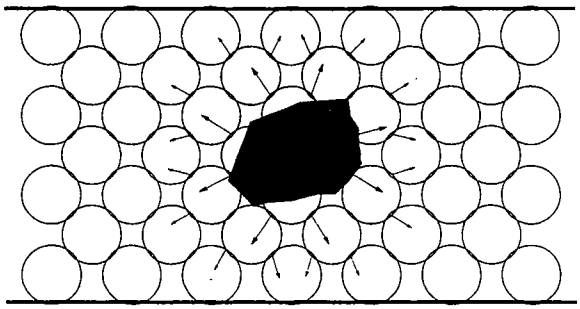
Deformation resistance of one single ring according to the bending resistance of the bundle of wires.

Figure 4. Deformation of Rings During Impact

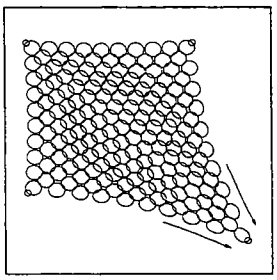
Performance

Unlike woven cable nets, which have limited ability to deform, ring nets behave in an elastic/plastic manner during significant rock impacts, thereby internally dissipating kinetic energy. Before rock impacts, rings are circular in shape, but deform temporarily to a square shape during rock impacts (see figure 4). In all but the most significant of impacts, the rings will regain most of the original circular shape after the rock impact due to the high tensile strength of the steel wire. The sum of the deformations in all the rings in the net equates to high total internal energy absorbing capacity of the net itself.

The configuration and interlocking nature of the rings also results in a very even distribution of forces throughout the net, resulting in efficient utilization of the entire net and smooth response of the net and support infrastructure (see figure 5).



Even distribution of the forces after an impact into the net.



The shape of the net adapts simultaneously -in 3 dimensions-to the dynamics of the impact force.

Figure 5. Distribution of Forces

Testing has also shown that ring net barriers are suitable against impacts from fallen trees sliding down slopes. In contract to woven nets, it is not possible for the opening of the net to be widened by the wedge effect of tree or rocks, resulting in gaps which may allow even big boulders to pass the barrier line.

Corrosion Resistance

Because the diameter of wire used for the rings is much larger than wire used for wire rope, a much thicker coating zinc or zinc/aluminum can be applied to the wire than is possible with wire rope. With heavy zinc galvanization, a minimum thickness of zinc coating of 100 g/m^2 can be applied to the ring net wire, which is up to 80% higher than the zinc coating of stranded wire ropes. This thicker application of coating provides much longer life when the wire is exposed to corrosive environments, than is observed with wire rope. At a coating thickness of 250 g/m^2, the zinc aluminum supercoating (95% Zn, 5% Al) provides far superior corrosion resistance at 2 to 3 times that of conventional zinc coating.

Conformance to Terrain

Because the rings are interlocked but can move freely within each other, the nets conform extremely well to irregular terrain where the netting meets the ground surface. This conformance allows gaps to be filled and rises to be topped that would otherwise present problems with woven cable nets. Additionally, in many cases, border rings can be cut and removed to allow a tight fit with the terrain if necessary, or can be laid upslope without compromising system performance.

Elevation changes along the line of the barrier can be accommodated more readily with ring nets. Because woven cable nets are securely clipped together, they tend to retain their shape and allow very little shape manipulation, such that elevation changes between any two posts of only 5-6 % can be tolerated without requiring specially shaped nets. On the other hand, the shape of ring nets can be physically manipulated to tolerate elevation changes between any two posts of 10-15% before special net shapes are required.

Ease of Installation

Installation of ring net barriers is simplified due to the reduced need for surveying of the barrier line, and other special layout requirements of other systems. Handling of the ring nets is relatively simple and problem-free during installation.

Reduced Maintenance

All other support infrastructure components remaining equal, rockfall barriers utilizing ring nets will have actual capacity much greater than rockfall barriers utilizing woven cable nets due to the internal energy dissipating characteristics of the netting. In other words, since the ring net absorbs a significant part of the impact energy, the braking elements respond later than with systems using woven cable nets, and thus need replacement only in the case of unusually large events. The net result is either a system with greatly reduced maintenance, or the allowance for a lighter support infrastructure and thus more economical system. In the U.S., Brugg Cable Products has elected to maintain the support infrastructure as has been used to date with woven cable nets, to realize the increased system capacity and reduced maintenance, until such time that lighter infrastructures are deemed acceptable by U.S. users of these systems.

In the unlikely event that a ring is damaged, it can easily be cut out and replaced with a new ring, without affecting the integrity of the net.

Aesthetics

The properly tensioned ring net leaves an attractive symmetric appearance. Ring nets made from the supercoated wire develop a dark gunbarrel gray color in a short period of time that is much less visually intrusive than zinc galvanization.

INSTALLATIONS

Hawaii D.O.T.; Kamehameha Highway

This 910 foot long, 12 foot high barrier was installed in May, 2000 along the Kamehameha Highway at Waimea Bay on the island of Oahu. With a design load for 74 ft-tons of kinetic energy, this system was installed near the edge of the roadway to protect motorists as an emergency project following a large rockfall that closed this main thoroughfare. A large catchment area between the slope and the road provides considerable capacity to dampen rockfall energies and provide collection of rocks. All elements of the barrier support infrastructure were per the standard design used by Brugg.

ROCCO ring nets were used instead of the old woven cable nets. Due to the immediate proximity to the ocean and consequent salt spray, supercoated Zn/Al wire was used for the rings, providing the extra corrosion protection needed. The supercoated rings also will darken over time, helping to

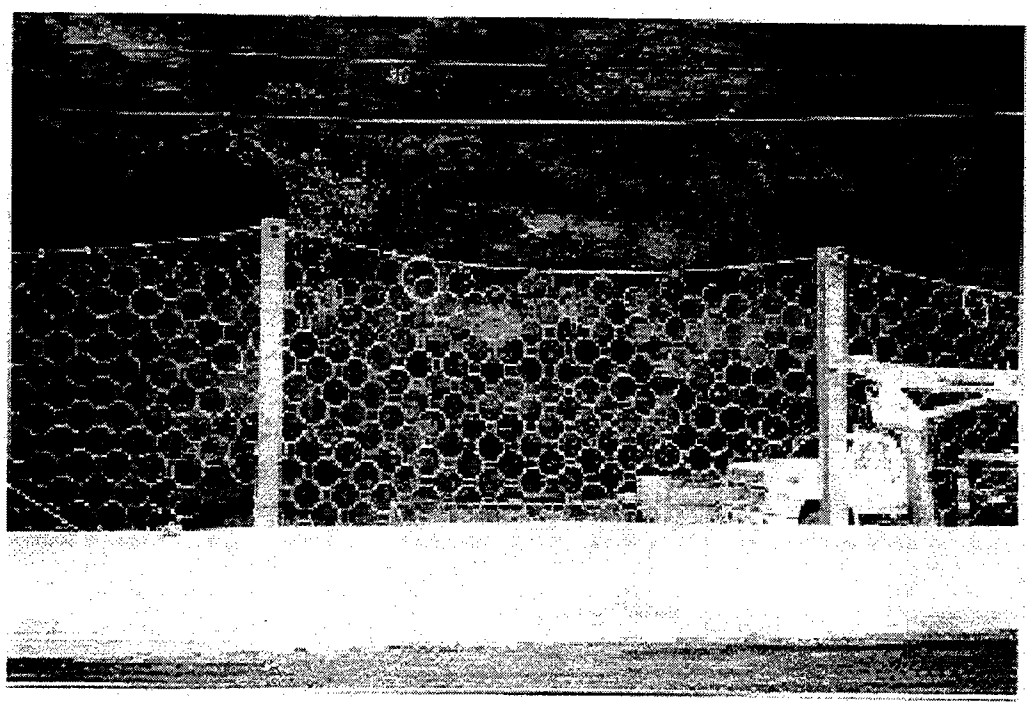


Figure 6. Kamehameha Highway ROCCO Net Barrier

minimize the visual impact in this aesthetically sensitive area. Figure 6 shows this system during installation, with the nets temporarily hung with shackles prior to lacing with the seam rope.

Caltrans; Highway 1 at Big Sur

This 22 foot long, 13 foot high barrier was installed in April, 2000 along the Pacific Coast Highway 1 along the Big Sur of California. With a design load for 129 ft-tons of kinetic energy, this system was installed below the roadway and loaded with rock, as an erosion prevention structure. Though this is a special application design, all elements of the infrastructure are more or less standard design elements.

ROCCO ring nets were used instead of the old woven cable nets. In this case, heavy galvanized wire was used for the rings, since supercoated wire was not quite ready for use at the time. However, the heavy galvanized wire still will provide significant extra corrosion protection needed in this coastal environment.

CONCLUSIONS

The last decade of rockfall barrier testing has produced a series of significant improvements to overall system design and individual components of rockfall protection barriers. One of the most significant advancements has been the development of the ROCCO ring net. Through the internal elastic/plastic response of these nets, overall system performance has been enhanced while reducing maintenance requirements for the systems. Other advantages include improved corrosion resistance, improved ability to conform to irregular terrain, ease of installation, and ease of repair. As a result of these advantages, ROCCO ring nets are quickly becoming the new standard for rockfall barrier netting worldwide.

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LANDSLIDE STABILIZATION
INTERSTATE 40, PIGEON RIVER GORGE
NORTH CAROLINA

by
Duncan Wyllie[1] and Russell Glass[2]

On July 1, 1997, a rock fall in the Pigeon River Gorge in North Carolina between Asheville and Knoxville blocked traffic on the Interstate I-40, and it was 71 days before the highway was open to single lane traffic. The landslide eventually extended to a height of 500 ft. above the highway. The rock at the site is a meta-sedimentary shale that contains bedding planes dipping at about 40 degrees towards the highway, and with continuous lengths of many of hundreds of feet. The cause of the slide was the development of three wedges formed by the intersection of the bedding and a conjugate joint set; the central wedge was formed by a single bedding plane extended over the full height of the slide.

Stabilization of the slope involved removal of about 200,000 cu. yd. of rock and soil, trim blasts, rock bolts, drain holes, rock catch fences and a rock fall barrier. A movement monitoring system was also set up so that the remedial work could continue while work was on progress on the slope. The work took about seven months to complete.

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Twenty-Seventh Annual Highway Geology Symposium - May 19-21, 1976 - Florida Department of Transportation and University of Florida - Orlando, Florida.

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"

* Paper Not Published

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 Economically 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."

Twenty-Eighth Annual Highway Geology Symposium - August 10-12, 1977 - South Dakota Department of Transportation, Geological Survey, and South Dakota School of Mines & Technology - Rapid City, South Dakota.

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."
Royster, David L., "Some Observations On The Use Of Horizontal Drains In The Correction And Prevention Of Landslides."

Twenty-Ninth Annual Highway Geology Symposium - May 3-5, 1978 - Maryland State Highway Administration and Geological Survey - Annapolis, Maryland.

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 "O" 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."

Thirtieth Annual Highway Geology Symposium - August 8-10, 1979 - Office of Federal Highway Projects, Federal Highway Administration - Portland, Oregon.

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, Reynold 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."

Thirty-First Annual Highway Geology Symposium - August 13-15, 1980 - Bureau of Economic Geology and The University of Texas at Austin in Cooperation with Texas Department of Highways and Public Transportation at Austin, Texas.

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: Highway 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, Tennessee.

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 Degrability For Compacted Shales."
Wilson, Charles; Pope, David; and Sherman, William, "A Review Of The Progress Of The Wyoming Heat Pipe Program."
Jones, Don H.; Bell, Bruce S.; 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 Of 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 Tests."
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, California.

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 O.; 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 at Purdue University - 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. O. 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 Does It Mean?"
Schulte, Michael P., "Dynamic Pile Monitoring And Pile Load Tests In Unconsolidated Sands And Gravels, Wyoming."
Olson, Larry D. and Church, Edward O., "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, Pennsylvania.

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 By 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 Geosynthetics 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 Tied 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."
Thronton, Sam I., and Elliott, Robert P., "Rapid Shear As An Evaluation For Base Course Material."

Fortieth Annual Highway Geology Symposium on "Symposium Venue" - May 17-19, 1989 - Alabama Highway Department - Birmingham, Alabama.

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, "Suitability 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 O.; Mutchler, Jack W.; Smith, Schaun M.; and Griffin, Paul M., Jr., "Slope Failures On Highway Embankments High In Shrink-Swill 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 At 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."

Forty-First Annual Highway Geology Symposium - August 15-17, 1990 - New Mexico State Highway and Transportation Department and New Mexico State University Department of Civil, Agricultural and Geological Engineering - Albuquerque, New Mexico.

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."

Forty-Second Annual Highway Geology Symposium on "Geologic Complexities in the Highway Environment" - May 28-31, 1991 - American Society of Civil Engineers, American Institute of Professional Geologists, New York State Department of Transportation, New York State Geological Survey - Albany, New York.

Bydlon, B., "Geologic Innovations on the Pennsylvania Turnpike."
Rudenko, D; Ackerman, H; and Lorence, W., "Seismic Refraction Technique Applied to Highway Design in a Strip Mined Area of Southwestern Pennsylvania."
Decker, M., and Jacobsen, G., "Evaluation of Acid Leachate Potential in Highway Construction."
Stokowski, S., "Quarry Layers - Stratigraphic Units that Serve the Public Interest."
Fischer, J., and Greene, R., "Roadways in Karst Terrane."
Mellett, J., and Maccarillo, B., "Highway Construction in Karst Terranes: Avoiding and Remediating Collapse Features."
Hardy, H., "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. L., III, and Gansfuss, J., "Rock Slope Investigations at Selected Hudson Valley Sites."
Burke, J., and LeFevre, S., "Rock Slope Inventory, Evaluation and Remediation for Sections Along the NYS Thruway."
Cross, R., "A Design for A Temporary Reusable Rock Catchment Barrier."
Bolton, C., "Evolution of Rock Excavation and Stabilization in New York State."
Abramson, L., "Complex Geology at Complex Sites."
Dunn, J; Banino, G; and LaGrand, D., "Treated Aggregate in an Asphaltic Concrete Road: An Apparent Success."
Hudec, P., and Achampong, F., "Improving Aggregate Quality by Chemical Treatment."
Parola, A., and Hagerty, D., "Highway Bridge Failure by Foundation Scour and Instability."
Butch, G., "Measurement of Scour at Selected Bridges in NY."
Horne, W.; Stevens, D.; and Batson, G., "Ground Penetration Radar Study of Riverbed Scour in NYS."
McGuffey, V., "Clues to Landslide Identification and Investigation."
Thornton, S., and Garrett, S., "Slope Failure Probability for Mixed Layer Soils."
Baskerville, C., "Northern New England Landslides."

Forty-Third Annual Highway Geology Symposium - August 19-21, 1992 - University of Arkansas Dept. of Civil Engineering, Arkansas State Highway and Transportation Department and Arkansas Geological Commission - Fayetteville, Arkansas.

Lin, P. S., and Lovell, C. W., "The Critical State of Debris Flow."
McManis, Kenneth , and Nataraj, Mysore, "The Influence of Post Depositional Effects on the Engineering Properties of a Marine Clay."
*Stone, Charles G., "Overview of the Complex Highway Geology in West-Central Arkansas."
O'Hara, Kevin C., and West, T. R., "Evaluation of Coal Refuse For Access Road Construction at an Abandoned Mine Lands Site, Southwest Indiana."
McFarland, John D., "Landslides on Crowley's Ridge."
Annable, Jonathan and Sharum, John, "Slope Failures on Highway 71 Relocation Projects."
*Munoz, Andy, Jr., "Slope Maintenance and Slide Restoration."
Graham, J. R.; Ingraham, P. C.; and Humphries, R. W., "Repairs to Rock Slopes at the US-22/SR-7 Interchange in Steubenville, Ohio."
Thornton, Sam I., and McGuire, Michael S., "Cannon Creek Embankment Instrumentation."
Lumbert, David W., and Thian, Boon K., "Design, Construction and Monitoring of a 76 Foot High Geogrid Reinforced Earth Embankment."
Moore, Harry, "The Use of Geomembranes for Mitigation of Pyritic Rock."
Lumbert, David W., and Stone, Charles G., "Highway Geology at Selected Sites in the Boston Mountains and Arkansas Valley, Northwest Arkansas."
Yarnell, Charles N., "1992 Update: Steel Wire Rope Safety Net Systems Application to Rockfall Mitigation in the U. S."
Macintosh, O. R., "Value Engineering Proposal Using Hilfiker Welded Wirewall for Soda Springs Highway Near Auburn, California."
Austin, Alvin L., and Annable, Jonathan A., "In-situ Moisture Content of Arkansas Subgrades."
Henthorne, Robert; Jones, William; and Rockers, Larry, "Investigation and Remediation of Undermined Highway."
Darrag, A. Amr; Lovell, C. W.; and Karim, A. M. K., "New Correlations for Piles Driven Into Cohesionless Soils."
Selvam, R. Panneer, and Elliott, Robert P., "Nonlinear Finite Element Analysis of Pavements Using Microcomputer."
Thornton, Sam I., and Ford, Miller C., Jr., "Aggregate Suitability (Stripping) for Asphalt Pavements."
Ruppen, Christopher A., and Baker, Michael, Jr., "When Does the Work End?"

* Abstract Only

Forty-Fourth Annual Highway Geology Symposium - May 19-21, 1993 - University of South Florida Dept. of Civil Engineering and Mechanics and Florida Department of Transportation - Tampa, Florida.

Goddard, Robert E., P.G. and Wisner, William A., Jr., P.G., "Use of Aggregate in the Sunshine Skyway Bridge Project."

Khalid, Jamil, and Thornton, Sam I., "Resilient Modulus vs Strength in Cement Stabilized Base Courses."

Javed, S., and Lovell, C. W., "Use of Waste Foundry Sand in Highway Construction."

Zayed, Abla M., "Effect of Fly Ash Quality on Concrete Durability."

Spencer, Steven M., P.G., "A Roadway Problem in a Cavernous Karst Environment at the Florida Caverns State Park."

Foshee, Jon, and Bixler, Brian, "Cover-Subsidence Sinkhole Evaluation State Road 434 - Longwood, Florida."

Yovaish, Douglas J., P.E., and Law, Donald S., P.E., "Technical Related Analysis, Design, and Construction 4 Lane Highway Over 8 to 20 Feet of Peat."

Marienfeld, Mark, P.E., "Highway Reconstruction Over an Expansive Subgrade Incorportating a High Strength Geosynthetic Moisture Barrier."

Pittenger, Robert A., and West, Terry R., "Investigation for Landfill Expansion in a Bedrock Area, Southcentral Indiana."

Behringer, David W., and Shakoor, Abdul, "A Study of Selected Landslides Along Cincinnati Roadways."

Ross, Mark A., and Vincent, Mark S., "Numerical Modeling of Sediment Erosion at Tidal Inlet Bridges."

Goddard, R. E., "44th Annual Field Trip Guide Unique Construction Projects and Problems in the Tampa Bay Area of Florida."

Carrier, III, W. David, "Mechanical Properties of Lunar Soils and Implications For a Lunar Base."

Sokol, Thomas P., P.E., and McCahan, Matthew L., "Pennsylvania Turnpike Expansion: A Reprospect."

*Ahmed, Imtiaz, and Lovell, C. W., "Laboratory Study on Properties of Rubber-Soils."

Schneider, Nicholas P., and Bauer, Robert A., "Environmental Property Assessments for Highway Projects: Key Elements for Successful Program Implementation."

Madrid, Larry D., P.E., and Smith, Ted J., P.G., "Construction of a Four-Lane Highway Embankment Over a Contaminated Landfill." (Not Published)

Kasim, Margaret F., and Shakoor, Abdul, "Predicting the Compressive Strength of Rocks From Aggregate Degradation."

Chen, W. Y.; Lovell, C. W.; Pyrak-Nolte, L. J; and Haley, G. M., "New Precursor of Stick-Slip Movement of Rock Block."

Marcozzi, Guy F., P.E., "Cement Amended Fly Ash as a Structural Fill."

* Abstract Only

Forty-Fifth Annual Highway Geology Symposium - August 17-19, 1994 - Portland State University, Oregon Department of Transportation, Washington Department of Transportation, Transportation Research Board - Portland, Oregon.

Koelling, Mark, "Ground Improvement Case Histories for Highway Construction."

Johnston, Robert E., and Abramson, Lee W., "Geosynthetic Reinforcing of Highway Embankments."

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

Kuhne, J. C., and Glass, F. R., "Probabilistic Analysis and Design of Slopes on Interstate 26 From Asheville, NC to Tennessee."

Shaw, Lee R.; Toh, Chin Leong; and Thornton, Sam I., "Effects of Lime on Cannon Creek Embankment Soil."

Armour, Tom A.; Johnston, Mark S; and Groneck, Paul B., "Innovative Rock Anchoring at Boundry Dam, Washington."

Badger, Tom, "Rock Excavations in Steep Ground."

Flatland, Robert; Watters, Robert J.; and Cochrane, David, "Application of the Rockfall Hazard Rating System to Rock Cuts in Mountainous Terrain."

Thommen, Robert A., and Montejo, Manuel A., "Rock Retaining Wire Rope Net System Installation at Orizaba, Mexico."

Hannan, Richard; Moran, Michael; and Scofield, David, "Design, Construction, and Performance of Horizontal Drains at Bonneville Navigation Lock, Oregon."

Humphries, Rich, "Recent Geotechnical Advances in the Design and Construction of Highway Tunnels."

Huddleson, Steven M., and Lovely, Anne, Esq., "Hazardous Materials in the Roadway."

Javed, S.; Lovell, C. W.; and Eastwood, D. A.; "Waste Foundry Sand in Subgrade and Controlled Low Strength Material."

Burk, Robert L.; Norrish, Norm; and Lowell, S. M., "Investigation, Design, and Construction of the Spirit Lake Memorial Highway, Washington."

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

Hager, G. Michael, and Falk, Mark, "Catastrophic Embankment Failure South of Sheridan, Wyoming, Interstate 90 - M. P. 41.1."

Kane, William F., and Beck, Timothy J., "Development of a Time Domain Reflectometry System to Monitor Landslide Activity."

Kobernik, Ricki M.; Toor, Frank N.; and Watanabe, Richard, "The Arizona Inn Landslide, Curry County, Oregon."

Peterson, Gary L; Squier, L. Radley; and Scofield, David H., "Engineering Geology and Hydrology Update, Arizona Inn Landslide, Curry County, Oregon."

Bruner, D. W.; Choi, J. C.; and West, T. R., "Evaluation of Indiana Aggregates for Use in Bituminous Highway Overlays."

Fisk, Lanny H., and Spencer, Lee A., "Highway Construction Projects Have Legal Mandates Requiring Protection of Paleontologic Resources (Fossils)."

Gaffney, Donald V., "BV-116: The Bridge by Addendum: Wetlands Dictate a Change in Design."

Selvam, R. Panneer; Elliott, Robert P.; and Arounpradith, A., "Pavement Analysis Using ARKPAV."

Sherman, William F., "The Impact of Registration on Geologists on the Professional Involved in Highway Geology."

Forty-Sixth Annual Highway Geology Symposium - May 14-18, 1995 - West Virginia Geological and Economic Survey, West Virginia Division of Highways, West Virginia University, Transportation Research Board - Charleston, West Virginia.

Kroenke, Mark A., and Shakoor, Abdul, "A Geotechnical Investigation of the Chagrin River Road Landslide Complex in the Moreland Hills Area, Cuyahoga County, Ohio."

Reed, Benjamin C; McConnell, W. T.; and Mullen, D. M., "Potential for Slope Instability and Acidic Runoff Along a Section of the U. S. 19 Corridor; Cherokee, Graham, and Swain Counties, North Carolina."

Hamel, James V., and Hamel, Elizabeth A., "Landsliding in Pennsylvania."

Martin, A. David, "Maryland Route 31 Sinkhole."

Fischer, Joseph A., and Fischer, Joseph J., "Remediation for Highways in Karst."

Stephenson, J. Brad; Beck, Dr. Barry F., P. E.; Smoot, Dr. James L. and Turpin, Anne, "Management of the Discharge and Quality of Highway Stormwater Runoff in Karst Areas to Control Impacts to Ground Water - A Progress Report."

Hager, G. Michael, and Falk, Mark, "Landslide Repair Utilizing A Steepened Geogrid Reinforced Slope, Interstate 90, M. P. 28.9, Sheridan County, Wyoming."

Singh, Yash P.; Cavan, Bruce P.; Rhodes, Gary W., "Design of Rock Reinforcement for Tallulah Gorge Bridge Foundations."

Bigger, Joseph C., "Rock Fall Mitigation Program at Nantahala Dam Using Wire Rope Nets in Conjunction with Other Techniques."

Seider, Gary L., and Smith, Walter, "Helical Tieback Anchors Help Reconstruct Failed Sheet Pile Wall."

Ludwig, Harald P., "Permanent Highway and Landslide Walls."

Hoffman, A. G.; Melick, D. F.; Clark, D. M.; Bechtel, T. D.; "Abandoned Deep Mine Subsidence Investigation, and Remedial Design, I-70, Guernsey County, Ohio."

Toh, Chin Leong, and Thornton, Sam, "Limestone Base Course Permiability."

Lovell, C. W.; Bernal, Andres; and Salgado, Rodrigo, "Uses of Scrap Rubber Tires in Civil Engineering."

West, Terry R., "Evolution of A Technique: Petrography of Aggregates for Concrete and Bituminous Highway Pavements."
Johnston, R. Michael, and Vierling, Michael P., "A Modified Presplit Blasting Method for Use in Environmentally Sensitive Areas."
Hall, Kevin D., and Watkins, Quintin B., "Effect of Soil Horizonation on Flexible Pavement Responses."
Brown, Christopher L., and Shakoor, Abdul, "Predicting the Unconfined Compressive Strength of Carbonate Rocks from Los Angeles Abrasion Test Data."
Ruppen, Christopher A., and Eliot, Gordon M., "Mitigating Landslide/Rockfall Hazards - A Quantitative and Qualitative Evaluation of Risk Reduction."

Forty-Seventh Annual Highway Geology Symposium - September 6-9, 1996 - Wyoming Department of Transportation, Wyoming Geological Survey, Cody, Wyoming.

Burns, Scott, "Earthflows, Debris Flows and Roads: The Aftermath of the Winter Storm of 1996, Portland, Oregon."
Fickies, Robert, "Landslide Hazards in New York State: A Geological Overview."
Adams, Wayne, and Findley, Dave, "Peters Road Landslide."
Willems, Henry and Podniesinski, Matthew, "Rock Slope Stabilization at Dunderberg Mt., Rockland County, New York."
Priznar, Nick, and Euge, Kenneth, "Geologic Investigation of the Gordon Canyon Slide."
Turner, J. P.; Haskenkamp, R. N.; and Edgar, T. V., "Reinforced Earth Retaining Wall for Landslide Control, Snake River Canyon, Wyoming."
Wyllie, Duncan, and Kielhorn, W., "Wedge Stability Analysis - Geometric and Groundwater Enhancements."
Duskin, Priscilla, "Seismic Refraction as a Method for Determining Thickness of Organic Sediments."
Beck, Timothy J.; Kane, William F., "Current and Potential Uses of Time Domain Reflectometry for Geotechnical Monitoring."
Boundy, Bret, and Dahill, James, "Instrumentation of a Shredded Tire Fill Used for a Landslide Repair, The Burning Issue."
Gaffney, Donald V.; McCahan, Matthew L., "Integrated Geohazard Management - A Systemwide Approach."
Adams, Jr., William R., and Ruppen, Christopher, "SGH Form: Format for Early Collection of Essential Geotechnical Data."
Heinert, Kevin, "Permanent and Temporary Earth Anchoring for Highway Applications."
Ludwig, Claus, "Making a Soil Nail Wall Look Like Rock."
Moore, Harry, "The Use of Underbenching in Embankment Construction Through Mountainous Terrain, I-26, Unicoi County, Tennessee."
West, Terry, and Park, Hyuck Jin, "Rock Durability of Argillaceous Carbonate Rocks in Cut Slopes for Indiana Highways."
Lovell, C. W., "Flowable Fill Using Waste Products."

Studebaker, Irving; Patton, Susan; Studebaker, Raymond, "Subsidence Definition and Effects on Surface Construction."
Lundvall, J. F.; Turner, J. P.; Stewart, J. M.; Edgar, T. V., "Roadway Settlement Over Culverts: Causes and Cures."
Duffy, John, and DeNatale, Jay, "Debris Flow Mitigation Using Flexible Barriers."
Hall, Kevin, "Preliminary Investigations Using ALROC Potliner Sand in Asphalt Concrete Mixtures."

Forty-Eighth Annual Highway Geology Symposium - May 8-10, 1997 - Tennessee Department of Transportation, University of Tennessee: Department of Geological Sciences Institute for Geotechnology, Department of Civil Engineering, Tennessee Division of Geology Transportation Research Board, Knoxville, Tennessee.

Hager, G. Michael, and Huntington, George S., "Internal Sulfate Attack: Wyoming's Expansive Experience."
Fischer, Joseph A., and Fischer, Joseph J., "Investigation and Remediation of Service Plaza Fuel Storage Tanks in Karst."
Stephenson, J. Brad; Zhou, W. F.; Beck, Barry F.; Green, Tom S.; Smoot, James L.; and Turpin, Anne M., "Management of Highway Stormwater Runoff Karst Areas-Baseline Monitoring and Design of a Treatment System for a Sinkhole at the I-40/I-640 Interchange in Eastern Knoxville, Tennessee."
Marks, B. Dan, "Stabilization of Karst Features on Major Engineering Projects."
Sebela, Dr. Stanka; Slabe, Dr. Tadej; Kogovsek, mag. Janja, "Motorways in Karst of Slovenia."
West, Terry R., and Pittenger, Robert, "Computer Analysis of Required Statistical Parameters for Sites with Multiple Monitoring Wells Measured for Extended Periods of Time."
Karim, A., Salgado, Rodrigo and Lovell, C. W. "Bill", "Building Embankments of Coal Combustion Fly Ash-Bottom Ash Mixtures."
Ruegsegger, L. Rick, and Lefchik, Thomas E., "Abandoned Underground Mine Inventory and Risk Assessment."
Bruhn, Robert W., Roth, Bruce L., and Chelednik, Craig, "Physical Distress Evaluation of the West Abutment of the SR-22 Bridge over the Conemaugh River."
Shakoor, Abdul, and Chu, Chein-Jen, "Physical and Chemical Evaluation of Soil-Tire Mixtures for Highway Applications."
Szturo, John F., and Duryee, Wayne, "Case Study-Drilled Shaft Foundations-Lake of the Ozarks Community Bridge."
Adkins, Grant, and Rutkowski, Nanette, "Resistivity Field Testing of Reinforced Earth Structures."
Hall, Kevin D., "Potential Effects of Superpave Implementation on the Arkansas Aggregate Industry."
Martin, A. David, "The Landslide at Waverly Street and Maryland Route 36: A Case History."

Shakoor, Abdul, and Kroenke, Mark A., "Analysis and Remediation of the Miles Road Landslide Complex, Cuyahoga County, Ohio."
Wayne, Mark H., and Wokasien, Sean, "A Case History on the Use of Bioengineering in Mechanically Stabilized Earth Slope Design."
Wright, E. M., "State of the Art for Rock Cut Slopes in Eastern Kentucky."
Arwood, Scott; Mauldon, Matthew; and Moore, Harry, "Rock Slope Stability Analysis Based on Potential Energy."
Siebert, Harry L., "The Role of a Contractor's Blasting Consultant-A Case Study."
Cross, Richard H., "Elevated Catchment Areas: A Performance Report."
Lambe, Philip C., and Sweitzer, Margaret, "Selecting Shear Strength Parameters for Weathered Metamorphic Rock."
Beard, Jerome, "Parameters Used for Design of I-26 in North Carolina."
Haston, Matt; Mauldon, Matthew; and Byerly, Don W., "Fractal Geometry for the Quantification of Rock Joint Roughness."

Forty-Ninth Annual Highway Geology Symposium - September 10-14, 1998 - Arizona Department of Transportation, Materials Group; Arizona Geological Survey, Northern Arizona University, Department of Geology Prescott, Arizona.

Bieniawski, Richard Z. T., "Design Principles and Methodology for Excavations in Geologic Media."
Scovira, D. Scott, Barber, Michael F., "Final Rock Slope Blasting Techniques: Consideration in Design and Execution."
Brawner, C. O., "Case Examples for Rock Stability on Transportation Projects."
Peterson, David E., and Dahlen, Keith H., "Rock Slopes and Bridge Foundations in a Foliated Rock Mass along portions of State Route 69 Near Mayer, Arizona."
Beck, Timothy J., and Kane, William F., "Site Characterization and Slope Stability Evaluation for the San Francisco-Oakland Bay Bridge."
Glass, F. R., "A Large Wedge Failure Along Interstate 40 at the North Carolina-Tennessee State Line."
Duffy, John D., "Performance Review of Rockfall Mitigation Measures."
Moore, Harry, "Rockfall Control Fence, Unaka Springs Road, Unicoi County, TN."
Vierling, Michael P., "The Performance of Rockfall Mitigation Measures in New York State."
Gattis, J. L., and Thornton, Sam I., "Tunnel to the 21st Century".
Humphries, Richard, "Tunnel Enlargement and Rehabilitation".
White, Jonathan L., "High Altitude Slope Reconstruction and Restoration on Colorado State Highway 82, at Independence Pass, A Continuing Case Study."
Beard, Jerome, "400,000 Cubic Yard Slope Failure on I-26."
Falk, Mark, and Hager, G. Michael, "Wolf Mountain Landslide Milepost 136.57 Snake River Canyon, Teton County, Wyoming."

Turner, John; Hasenkamp, Robert; and Wolosick, John, "Micropile and Tieback Reinforced Walls for Landslide Stabilization, Snake River Canyon, Wyoming."

Schock, Robert E., "Three Interesting Drill Shaft Foundations Completed in Arizona."

Wright, Michael J., "Statnamic and Static Lateral Load Analysis of a Battered Pile Group vs. A Drilled Shaft Group."

Thommen, Robert, and Duffy, John D., "Testing and Application of Flexible Wire Rope Netting Barriers to Retain Mud & Debris Flow."

Siel, Barry D., and Siebert, Harry L., "Soil Nailing: Geology and Construction Considerations as They Relate to Field Inspection."

MacIntosh, O. R., and Alexander, Douglas E., "Call of the Canyon Mechanically Stabilized Earth Structure with Simulated Rock Facing."

Taylor, Gary E.; Gorski, Glen A.; and Alexander, Douglas E., "Compaction Grouting of a Deep Embankment Fill Along Interstate 15, Virgin River Gorge, Arizona."

Pearthree, Philip A., "Paleoseismology and Seismic Hazard Assessment in Arizona."

Hatheway, Allen W., "Former Manufactured Gas Plants (FMGP); Nemesis of Transportation Planning."

Lindberg, Paul A., "Structural Controls of Devils Kitchen Sinkhole; Sedona, Arizona."

Henthorn, Robert W., "Geologic Investigation and Construction of the Spring River Bridge."

Peterson, David E.; Rosberg, Lorena M., and Price, Ronald E., "Roadway Construction Considerations through a Young Volcanic Center, at the south end of Big Sandy Valley, near Wikieup, Arizona."

Slabe, Tadej, Dr., "Caves Discovered During The Construction of a Motorway Over Slovene Karst."

Fischer, Joseph A.; Fischer, Joseph J.; and Ottoson, Richard S., "Geotechnical Investigation in Karst: Bent Rock Versus Flat Rock."

Benson, Richard C.; Kaufmann, Ronald D.; Yuhr, Lynn B.; and Martin, David, "Assessment, Prediction and Remediation of Karst Conditions on I-70, Frederick, Maryland."

Euge, Kenneth M.; Hennon, Kerry; Williams, Jason C.; and Priznar, Nick, "Ground Penetrating Radar: An Effective Tool for Locating Dry Caves Along a Portion of State Route 69 Near Mayer, Yavapai County, Arizona."

Qubain, Bashar S.; Seksinsky, Eric J.; and Li, Jianchao, "Design Considerations for Bridge Foundations in Karst Terrain."

Meier, W. R., Jr., and Rosner, John C., "Prospecting for Local Material Sources."

Stokowski, Steve, "Sequential Deterioration of Concrete, Congress Street Bridge, Boston, MA."

Mather, Bryant, "How to Avoid Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction."

Duffy, Dennis, and Zaniewski, J. P., "Runaway Truck Ramp Testing Program."

West, T. R., Dr., "Evaluation of Crushed Stone Used for High Traffic, Bituminous Pavement Wearing-Courses in Indiana."
Chen, Dar-Hao; Bilyeu, John; and Murphy, Mike, "The Effect of Bedrock Depth On Pavement Response."
Paine, Jeffrey G., and Murphy, Michael R., "Influence of Bedrock Type and Depth on Highway Deflections."
Rucker, Michael L., "Percolation Theory Approach to Estimating Earthwork Factors in Weathered Granites."
Wilburn, David, and Goonan, Thomas, "Recycled Aggregate in Road Construction - An Economic Assessment."
Kelly, Thomas, "Crushed Concrete Substitution for Construction Aggregates - A Materials flow Analysis."
Maley, Terry S., "The Ultimate Roadside Geology: Geological Heritage Sites."

Fiftieth Annual Highway Geology Symposium - May 20-23, 1999 - Radford University, Department of Geology/Institute for Engineering Geosciences, Virginia Department of Mines, Minerals and Energy/Division of Mineral Resources, Virginia Department of Transportation - Roanoke, Virginia.

Moore, Harry, "A Brief History of the Highway Geology Symposium."
Hubbard, David A., Jr., "Highways in Virginia Karst: Resource and Hazard Considerations."
Gaffney, Donald V., and Dippel, Jeffrey G., "Karst Considerations in Final Design of Interstate 99 from State College, Pennsylvania, to Interstate 80."
Daniel, Joel C.; Benson, Richard C.; and Samford, Ann M., "Use of Geophysical Techniques to Evaluate Karst Features for Roadway Design."
Prowell, Marcia V.; Garst, Hugh C.; and Samford, Ann M., "A Case History on the Use of Compaction Grouting for Roadway Improvements in Mature Karst Topography."
Reccelli-Snyder, Heather L; Stahl, Beth A.; and Leberfinger, Jeffrey L., "Electrical Imaging: A Method for Identifying Potential Collapse and Other Karst Features Near Roadways."
Rehwoldt, Eric; Matheson, Gordon M.; and Dunscomb, Mark H., "Resistivity Imaging: Getting the Big Picture in Karst."
Nowak, Ethan, "Near Surface Cavity Detection Logon Co., Ohio."
Richard, B. H.; Wolfe, P. J.; Hauser, E. C.; and Hicks, J. D., "Geophysical Techniques to Locate Old Coal Mines Beneath Highways."
Zdinak, Aaron L.; Kaulfers, David A.; and Pelnik, Thomas W., III, "Detailed Subsurface Explorations Facilitate Cost Saving Designs for Transportation Projects."
Sayre, David M., and Nash, Bill, "Use of Environmental Site Assessments to Facilitate Roadway Design and Construction in the Church Street Urban Corridor, Norfolk, Virginia."

Mathewson, Christopher C., and Morris, Lloyd E., "Protection and Preservation of an Archaeological Site below a Constructed Highway Fill."

Benson, Richard C., "An Overview of NDT Methods for Characterization of Roads and Bridges."

Artman, Lawrence J., II, and McCahan, Matthew L., "Replacement of Bridge B-451, Milepost 78.98, Carrying S. R. 0819 Over the Pennsylvania Turnpike."

Wargo, Keith, and Zang, Scott D., "Pile Relaxation in Soft Shales - A Critical Evaluation of the Effectiveness of Various Remedial Strategies based on Experience on the Airport Busway Project in Pittsburgh, Pennsylvania."

Mathis, Henry, "Soil Modification and Stabilization Utilizing Lime."

Hudec, Peter P., and Mitchell, Andrew, "Multivariable Statistical Techniques Applied to Aggregate Inventory Testing Program in Northwestern Ontario."

Sherwood, W. Cullen; Mahone, David C.; and Eaton, L. Scott, "The Effect of Aggregate Type and Mix Design on Wet Pavement Skid Resistance."

Rupe, C., and Knight, K. L., "Grid-based model for prediction of debris flow susceptibility applied to the Appalachian Valley and Ridge, Spruce Run Mountain, Giles County, Virginia."

Dean, W. T.; Watts, C. F.; and Seaton, W. J., "Evaluation of a Suspected Ancient Mass Movement Using Electrical Resistivity."

Shakoor, Abdul, "A Reconnaissance Survey of Hazard Potential of Slope Movements Along the Karakoram Highway, Northeast Pakistan."

Duffy, John D., and Finegan, Mike, "1997 California Storm Damage: The Gorda Landslide."

Beck, Timothy, "Last Chance Grade and Wilson Creek Wall Landslides, Del Norte County, California."

Jensen, W. G.; Turner, J. P.; and Wolosick, J. R., "Soil Nail Wall for Stabilization of the Elbow Fill Slide, Snake River Canyon, Wyoming."

Marks, B. Dan, "Geogrid Reinforced Slope Repairs Airport Service Road Tri-State Airport Huntington, West Virginia."

Newman, F. Barry, and Adams, William R., Jr., "I-279 Landslide Repair."

Morris, A. J., and Wood, D. F., "Rock Slope Engineering and Management Process on the Canadian Pacific Railway."

Senior, S. A., "Rockfall Hazard Remediation Along Ontario Highways."

Sheahan, James M.; Hite, Stan L.; and Graybeal, Sam, "Design and Construction of Major Cuts on US 460 in Virginia."

Wright, Earl, "Rock Slope Engineering in Complex Geology: Jenkins By Pass."

Euge, Kenneth M., and Kandaris, Peter M., "Controlled Removal of Unstable Overhanging Rock above Roadway and Hydro Power Facility, Horse Mesa Dam, Salt River Canyon, West of Phoenix, Arizona."

Ruppen, Christopher A., "The MP 5.9 Rockslide Emergency Clean-up, Design and Construction."

Rorem, Erik J., "Flexible Wire Rope Net Barriers for Debris Flow Protection: A Review of Installations to Date."
Andrew, Rick; Fry, David; and Bookwalter, Robert E., "Rock Fall Control Test Report #1 Rifle, Colorado."
Kane, William F., and Beck, Timothy J., "Advances in Highway Slope Stability Instrumentation."
Cole, Joshua H., and Mauldon, Matthew, "Rock Slope Analysis Using Interactive Visual Software."
Park, Hyuckjin, and West, Terry R., "Discontinuity Orientation Measurements for Rock Slope Design in Western North Carolina."

Fiftieth - Anniversary Special Proceedings Historical Volume - Highway Geology Symposium National Steering Committee - 1999.

2nd Annual Symposium, February 16, 1951, Richmond, Virginia.
Granger, A. T., "What Does The Engineer Expect Of The Geologist?"
3rd Annual Symposium, February 29, 1952, Lexington, Virginia.
Baker, R. F., "The Design of the Slopes of Highway Rock Excavations in West Virginia."
4th Annual Symposium, February 20, 1953, Charleston, West Virginia.
Marshall, Harry E., "Some Experiences of the Department of Highways with Landslides in Ohio."
Price, Paul H., "The Landslide Problem."
5th Annual Symposium, March 16, 1954, Ohio State University.
Warrick, W. A., "Application of Geology To Highway Engineering as seen by a Highway Engineer."
6th Annual Symposium, February 18, 1955, John Hopkins University.
Singlewald, Joseph T., "The Role of the Geological Surveys in Geology As Applied to Highway Engineering."
9th Annual Symposium, February 21, 1958, University of Virginia.
Legget, Robert F., "Geology and Transportation Routes."
24th Annual Symposium, Sheridan Center, Sheridan, WY, August 9-10, 1973.
Bukovansky, Dr. Michal, "Engineering Geology & Rock Mechanics Help to Design a Highway in Northern Spain."
25th Annual Symposium, May 23-24, 1974, Raleigh, North Carolina.
Royster, David L., "Construction of a Reinforced Earth Fill Along I-40 in Tennessee."
28th Annual Symposium, August 10-12, 1977, Rapid City, South Dakota.
Burgat, Virgil A., "Cut Slope Design Based on Stability Characteristics."
Royster, David L., "Some Observations on the Use of Horizontal Drains in the Correction and Prevention of Landslides."
30th Annual Symposium, August 8-10, 1979, Portland, Oregon.
Newton, Jackson, "Summary of Use of Sawdust for Highway Fills."
Royster, David L., "Landslide Remedial Measures."
41st Annual Symposium, August 15-17, 1990, Albuquerque, New Mexico.

Moore, Harry L., "Rockfall Mitigation along I-40, Cocke and Cumberland Counties, Tennessee."

(41st Annual Symposium, Cont.)

Findley, David P., and Deardoff, George, "17 Miles to Mount St. Helens: Operational Aspects of the Geotechnical Investigation."

42nd Annual Symposium, May 28-31, 1991, Albany, New York.

Burke, J., and LeFevre, S., "Rock Slope Inventory, Evaluation and Remediation for Sections Along the New York Thruway."

McGuffey, Verne, "Clues to Landslide Identification and Investigation."

43rd Annual Symposium, August 19-21, 1992, Fayetteville, Arkansas.

Henthorne, Robert; Jones, William; and Rockers, Larry, "Investigation and Remediation of Undermined Highway."

Moore, Harry L., "The Use of Geomembranes for Mitigation of Pyritic Rock."

45th Annual Symposium, August 17-19, 1994, Portland, Oregon.

Thommen, Robert A., and Montejo, Manuel A., "Rock Retaining Wire Rope Net System Installation."

Sherman, William F., "The Impact of Registration of geologists on the Professional Involved in Highway Geology."

46th Annual Symposium, May 14-18, 1995, Charleston, West Virginia.

Johnson, Michael R., and Verling, Michael, "A Modified Presplit Blasting Method for Use in Environmentally Sensitive Areas."

West, Terry R., "Evolution of a Technique: Petrography of Aggregates for Concrete and Bituminous Highway Pavements."

49th Annual Symposium, September 10-14, 1998, Prescott, Arizona.

Glass, F. R., "A Large Wedge Failure Along Interstate 40 at the North Carolina-Tennessee State Line."

51ST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

FIELD TRIP

Seattle, Washington
August 29th -September 1st 2000

Sponsored By
Washington State Department of Transportation

51st Annual Highway Geology Symposium

Mount Rainier Field Trip

Field Trip Organizers

Dave Jenkins - WSDOT
Pat Pringle - WADNR

Tentative Schedule

Leave Double Tree Guest Suites: 7:30AM

Stop No. 1 - Orting Buried Forest: 8:15AM - 9:00AM
Steve Bailey
Pat Pringle

Stop No. 2 - Mud Mountain Dam: 10:15AM - 11:00AM
Pat Pringle
Bernie Hargrave
Larry Mann

Stop No. 3 - Sunrise Visitor Center: 1:00PM - Lunch
Pat Pringle - trail guide (optional)

Leave Sunrise Visitor Center: 3:00PM

Arrive Double Tree Guest Suites: 5:30PM

Special thanks to the following individuals:
Steve Palmer - WADNR
Matt Brunengo - WADNR
Steve Bailey - Pierce County Emergency Management Richard Schrodell - Pierce County Emergency Management
Bernie Hargrave - Corps of Engineers
Larry Mann - Corps of Engineers

HGS/NWGW FIELD TRIP

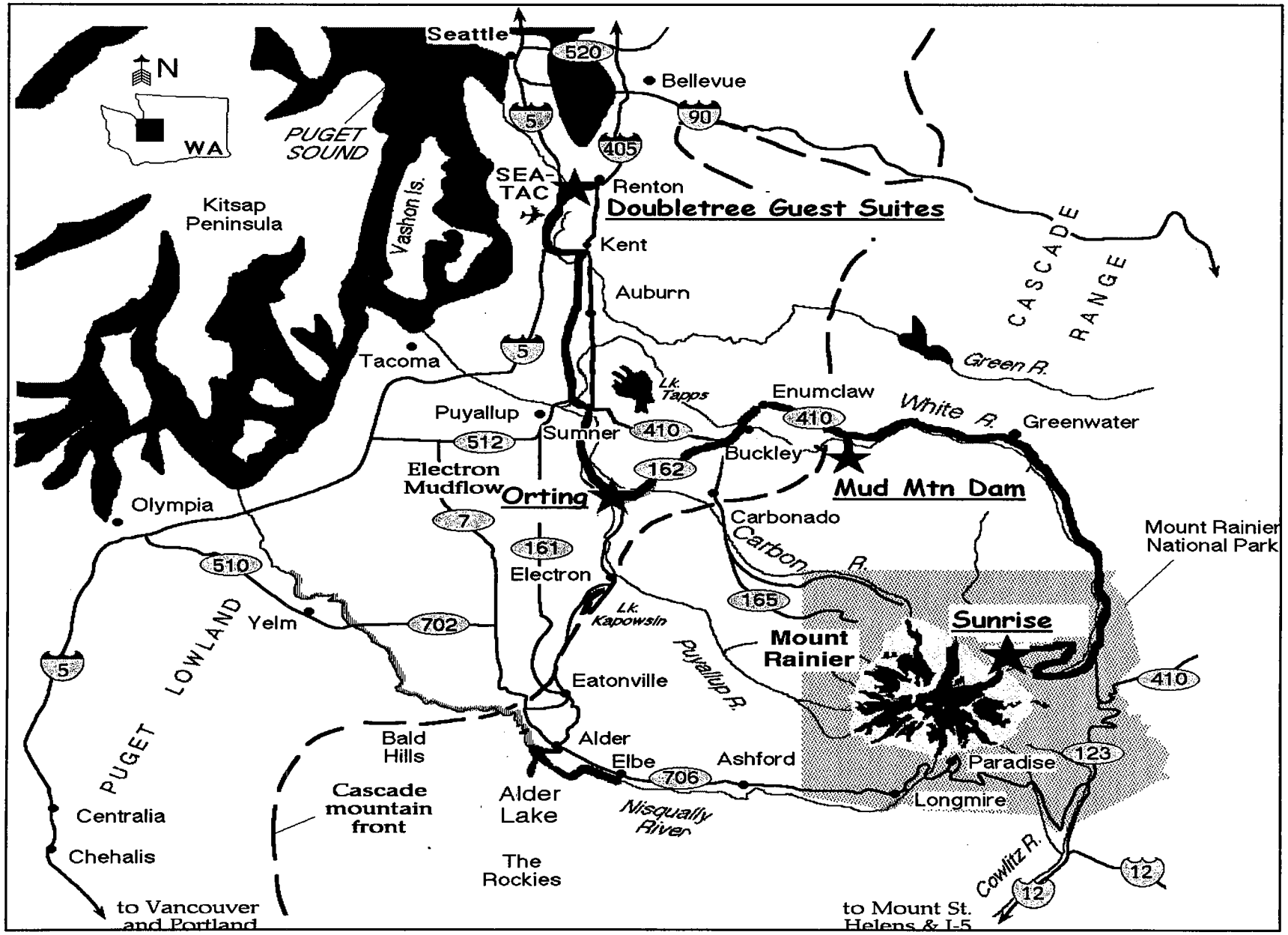

FIGURE 1

Mount Rainier

Highway Geology Symposium Field Trip 2000

Modified from "a Decade Volcano 1994 GSA Field Trip"

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Introduction

This trip journeys to the Sunrise area at Mount Rainier via the Duwamish, Puyallup, South Prairie Creek, and White River valleys. Along the way we examine a wide variety of geologic features, deposits, and geologic hazards (Fig. 1). Because of its hazards and increasing population pressures in the region, Mount Rainier was designated a Decade Volcano by the International Association of Volcanology and Chemistry of the Earth's Interior in 1992. In keeping with this designation, researchers have made considerable progress in examining the history, behavior, and hazards of the volcano. Decade Volcano investigations continue with the main goal of better understanding the volcano in order to mitigate risk.

This field trip provides an overview of what we presently know about Mount Rainier's geologic and glacial history. During the trip, we also will hear about recent research at the volcano and will inspect the geologic evidence for debris flows, the most important class of geologic hazards at Mount Rainier.

This text is a compilation of the work of numerous researchers, many of whom are continuing their work at Mount Rainier. We also have relied heavily on the work of Crandell, Mullineaux, Fiske, Hopson, Waters, and others whose earlier efforts have provided an excellent foundation for further studies.

All units are metric except road-log mileage. A note about National Park Service (NPS) etiquette: please do not do any digging within the boundaries of Mount Rainier National Park. Permits, which can be obtained from the Park Superintendent, are required for sample collecting.

Geologic History of Mount Rainier

Mount Rainier is the highest and third most voluminous volcano in the Cascade Range. It is potentially the most dangerous volcano in the range because of the increasingly large population living along its lowland drainages. These riparian areas are at risk because of the mountain's great relief and the huge area and volume of ice and snow on the cone (92 x 10^6 m^2 and 4.4 x 10^9 m^3 respectively; Driedger and Kennard, 1986) that could generate lahars during eruptions. In addition, enormous (>2 x 10^8 m^3) sector collapses of clay-rich, hydrothermally-altered debris from the cone have occurred at least 6 or 7 times since the Mount Mazama layer "O" ash was deposited (6,845 ± 50 yr B.P.; Bacon, 1983) and include the Osceola, Round Pass, and Electron Mudflows. Mount Rainier's steep, glacially carved slopes, hydrothermally altered core, active hydrothermal system, bedding characteristics (thin lava flows and interbedded volcaniclastics on generally outwardfacing dip planes), and exposure to pulses of tectonic and/or volcanic energy make all valleys surrounding Rainier susceptible to future sector collapses.

Prior to the 1990's relatively little was known of the eruptive history, composition, and age of Mount Rainier volcano compared to most other Cascades Range peaks. The most complete description of the pre-Holocene volcanic history of Mount Rainier has been made by Fiske and others (1963). Most of the Holocene history has been pieced together in greater detail through studies of its fragmental deposits, chiefly tephra, lahar, and glacial deposits (Crandell, 1963b, 1971; Mullineaux, 1974; Crandell and Miller, 1974; Scott and others, 1992).

Mount Rainier rests on middle Tertiary volcanic rocks of the Ohanapecosh, Stevens Ridge, and Fifes Peaks Formations. In the immediate vicinity of Mount Rainier, these rocks were gently folded along northwest-trending axes and then intruded by granodiorite and quartz monzonite of the Tatoosh plutonic complex. Descriptions of these rocks are found in Fiske and others, (1963), Vance (1987), and Hammond (1989). New evidence further constrains the age, location, and structure of the Fifes Peaks volcano and its stratigraphic relations with the other middle Tertiary rocks (Hammond and Brunstad, 1993; Hammond and others, 1991, 1993). The Tatoosh rocks are predominantly the eroded roots of a volcanic complex, from which the welded tuff exposed at the Palisades in Yakima Park is inferred to have erupted (Fiske and others, 1963). Radiometric ages of the Tatoosh rocks range from 17.5 to 14.1 Ma, although several ages as old as 26 Ma have been reported (Mattinson, 1977).

An apparent hiatus in volcanic activity and plutonism of more than 11 m.y. precedes the first evidence of a proto-Mount Rainier. This early thick sequence of volcaniclastic debris west of the mountain is known as the Lily Creek Formation. Constraining radiometric ages of this formation are 2.9 Ma and 0.84 Ma (Easterbrook and others, 1981; Smith, 1987). Crandell (1963a) discovered two distinct Lily Creek surfaces whose tops have different elevations. He suggested that the deposits composing the higher (older) surface correlate with mudflows of Alderton age. These deposits are northeast of Cowling Ridge and were deposited in the ancestral Mowich River valley. After the upper Mowich valley

was blocked by a lava flow from Mount Rainier, the lower depositional surface (southwest of Cowling Ridge) was created in the ancestral Puyallup River valley, and these sediments correlate with mudflow deposits of Puyallup age. The Alderton and Puyallup deposits are separated by the Stuck Drift of Crandell and others (1958). Sisson and Lanphere (1999) have noted a hornblende dacite of Panhandle Gap (shown as diorite on Fiske and Hopson's map) is 1.036—so perhaps some of Cliff Hopson's early speculations were right, and the Lily Creek was formed by eruptions from an earlier proto-Mt. Rainier. This 1 Ma dacite overlies Tertiary rocks unconformably and is overlain by typical Rainier rocks unconformably.

Construction and Destruction of the Mount Rainier Cone

Early lava flows of present Mount Rainier flowed onto a dissected surface of the Tertiary basement. Prior to the later 1990s, the only ages on lavas were only crudely known—the earliest lavas predated the Hayden Creek Drift (70140 ka) and had yielded (as of April, 1994) only two KAr ages, 320 and 600 ka (Crandell and Miller, 1974). Sisson, however (Sisson, 1999; Sisson and Lanphere, 1999) Small amounts of basaltic andesite were erupted during the late Pleistocene from two satellite cones on the northwest flank of the mountain, Echo Rock and Observation Rock. The mineralogy of the mafic inclusions, cumulate textures, and presence of minor amounts of glass suggest that at least one of these is cognate, or genetically related to Mount Rainier; others may be xenolithic (J. McKenna, Fig. 3). Approximately 140 km^3 of lava has been erupted from Mount Rainier in the past 1 million years (Sherrod and Smith, 1989).

A thick, biotite bearing pumice layer northeast, east, and southeast of the volcano is interpreted to have erupted from Mount Rainier about 384 ka (Sisson, commun., 1998). Although outcrops are limited, preliminary estimates suggest this so-called "U"- tephra layer is an order of magnitude larger in volume than any Holocene tephra layer from Mount Rainier. Holocene explosive eruptions at Mount Rainier produced 11 tephra beds (studied by Mullineaux, 1974) totaling more than 0.5 km^3 (Table 1). Roughly 30-40 percent of this volume (eight layers) was erupted between 6,500 and 4,000 radiocarbon years B.P. Some of the tephra layers (for example, layers S and F) are rich in lithic components and are thought to be the result of phreatic or phreatomagmatic eruptions. Layer F is unique among the postglacial tephras because of its large percentage of clay minerals (chiefly montmorillonite with some illite and kaolinite). The clay minerals were formed before deposition, and thus they were probably deposited by a violent phreatic or phreatomagmatic event that penetrated an area of hydrothermal alteration. Layer F is similar in age and clay content to the Osceola Mudflow (discussed below) but does not overlie the Osceola. The two deposits therefore seem correlative (Mullineaux, 1974).

Layer C, which accounts for about 60 percent of the volume of postglacial tephras, is also the most widespread, covering much of the eastern half of Mount Rainier National Park with 2-30 cm of lapilli, blocks, and bombs. It is also the coarsest of the Rainier tephras: 25-30 cm bombs can be found 8 km to the east of the summit. Apparently Columbia Crest, the 250mhigh summit cone, is younger than 2,300yrold layer C because the tephra

does not occur on its snowfree parts (Mullineaux, 1974).

During investigations of liquefaction in the City of Puyallup, one of the liquefiable sand units was identified as a lahar runout or laharic flood from Mount Rainier (Palmer and others, 1991; Pringle and Palmer, 1992). The presence of Mount Rainier "C" tephra and ^{14}C age of 2,320 ± 120 yr from a twig found in the deposit correlates that unit with newly discovered lahar deposits in upper reaches of the Puyallup River and with block and ashflow deposits noted by Crandell (1971) on the west flank of Mount Rainier. Deposits from that eruptive episode had not previously been discovered as far downstream as Puyallup.

Recent work on the C tephra (Venezky and Rutherford, 1993) provides evidence for magma mixing from analysis of phenocryst zonation, large temperature variations within and among pumice clasts, large matrix glass variations, physical mixing textures, and bimodal bulk rock compositions (Fig. 4).

In addition to the more pumiceous or scoriaceous tephras, more than 25 lithic tephra layers have not been investigated in detail (D.R. Crandell, written commun., 1992).

Postglacial deposits at Mount Rainier are dominated by lahars–more than 60 have been identified. Although relations between Holocene tephra and lahar deposits remain speculative, at least some lahars were probably eruption induced, such as the Paradise Lahar and Osceola Mudflow of Crandell (1971). The 5,700 ka **Osceola Mudflow** had a volume of more than 4 km^3, inundated at least 485 km^2, and flowed into Puget Sound more than 100 km channel distance Mount Rainier (Dragovich and others, 1994). As interpreted from well logs (neglecting minor relative sealevel changes), syn, and post-Osceola sedimentation has pushed the shoreline seaward 25 and 50 km respectively in two Puget Sound embayments, the Puyallup and Duwamish, and added more than 400 km^2 of new land surface (Dragovich and others, 1994) (Fig. 5).

Wood from trees buried in the Round Pass Mudflow has been dated at about 2,600 radiocarbon yrs B.P. This clay-rich diamicton is characterized by great thickness (locally >250 m), hummocky surface, and megaclasts of lithologically homogeneous material. It probably began as a debris avalanche of hydrothermally altered material from high on the western slopes, and most of it was deposited in the upper 20 km of the Puyallup River valley. Another clay-rich lahar, the Electron Mudflow, has been dated at about 530 radiocarbon years B.P. This lahar, which evidently began as a failure of part of the western edifice, has not been correlated with any eruptive activity at Mount Rainier. It and most other cohesive lahars may have occurred without precursory eruptive phenomena.

The **Electron Mudflow** was very fluid and underwent minimal downstream attenuation of discharge. This behavior is demonstrated by the relatively high peak stage of the lahar about 36 km west of Mount Rainier, slightly downstream of Puget Power and Light's Electron Power Plant. There, the Electron Mudflow was more than 30 m deep as it exited

the Cascade Mountain front and flowed onto the Puget Lowland. More than 40 trees were exhumed from Electron deposits at the town of Orting during the summer of 1993 and may provide clues to the exact age of the lahar.

Mount Rainier has a greater volume of snow and glacier ice than all of the other Cascade Range volcanoes combined (Driedger and Kennard, 1986). Several of Mount Rainier's 26 named glaciers (Fig. 6) have been the focus of classic studies of neoglacial moraine development and glacial dynamics (Sigafoos and Hendricks, 1961, 1972; Burbank, 1981; Porter, 1981; Heliker and others, 1984). Glacial outburst floods from South Tahoma Glacier have repeatedly scoured Tahoma Creek during the late 1960s and during the middle and late 1980s and early 1990s; the floods are usually associated with seasonally extreme weathereither unusually warm or unusually wet conditions (Walder and Driedger, 1994). The Tahoma Creek events are discussed in detail in the field guide. Similar events have occurred in many other drainages, most notably Kautz Creek and Nisqually River during historical time (Driedger and Fountain, 1989). Future lahars pose the greatest risk to populated areas near Mount Rainier, particularly on downstream flood plains of the Nisqually and Puyallup River valleys and to sections of the White River valley upstream from Mud Mountain Dam.

Hydrothermal System and Hydrothermal Alteration

Studies of the hydrothermal system at Mount Rainier indicate that a narrow, central zone of heat emission maintains snowfree areas at the summit craters and forms the caves in the summit icecap (Moxham and others, 1965; Frank, 1985). Heat flux is substantial in comparison to most other volcanoes in the Cascade Range, comparable to that at Mount Baker and Mount Hood.

Hydrothermal activity has played an important role in the construction and destruction of Mount Rainier. Hydrovolcanic activity (phreatic or phreatomagmatic eruptions) produces fragmental deposits. Hydrothermal alteration produces secondary hydrothermal minerals from primary volcanic minerals and results in changes in the permeability of the primary rocks.

Hydrothermal alteration occurs both in currently active thermal areas and in prehistoric deposits that show no evidence of lingering hydrothermal activity. These Quaternary hydrothermal areas and deposits are significant because they influence edifice stability and the type and size of debris flows that may form (Scott and others, 1990; Zimbelman and others, in press). Generally, these areas contain argillization and silicification alteration types, both as pervasive masses and as selective pockets, lenses, and veins (Frank, 1985; Zimbelman and others, in press). Several representative areas of alteration are:

Summit craters — An extensive area (>12,000 m^2) of heated ground and slightly acidic boilingpoint fumaroles occurs at East and West Craters on the volcano's summit. Maximum fumarole temperatures generally range from 76E to 86E C. Rocks in the

summit area contain alteration products with major concentrations of clay minerals including smectite, halloysite, and disordered kaolinite. Silica phases include cristobalite, tridymite, and opal. Other prominent phases are alunite, gibbsite, and calcite (Frank, 1985).

Upper Flank — A small area (< 500 m^2) of heated ground and subboilingpoint fumaroles lies on the upper flank at Disappointment Cleaver. Maximum temperatures at these fumaroles are about 60E C. Other areas of possible similar hydrothermal activity have been identified by infrared surveys, some of which have recently been field checked during a new USGS study as part of the Decade Volcano project. These occur at Willis Wall, Sunset Amphitheater, and the South Tahoma and Kautz headwalls. An area of fossil alteration is found at Sunset Amphitheater. There large areas of selectivelytopervasivelyaltered rock, are accompanied by minor veining.

Lower Flank — Sulfate, and carbon dioxideenriched thermal springs are present on the lower flank of the volcano on valley walls near the Winthrop and Paradise Glaciers. Maximum spring temperatures typically range from 9° to 24°C. The thermal springs deposit small amounts of calcite, opal, and gypsum as precipitation products that encrust gravel and cobbles in downstream channels.

Additional thermal activity occurs beyond the outcrop area of Mount Rainier andesite. Chloride and carbon dioxide enriched thermal springs described above issue from thin sediments that overlie Tertiary rocks in valley bottoms of the Nisqually and Ohanapecosh Rivers. Longmire Springs in the Nisqually River valley have maximum temperatures of 25°C and have produced an extensive area of travertine mounds and sulfiderich muds. The relative abundance of dissolved constituents at Longmire Springs is similar to that of higher altitude springs. All three sets of springs could conceivably be derived from similar acidic sulfatechloride waters that originate in a central, steam-heated hydrothermal system in the upper part of the volcano. Cooling of thermal waters during transit away from the hydrothermal system to lower elevations could take place by dilution with shallow cold ground water. The trend in composition from Winthrop to Paradise to Longmire Springs shows increasing ionic strength, progressive enrichment of calcium and chloride, and depletion of sulfate.

Hydrothermal alteration of Tertiary rocks in the Mount Rainier area is widespread. On Mount Rainier, the Glacier Basin area contains some of the most widespread alteration of Tertiary rocks found in the park. A copper-silver mining camp (now abandoned) was situated on vein and stock-worked areas of alteration in both Tertiary rocks and in surficial materials derived from those rocks. This area of alteration may have contributed some clay-size minerals to the Osceola Mudflow (Zimbelman and others, in press). Other areas of hydrothermally altered Tertiary rock are scattered throughout the park near the east and west sides of Winthrop Glacier, near the confluence of June Creek and the Carbon River, near Mowich Lake and the Mowich River, near Glacier Island and Pyramid Peak, and near the confluence of the Paradise and Nisqually Rivers

Seismicity

An average of about 30 small earthquakes occur under Mount Rainier per year, making it the most seismically active volcano in the Cascade Range after Mount St. Helens (Malone and Swanson, 1986). Malone and others (1991) note that more than 800 seismic events have been located within a 1,600 km^2 area centered on Mount Rainier during the past 20 yr. It is expected that eruptive activity at Mount Rainier would be preceded by a systematic increase in seismic activity.

Surficial events are another source of seismicity at Mount Rainier. The various types of events have been summarized by Weaver and others (1990).

Tectonic earthquake activity has led to the identification of a north trending fault zone west of Mount Rainier (Crosson and Frank, 1975; Weaver and Smith, 1983). The proximity of shallow crustal fault zones to Mount Rainier is significant because earthquake activity could cause slope failures at the volcano. For example, a debris avalanche at Ontake volcano in Japan was triggered by a regional tectonic earthquake (Nagaoka, 1987). Regional interplate or intraplate earthquakes also could trigger slope failures at the volcano.

Trilateration and tilt networks established on the volcano in 1982 have shown no significant displacements (Chadwick and others, 1985), and seismic activity is probably within the range of normal for a quiescent composite volcano that has an active hydrothermal system.

Debris Flow Processes at Mount Rainier

Numerous debris flows and related floods have inundated valleys and adjacent terraces far from Mount Rainier. From a process perspective, these flows have been categorized as cohesive or noncohesive, representing a continuum of debris flow behavior that relates empirically to clay content (Scott and others, 1992).

Typically originating as deep-seated failures of the volcanic edifice, cohesive debris flows generally contain more than about 3 percent clay-size particles and travel great distances (>100 km), undergoing minimal rheologic change en route. Debris flows are a major destructional process of the Mount Rainier cone, and they can be initiated by tectonic or magmatic seismicity or hydrothermal destabilization. The stability of the Mount Rainier edifice is in a state of flux because of glacial and fluvial erosion, glacier mass and volume changes, and ground water (pore pressure) fluctuations.

Noncohesive debris flows, containing less than about 3 percent clay sized particles, commonly form when flood surges, triggered by eruption generated snow melt, incorporate sediment and become debris flows. They may also form by shallow slope failures that contain less than the critical amount of clay. In their distal phases these flows transform from debris flows (>60 percent sediment by volume) to hyper-concentrated

flows (lahar run out flows) and then may be further diluted to normal stream flow (<20 percent sediment by volume). These types of debris flows may be a major source of liquefiable sands in valleys draining Mount Rainier (Palmer and others, 1991; Pringle and Palmer, 1992).

Debris Flows on Tahoma Creek 1986-1992

Since 1986, debris flows along Tahoma Creek have repeatedly damaged the Westside Road, the principal access route to National Park trails and facilities on the southwest side of Mount Rainier. Debris flows have obliterated a picnic area, parts of the Westside Road, and the lowest 1 km of the Tahoma Creek hiking trail. Their suddenness and rapid movement down-valley make them dangerous to any objects in their path. Several individuals have witnessed the debris flows, but no one has yet been injured. However, about 60 persons were stranded on July 14, 1988, when a debris flow destroyed sections of the Westside Road.

As of July 1994, fifteen debris flows have been recorded in the sequence that began in 1986, and a total of 23 since 1967. Some of those earlier flows were described by Crandell (1971), and the sedimentology of two recent flows was described by Scott and others (1992).

These relatively small events, while posing a hazard mainly to areas in the national park, have frustrated efforts to keep the Westside Road open and served as a reminder of the much larger, less frequent debris flows.

Field Trip Log

(Mileage increments between points are given at the end of each entry.)

Mileage

0.0 The field trip log starts at the intersection of State Route (SR) 516 and the valley freeway (SR 167) at Kent in the Duwamish valley. Go south on SR 167. The Duwamish valley was carved during and after the Vashon glaciation. An arm of Puget Sound extended up the Puyallup valley about as far as Sumner before deposition of the Osceola Mudflow about 5,000 yr B.P.. Tens of meters of debris flow deposits from Mount Rainier as well as fluvial deposits now underlie the valley floor. A study of well logs (Dragovich and others, in press) confirms that, at present sea level, a pre-Osceola Mudflow arm of Puget Sound in the Duwamish valley extended south to about 6 km north of the Sumner. Similarly, in the Puyallup valley an arm of the sound extended east to near the west side of the City of Puyallup. Increases in slope of the surface of the Osceola deposit and a dramatic decrease in thickness (from as much as 18 m slightly upstream of the delta top to <4 m on the delta front) mark the passage of the mudflow over deltas. Apparently there has been more than 400 km^2 of alluviation in the Puyallup and Duwamish valleys since the Osceola Mudflow. Near Main St., and just west of the SR 167 in Auburn, a buried forest was discovered during excavations at the Emerald Downs Racetrack in 1995. The outer rings of a deciduous stump buried in a 0.8 m-thick pumiceous volcanic sand (laharic flood) deposit yielded a radiocarbon age of 1100 yr B.P.; an equivalent sand deposit was later discovered in the lower Duwamish River valley (Pringle and others, 1997)—these laharic deposit, which are correlative with the "Deadman Flats" lahar assemblage noted upstream, are the result of a moderatly explosive eruption of Mount Rainier (Scott and others, 1995).

13.1 Exit right onto SR 410. Head east on SR 410 for about 2 mi and exit onto SR 162. Go south (right) toward Orting. We cross the Puyallup River about 0.6 mi south of SR 410. The farthest downstream outcrops of the Electron Mudflow (channel-fill deposit) and the Osceola Mudflow are here. The top of the Osceola deposit is about 7 m below the bridge. Many eyewitness accounts of liquefaction were documented in this area during the M 7.1 1949 earthquake (Chleborad and Schuster, 1990). Black sands composing the sand boils had originated in lahar runout deposits from Mount Rainier (Palmer and others, 1991; Pringle and Palmer, 1992).

Lower(?) Pleistocene laharic deposits of the Alderton and Puyallup Formations of Crandell and others (1958) are exposed in the valley walls near Alderton. The two units are separated by the Stuck Drift of Crandell and others (1958); all are reversely magnetized (Easterbrook, 1994) and thus lie within the Matuyama Reversed Polarity Chron (0.8 to 2.4 Ma).

20.0 Stop 1 - Orting Buried Forest

At the town of Orting turn right near the Orting Elementary School or Whitehawk Subdivision . This area was inundated by the Electron Mudflow about 530 yr B.P. Recent excavations on the west side of the highway have exposed remnants of a buried old growth forest (mostly Douglas fir) that was preserved within as much as 6 m of mudflow. Some of the excavated snags are lying on the surface, and several inplace snags stick up in the storm water retention pond north of the subdivision entrance. A 5 m-long chunk of andesitic volcanic breccia that was excavated near here was one of many large boulders carried by the flow.

The four innermost rings of a giant Douglas fir stump that was about 358 years old when it died yielded radiocarbon ages of 1010 ±50 and 950 ±50. After calibrating and offsetting the ages to reflect number of rings in from the outermost ring under bark (last ring produced before death of tree), the data can be combined with other radiocarbon ages for the Electron Mudflow to show that the event probably occurred near A.D. 1400. However, wiggles in the radiocarbon calibration curve yield multiple intercepts that allow considerable error. Preliminary analysis of tree rings, particularly the "latewood" or dark, dense cells that form during the summer, may indicate that the Electron Mudflow occurred in the early 1400s.

Development pressures have been intense in this area of the Puyallup Valley, which was designated as an area of moderate volcanic hazards by Crandell (1973). Recent legislation, the Washington Growth Management Act, (Title 36.70A RCW) mandates that all qualifying jurisdictions will (among other things) designate "critical areas" including geologically hazardous areas. The Washington Department of Community, Trade and Economic Development developed "minimum guidelines" for defining volcanic hazard areas (Chapter 365190 WAC, Section (4fi)). Local geology professor Al Eggers (University of Puget Sound) served on a citizens advisory board to the Pierce County Planning Department and was influential in the appraisal of volcanic (and other) hazards that have been considered in designing the county comprehensive land use plan.

At this stop Steve Bailey of Pierce County Emergency Management will discuss the county's role in preparedness efforts related to volcanic hazards. Pierce County has partnered with the US Geological Survey in a pilot project that involves the installation of a series of acoustic flow monitors (AFMs), essentially geophones that can detect vibrations in the range of 50-90 hz that would be produced by lahars coming down the Puyallup or Carbon River drainages (see attached USGS information sheet). When completed, the warning system will include sirens and a network of local FM radio stations that will broadcast hazards-related information in the case of an approaching lahar. Lahar warning signs are in the planning process.

The Mount Rainier Volcanic Hazards Working Group (MORAWOG) consists of representatives of federal, state, and local agencies, local governments, school systems, and members of the public and other organizations. This group has been meeting for more than 10 years and plans and sponsors educational and outreach efforts related to the hazards of Mount Rainier. The group has just completed a working draft of a response plan for Mount Rainier.

Continue south on SR 162 through the town of Orting.

24.7 At the Electron turnoff continue east on SR 162 up the valley of Prairie Creek, interpreted by Crandell to be the pre-Osceola valley of the White River.

27.0 New Carbon River bridge replaces the old trestle bridge that was partly undermined by flood erosion in 1990.

33.9 End of SR 162 at the intersection of SR 165 to Buckley and the Carbon River entrance to Mount Rainier National Park. This is the closest access to the park from Seattle. Turn left.

35.6 At the stop sign jog left and then turn right onto SR 410. At the towns of Buckley and Enumclaw, the highway traverses a plain underlain by the Osceola Mudflow. An occasional large boulder is visible on the surface of the plain, but for the most part it is smooth and the Osceola surface lacks the mounds visible in several localities farther upstream. Within a few kilometers of this area, an archeological excavation unearthed more than 200 knives, projectile points, scrapers and other artifacts beneath the Osceola Mudflow deposit, evidence of human occupation of the area before, and possibly at the time of the mudflow (Williams, 1973; Hedlund, 1976).

37.3 The 3-to-5m thick Osceola Mudflow caps glacial deposits of Vashon age in the 30m terrace. Inset against this terrace is a 16m terrace that includes two laharic units. The 6m terrace-capping lahar deposit contains rounded set C pumice from Mount Rainier.

Continue west on SR 410 through Enumclaw.

44.8 Stop 2 - Mud Mountain Dam.

At the junction of SR 10 and the Mud Mountain Dam access road, drive 2.4 mi on the access road and then a short distance left to the dam viewpoint. In the recent clear-cut near the intersection of the entrance road with SR 410, note the small hummocks or mounds in the Osceola Mudflow deposit. The mounds commonly consist of mega-blocks of rock debris either from Mount Rainier, or composed of non-Mount Rainier rocks that were picked up by the Osceola en route down the valley. At the viewpoint, the Osceola deposit is the thick rocky diamicton that caps

Vashon Till and advance outwash in the terrace exposed in a recent landslide scarp to the north. Note that this deposit has a flat bottom and uneven top. In their 1997 paper, geologists Jim Vallance and Kevin Scott, who have studied the Osceola Mudflow in great detail, note that:

"*When it (Osceola) encountered a narrow gorge of the White River at Mud Mountain that is only 300 m wide, the Osceola Mudflow spread out over glaciofluvial [and till] terraces of Vashon age that are up to 110 m above the White River...As it continued westward, the mudflow poured over terrace scarps to form a spectacular pair of falls...The upper fall would have formed an arc more than 6 km wide and more than 80 m high, and the second would have been more than3 km wide and more than 110 m high.*"

Lower, inset terraces that are visible from the viewpoint are lahars of the Deadman Flat assemblage, noted earlier. Upstream of here about 1 km, these deposits, which mainly consist of sandy, noncohesive, probably eruption-triggered lahars from Mount Rainier, veneer terraces as high as 65 m above the White River. Mud Mountain Dam was built by the Corps of Engineers solely for flood control in the lower White River basin. The earthfill embankment dam was constructed between 1939 and 1942, although flood gates were not constructed until 1948. Additional information about the dam can be found in Galster (1989a, 1989b).

The Three Sisters, the cone-shaped peak to the southeast with a radio tower on it's summit, has been identified by Tom Sisson of USGS (personal commun., 1998) as basaltic andesite vent with agluttinated spatter and possible feeder dikes. The volcano yielded a K/Ar age of 359 ka.

47.4 Roadcut on north side has hydrothermally altered Tertiary andesite estimated to be early Miocene by Hammond (1980). The rock is rich in alunite. A nearby caprock serves as a silica resource for the Superior quarry, one of Washington's three silica mines.

50.3 Clearcut on north side of highway is on a large Quaternary landslide Additional alunitized zones occur along north-side roadcuts for the next 5 mi. Hammond mapped the northwest-trending Twin Creek fault, perhaps the pathway for alteration fluids in Tertiary andesites, along the slope to the north of the highway.

53.4 The road curves to the right. A quick view of the White River, adjacent to the highway, shows the high turbidity that results from glacial rock flour and other suspended sediments. The road passes over Twin Creeks. After about 0.8 mi, turn

left on a spur road.

56.2 Federation State Forest. The road here is on a terrace underlain by lahars of the Deadman Flat assemblage, noncohesive lahars from Mount Rainier, the largest of which has been dated at 1,120 ± 80 yr B.P. (Scott and others, 1992). The largest lahar in this assemblage was at least 18 m deep at this location (Fig. 9). As noted above, it is now known that the Deadman Flat lahars inundated the lower White River and Duwamish River valleys as far as the Port of Seattle.

At the Federation Forest State Park campground south of the highway, a water supply well penetrated the Osceola Mudflow at a depth of 61 m; the top of the Osceola is 11 m above the White River (Crandell, 1971).

57.6 Town of Greenwater. Lahars have left abundant deposits in this area, perhaps because the valley is fairly constricted downstream of here. The West Fork White River joins the White River about 2 mi to the east. This river drains the north slope and heads at Winthrop Glacier. The West Fork valley was one of the pathways of the Osceola Mudflow.

65.5 Entrance to Mount Baker/Snoqualmie National Forest.

65.9 View of the northeast slope of Mount Rainier through the valley of Huckleberry Creek.

70.4 Dalles Campground.

71.3 Light-colored bedrock outcrops east of the road are Stevens Ridge Formation volcaniclastics of Fiske and others (1963). This formation ranges from late Oligocene to early Miocene. Source of this unit is unknown.

75.8 Dark-colored bedrock outcrops east of the road are Ohanapecosh Formation volcaniclastics of Eocene-Oligocene age.

76.4 Silver Springs. A water supply well for a USFS ranger station penetrated the Osceola Mudflow at a depth of about 15 m and was still in it when drilling stopped at 61 m. Nearby the mudflow also veneers the valley sides 38 m above the river (Crandell, 1971).

77.4 **Mount Rainier National Park boundary.**

79.3 Vertical contact between Ohanapecosh Formation and Sunrise Point pluton. This is the northern boundary of the Sunrise Point pluton. Eocene Ohanapecosh Formation lavas and breccias make up the country rocks. The contact slices across contours on either side of the White River and is only mildly altered, unlike the hornfelsed Ohanapecosh rock farther upstream. The Sunrise Point pluton is a quartz monzonite at this location and will be described at the type locality, MP 4.0 of the Optional trip to Sunrise.

81.9 **Turnoff, Yakima Park Road to Sunrise veers off to southeast.** View is up the White River which heads at Emmons Glacier and drains the east side of the volcano. The White River was a major pathway for the Osceola Mudflow. Orange diamicts in roadcuts along the valley floor are scattered remnants of the Osceola.

84.5 Shay Creek

87.1 **Junction of Yakima Park Road and White River Campground Road.** Turn left into the campground and drive to the far west end. The clay-rich Osceola Mudflow overlies debris flow deposits at the road junction. Porter and others (1965) estimated that the mudflow was at least 210 m thick here.

Road to Sunrise Park

87.1 The Osceola Mudflow is locally visible as a <10 m veneer for about the next 1.0 mi with tephra layers Yn, C, and Wn visible on top of it in some outcrops. Spectacular columns of Mount Rainier Andesite, as well as till, a tephra of pre-Evans Creek age (>38 ka), and beds of fine-grained material (glacial-lake deposits) are visible between road miles 1.7 and 2.1.

88.9 Pull-off to see tephra deposits underlying Evans Creekage glacial deposits.

89.9 **Yakima Park flow of the Mount Rainier Andesite.** This flow is one of the longer intra-canyon flows preceding the main cone-building stage of the volcano. (See Fiske and others, 1963.) The flow apparently followed the course of the ancestral White River and chilled against the walls to form the picturesque columnar joints exposed in the road cut.

91.1 Sunrise Point switch-back parking lot. This is the type area of the Sunrise Point quartz monzonite of Mattinson (1977). At most outcrops the rock is a light-grey to greenish-grey, medium- to fine-grained, equigranular quartz monzonite composed of plagioclase in a granophyric matrix of K feldspar and quartz. Biotite, chlorite and hornblende are less common. Magnetite, ilmenite, titanite, and epidote are also present. The rock is commonly altered. UPb zircon data (Mattinson 1977) suggest an age of 24 Ma, considerably older than the White River pluton. From this last

97.2 Stop 3 - Sunrise Parking Lot - Lunch and optional hike

This optional hike goes north from the parking area and up about 400 ft (120 m) onto Sourdough Ridge to a saddle. There we will view an outcrop of a biotite-phyric pumice deposit that may record the largest known explosive eruption of Mount Rainier. The 2m thickness of this unit 12 km northeast of the present Mount Rainier summit is greater than that of tephra layer Yn (3,600 yr B.P.) at an equivalent distance from Mount St. Helens (Mullineaux, 1986). The volume (dense rock equivalent) of layer Yn has been estimated at 4 km^3 (Carey and others, 1989). The tephra clasts are crumbly and many represent fragments of much larger clasts. Sisson obtained a K/Ar age on the pumice of about 384 ka (personal commun., 1998). The origin of a fragmental deposit overlying the tephra is unclear. Is this explosion ejecta or possibly part of a glacial moraine?

The crest of Sunrise Ridge is composed of lavas and breccias of the Mount Fremont dome complex. The Mount Fremont aphanite (Fiske and others, 1963) is andesite to dacite in composition with plagioclase the only abundant phenocrysts. Rare augite and magnetite/ilmenite are also found. This unit has not been dated; however, it is probably slightly older than the 25.1 Ma (Mattinson, 1977) tuff of the Palisades, exposed about 3 km to the north. Mapping of crumble breccia, vent-filling tuff breccia, aligned vesicles and flow foliation (Murphy and Marsh, 1993) reveals at least two vents in this vicinity, sub-parallel and NWSE trending. A good cross section of the dome complex can be seen during the short (4 km) hike to the Mount Fremont fire lookout. Fine-grained, laminated, well-indurated rocks are found on the tops of McNeeley Peak and Dege Peak, probably either Fifes Peak Formation-equivalent (Fiske and others, 1963) or associated with late-stage dome explosion or epiclastic degradation (Murphy and Marsh, 1993).

The optional 5km loop hike to Frozen Lake transects the excellently exposed upper contact of the Sunrise Point pluton and the overlying dome complex. Look for the granitic rocks along the creek draining Frozen Lake, but try to stay on the trail elsewhere. The meadow biota are highly sensitive to foot traffic.

The Emmons Vista trail (0.3 mi round trip) leads to several scenic overlooks.

This concludes the field trip.

Layer	Age (yr B.P.)	Predominant Materials	Volume
X	+-150	Pumice	1
C	2,300	Pumice, scoria, lithic fragments	300
B	>4,000	Scoria, lithic fragments	5
H	>5,000	Pumice, lithic fragments	1
F	5,000	Lithic fragments, pumice, crystals, clay	25
S	5,200	Lithic fragments	20
N	5,500	Lithic fragments, pumice	2
D	6,000	Scoria, lithic fragments	75
L	6,400	Pumice	50
A	6,500	Pumice, lithic fragments	5
R	>8,750	Pumice, lithic fragments	25

EXOTIC TEPHRA UNITS FOUND AT MOUNT RAINIER

Layer	Age	Volcano
Wn	A.D. 1479	Mount St. Helens
P	2,500	Mount St. Helens
Yn	3,500	Mount St. Helens
O	6,845	Mount Mazama

TABLE 1: Tephra layers at Mount Rainier, after Mullineaux (1974). Ages of X and Wn determined from tree rings; all others in radiocarbon years. Volume in millions of cubic meters. Precise calendric age of Wn is from Yamaguchi (1983, 1985) and Fiaccao and others (1983); ages of P and Yn from Mullineaux (1986); age of O from Bacon (1983).

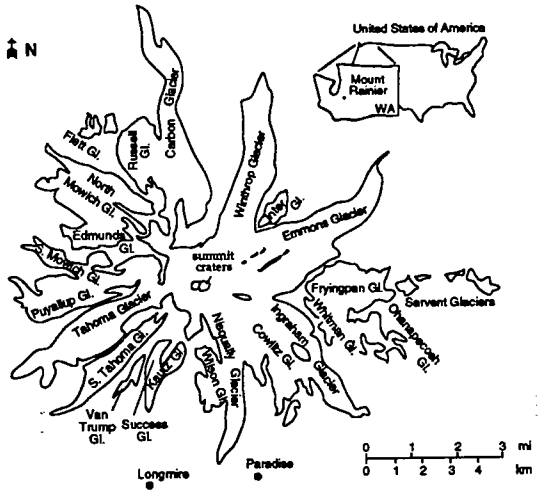


FIGURE 6: Glaciers of Mount Rainier, circa 1983.

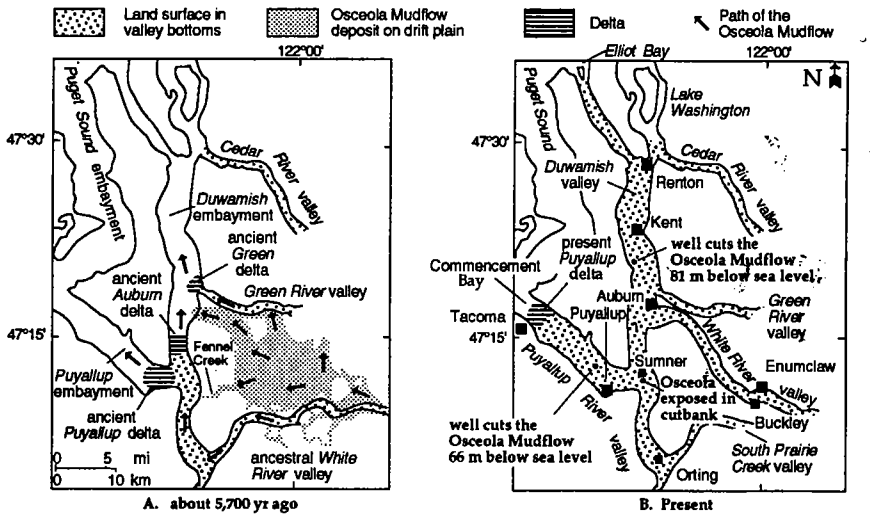


FIGURE 5: Comparison of the modern shoreline of Puget Sound (B) with that just after deposition of the Osceola Mudflow (A). Dark stippling shows distribution of the drift plateau. From Dragovich and others (in press), modified from Luzier (1969).

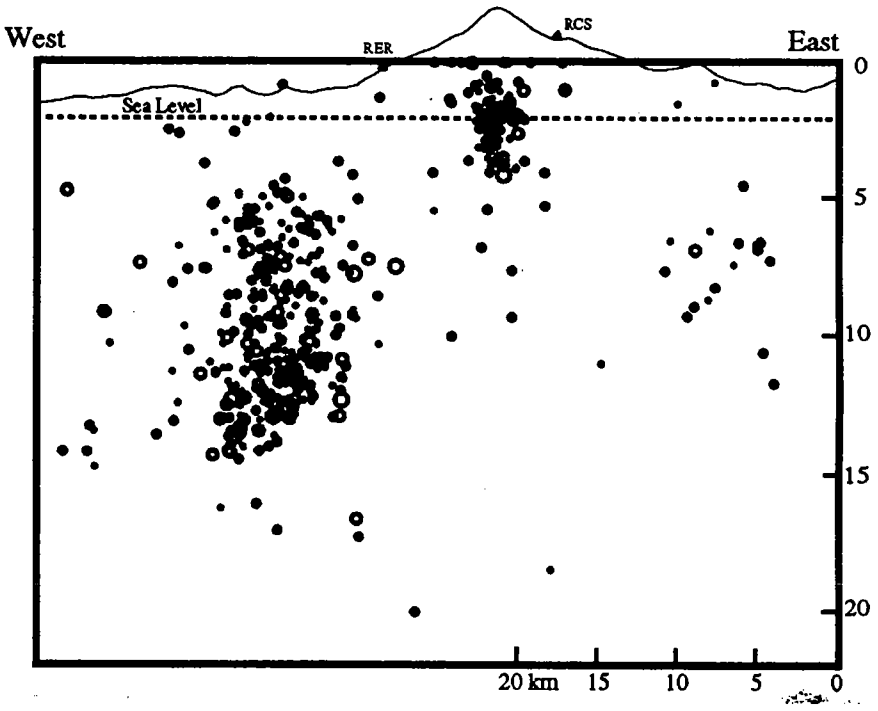


FIGURE 7: East-West profile of Mount Rainier area showing hypocenters of earthquakes in the University of Washington's seismic network catalog from 1969 - 1990. Triangles PER and RCS are two of the five seismic stations now in operation of the flanks of the volcano. Zone of earthquakes located shown west of Mount Rainier delineate a north trending crustal fault zone about 15 km west of Mount Rainier.

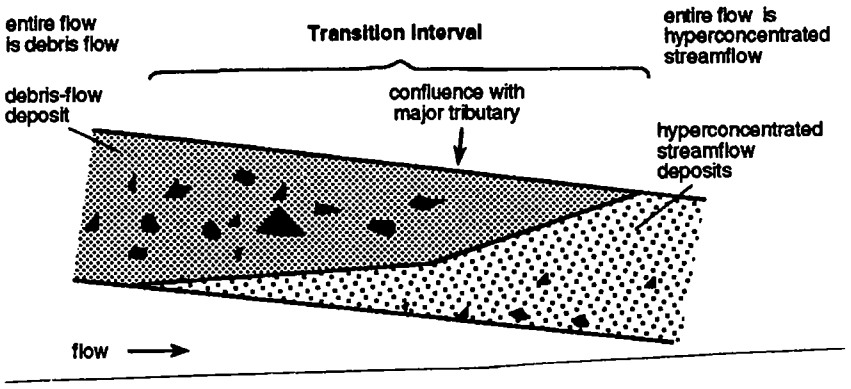


FIGURE 8: Diagram showing downstream textural changes that result from transformation of debris flow to hyperconcentrated streamflow. Accelerated changes have been observed where tributary streams add more water for the dilution process. Modified from Scott (9185)

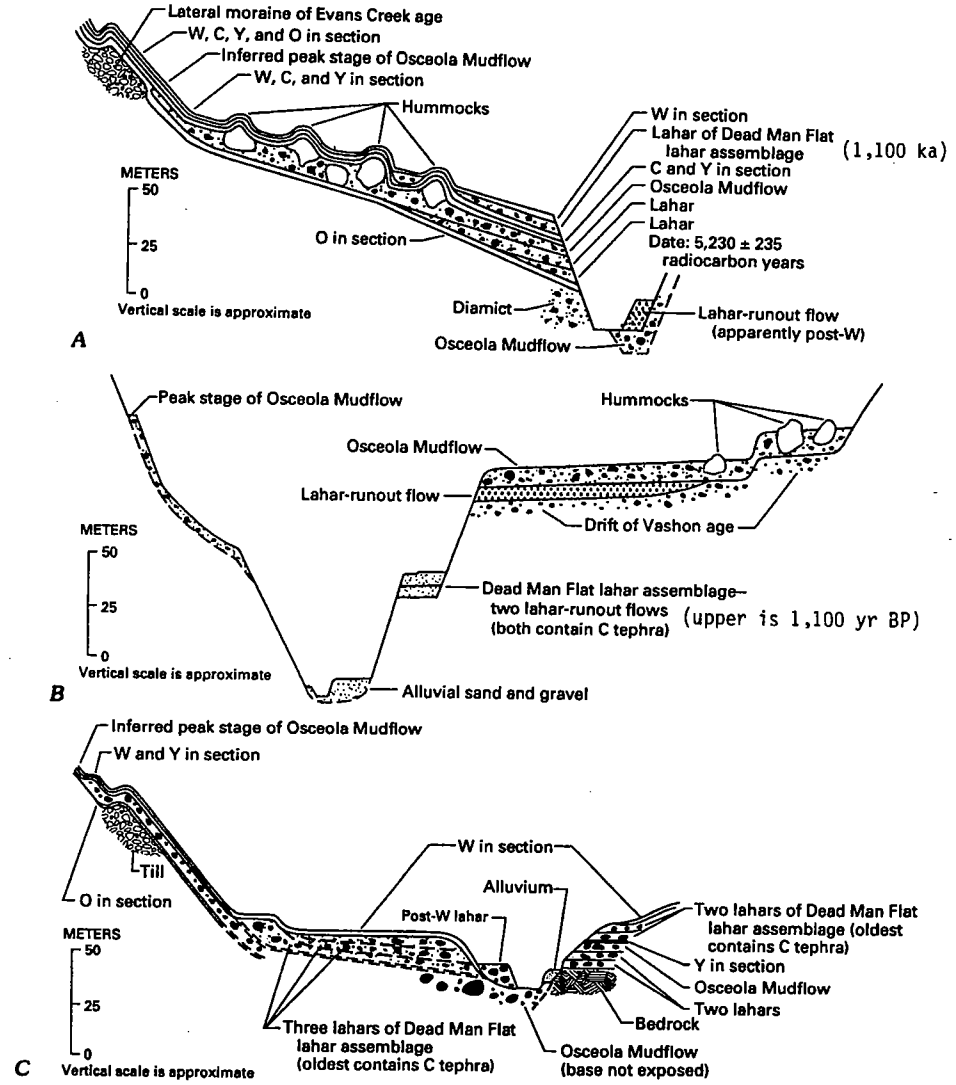
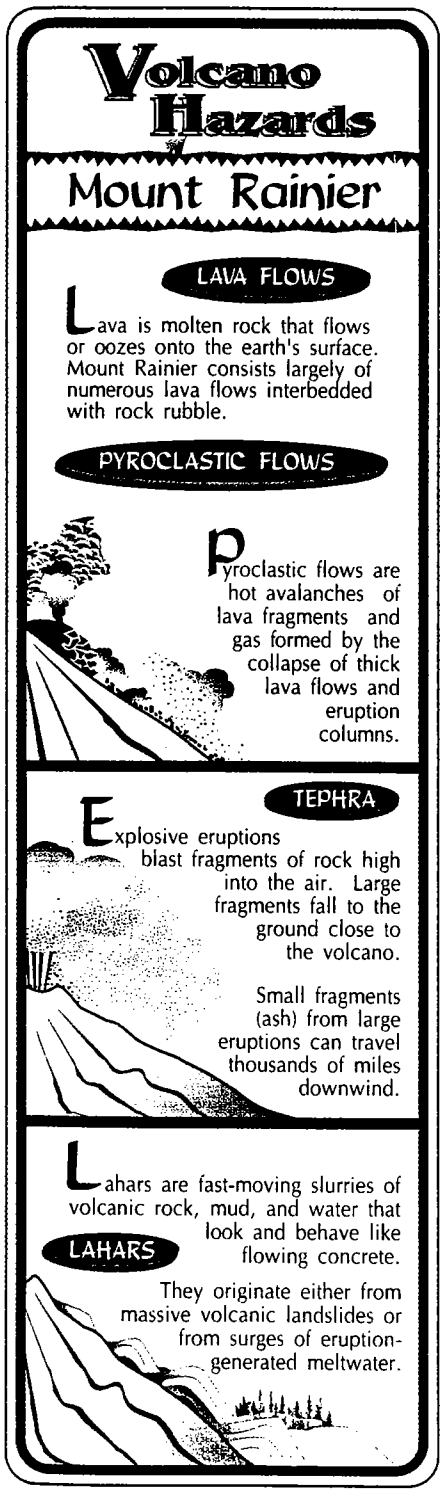


FIGURE 9: Diagrammatic composite sequences of valley-fill deposits in the White River Valley (successive downstream sections). Vertical scale is shown to indicate approximate thickness. Horizontal scale is variable for better portrayal of stratigraphic relationships and is not shown. However vertical exaggeration ranges approximately between 5x and 10x. A, White River between Buck and Huckleberry Creeks. B, White River at Mud Mountain Reservoir. C; West Fork White River between 25 km from Mount Rainier and confluence with mani fork of Whit River. *From USGS prof.paper 1547 by Scott, Vallance, and Pringle (1995)*

USGS
science for a changing world

U.S. GEOLOGICAL SURVEY— REDUCING THE RISK FROM VOLCANO HAZARDS

Mount Rainier– Living with Perilous Beauty



Mount Rainier is an active volcano reaching more than 2.7 miles (14,410 feet) above sea level. Its majestic edifice looms over expanding suburbs in the valleys that lead to nearby Puget Sound. USGS research over the last several decades indicates that Mount Rainier has been the source of many volcanic mudflows (lahars) that buried areas now densely populated. Now the USGS is working cooperatively with local communities to help people live more safely with the volcano.

FIGURE 1.—*View across Puyallup River valley toward Mount Rainier (Photograph by David Wieprecht.)*

What are the hazards?

Mount Rainier (fig. 1) is an active volcano that is currently at rest between eruptions. Its next eruption may produce volcanic ash, lava flows, or pyroclastic flows. The latter can rapidly melt snow and ice, and the resulting meltwater torrent may produce lahars (the widely used Indonesian word for volcanic mudflows) that travel down valleys beyond the base of the volcano to areas now densely populated. Lahars may also occur during non-eruptive times–without the profound seismicity and other warnings that normally precede eruptions.

Lahars look and behave like flowing concrete, and their impact forces destroy most man-made structures. At Mount Rainier, they have traveled 45-50 miles/hr at depths of 100 feet or more in confined valleys, slowing and thinning in the wide, now-populated valleys.

At Mount Rainier, lahars are a greater hazard than other volcanic products such as lava and poisonous gases that have been popularized by TV and film. Lava flows and pyroclastic flows are unlikely to extend more than a few miles beyond the National Park boundaries. Volcanic ash (tephra) will be distributed downwind, 80 percent of the time toward the east away from large populations (fig. 2).

The USGS, in cooperation with the University of Washington, monitors many Cascade Range volcanoes, including Mount Rainier, to detect precursors to eruptive activity. Mount Rainier was last active in the 19th century; one or more small eruptions produced local ashfall from one of the summit craters.

Why is Mount Rainier the most dangerous volcano in the United States?

Although Mount Rainier has erupted less often and less explosively in recent millennia than its neighbor, Mount St. Helens, the proximity of large populations makes Mount Rainier a far greater hazard to life and property.

1. *The population at risk*--More than 100,000 people reside on the deposits of previous lahars. The risk that an individual structure will be impacted by a lahar from Mount Rainier in much of this area is comparable to its risk of damage by fire.
2. *The size and frequency of lahars*--During the past few millennia lahars that have reached well into the Puget Sound lowland have recurred, on average, at least every 500 to 1,000 years. Smaller flows occur more frequently. A flow with a 500-yr return period has about a one in seven chance of occurring in an average human life-span.
3. *We may not have advance warning*--USGS research shows that some lahars may occur with little or no warning. Our only warning could be a report that a flow is under way.

How are lahar deposits identified?

A lahar flowing down valley from Mount Rainier leaves a thick valley-bottom deposit of boulders and hardened mud that may envelop stumps and logs of a buried forest. Some of the deposits can be traced upstream to the volcano's flanks, and all contain volcanic fragments unique to Mount Rainier. Geologists map the deposits and determine the tree ages to learn when the trees were engulfed and killed by the lahar. Old-timers recall encountering huge buried stumps and logs when plowing fields and digging wells. The youngest such forest was buried about 500 years ago and uncovered during excavations for new homes in the Puyallup River valley.

How and why do lahars form?

The most dangerous lahars start as enormous landslides--flank collapses--

Probability of Tephra Accumulation

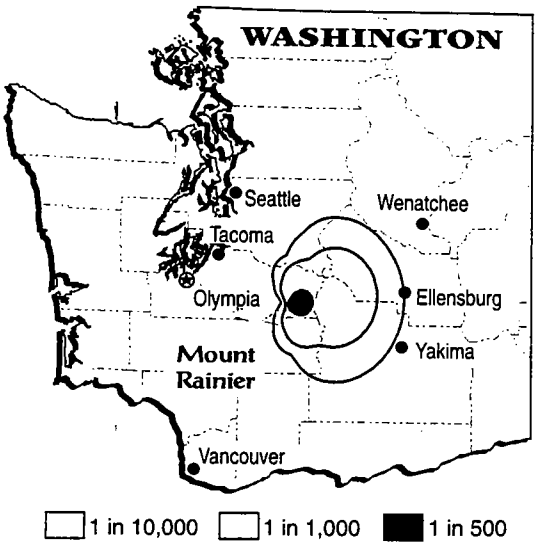


FIGURE 2.—*Map showing the annual probability that volcanic ash will be deposited to a thickness of 1/3 inch or more from an eruption of Mount Rainier. Volcanic ash, of this thickness or less, can cause disruption of ground and air transportation, and can cause damage to electronics and machinery.*

from the slopes of the volcano. Hot, acid water percolates through the volcano and slowly changes hard volcanic rock into soft rock that is clay-rich in a process called hydrothermal alteration. The mountain progressively weakens, like a house infested with termites. Eventually, a section of the volcano collapses, perhaps unexpectedly, and the muddy, water-rich mass transforms rapidly into a muddy slurry--a clay-rich, or cohesive, lahar (Case 1 in figure 3)--that is funneled at high speed into one or more surrounding valleys.

Sand-rich, or noncohesive, lahars form during eruptions of Mount Rainier when hot pyroclastic flows melt snow and ice. (Mount Rainier supports more than one cubic mile of glacial ice--as much as all other Cascade Range volcanoes combined.) These lahars may be less of a risk than cohesive lahars because they commonly have been smaller, although more frequent in the volcano's history (Case 2 in figure 3). Because noncohesive lahars occur with volcanic activity, they are likely to be preceded by events that will warn of an impending eruption; thus, they will be expected.

Small lahars, only a few miles long are caused by local avalanches of rock debris, sudden releases of glacial meltwater, or storms. These lahars (Case 3 in figure 3) happen commonly–many times each century.

New lahar deposits may get redistributed downstream over a period of many years as the disrupted drainage network becomes re-established. Thus valley-floor areas that were not impacted by the initial lahar deposits may suffer enhanced flooding and progressive burial by remobilized sediment (zone of post-lahar sedimentation in figure 3).

What triggers a flank collapse?

Flank collapses can be triggered when magma intrudes into a volcano and destabilizes it, as happened at Mount St. Helens in 1980. However, scientists studying the timing of flank collapses find that some flank collapses were apparently not accompanied by eruptions. They fear that a collapse could happen suddenly during non-eruptive periods and without the swarms of small earthquakes and other detectable phenomena that typically accompany rising magma.

A large earthquake unrelated to rising magma could trigger a flank collapse on Mount Rainier. At least one flank collapse at Mount Rainier could have been caused by a great prehistoric earthquake that occurred at about the same time.

A neighboring volcano--Mount Baker--produced flank collapses in the 1840's that were apparently triggered by steam explosions. Steam explosions at Mount Rainier could possibly trigger flank collapses and lahars with no advance warning.

Some flank collapses and lahars may happen with no obvious trigger. They may simply result from progressive hydrothermal alteration of the rock, saturation by groundwater, and the continuing pull of gravity.

What areas are at risk?

The courses of lahars will be the river valleys that drain Mount Rainier. Four of the five major river systems flow westward into suburban areas of Pierce County. These flow pathways are mapped by the USGS (fig. 3), just as flood-inundation maps show the areas at risk of flooding. Lahars occurring during an eruption may affect valley areas miles from the volcano, but a precursory warning should allow ample time to prepare for evacuation.

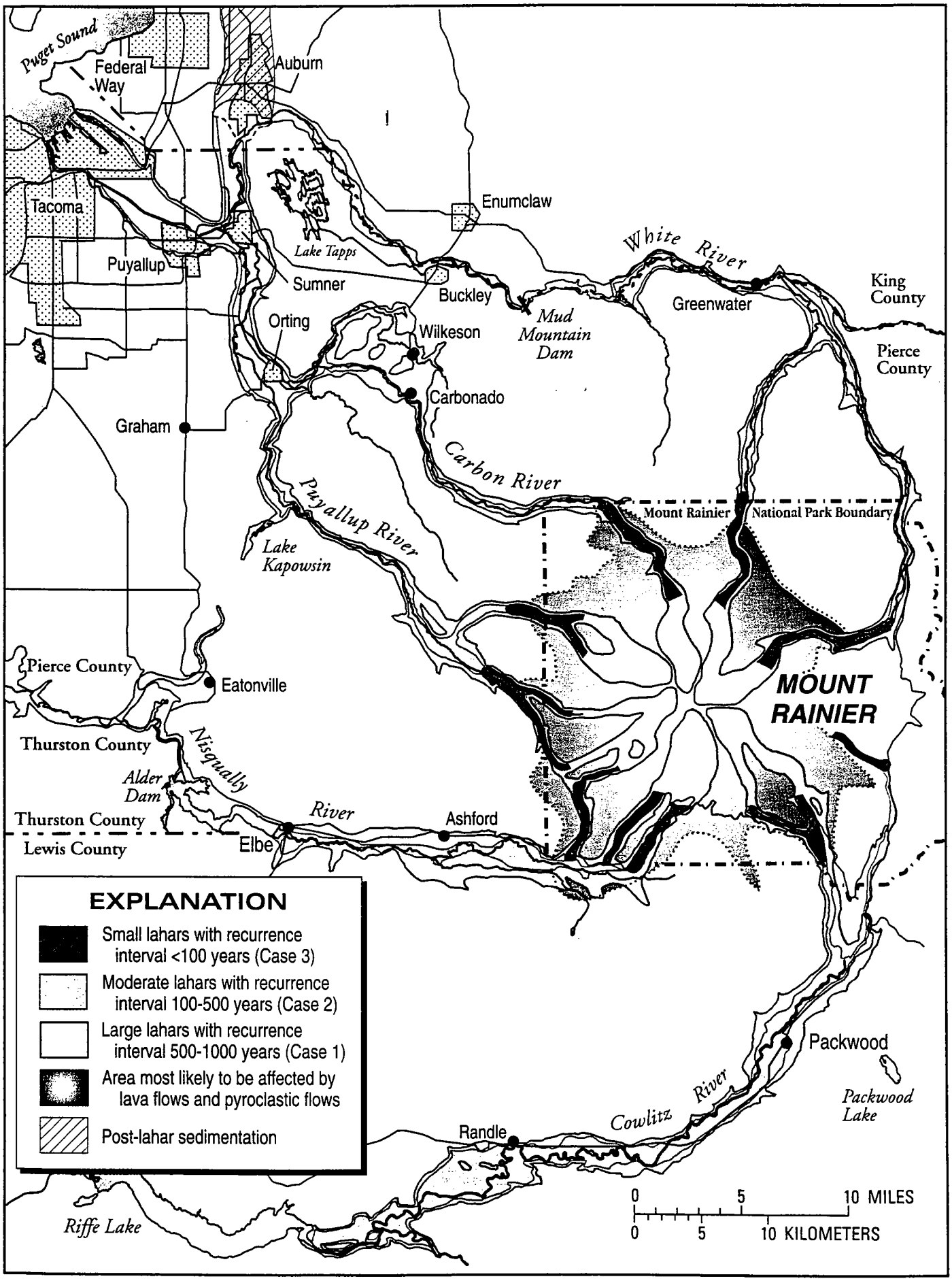


FIGURE 3.—*Hazard zones for lahars, lava flows, and pyroclastic flows from Mount Rainier (Hoblitt and others, 1998; US Geological Survey Open-File Report 98–428).*

A catastrophic flow will likely spread into multiple drainages. The largest known flow entered all five drainage sectors, and most of the known large flows have entered two or more. Drainage sectors converge toward the summit and are separated by sharp, unstable divides. Lahars have been as much as 800-1000 feet deep on the flanks of Mount Rainier, overtopping divides even when originating in one drainage.

Are all parts of the volcano susceptible to landslides?

All flanks of the volcano are susceptible to landslides and rockfalls. However, the east and west flanks are the least stable and have been the source of the largest landslide-generated lahars because of an east-west-trending dike and fracture zone that promotes circulation of hydrothermal fluid in those sectors. In addition, large lahars commonly enter more than one drainage, placing multiple valleys at risk.

How are we responding to the risks?

The USGS, in cooperation with the University of Washington, monitors the state of the volcano and assesses eruptive and hydrologic hazards stemming from volcanic activity.

The lahar pathways mapped by the USGS guide the hazard-area regulations of the comprehensive land-use plan for Pierce County. The plan's urban growth boundary and its proposed land uses in unincorporated areas are designed to minimize population growth, where possible, within hazard zones.

Local, county, state, and federal agencies including the USGS have joined to develop a Mount Rainier volcanic hazards plan that, when complete, will address such issues as emergency-response operations and strategies for expanded public awareness and mitigation of volcanic hazard.

What about a lahar-warning system?

A system for automatic detection and notification of an unanticipated flank-collapse lahar could reduce, but not eliminate, the risk in the lahar pathways. Two critical parts of such a detection and warning system include: (1) seismic detection and automatic notification of a major collapse event on Mount Rainier; and (2) detection, location, and automatic notification of a lahar in progress.

1. Seismic detection of collapse—work is under way to structure the seismic network run cooperatively by the University of Washington and the USGS to discriminate the seismic signal of a major collapse event on Mount Rainier and issue an automatic notice.
2. Lahar detection—the acoustic flow monitor (AFM) developed by USGS scientists can detect the ground vibrations of a lahar. The USGS and Pierce County Department of Emergency Management, together, are installing a pilot lahar-detection system in the Puyallup and Carbon River valleys. Computerized evaluation of data from arrays of five AFM's in each valley are expected to confirm either presence or absence of a flowing lahar and issue an automatic alert.

How much warning will we have?

Travel time of a lahar increases with distance. Thus the travel time of a large lahar from Mount Rainier to the lowland valleys is estimated at 1 to 2 hours, whereas the travel time to the nearest area of large population may be as little as 45 minutes. However, as the AFM arrays must be located beyond the volcano flanks, as little as 30 minutes, after warning from the pilot lahar-detection system, may be available for evacuation of urban areas closest to Mount Rainier in the Puyallup valley. Time is short, and successful evacuation will depend on detection of the approaching lahar, effective notification of people at risk, public understanding of the hazard, and practiced response by citizens.

What can you do?

Learn: Determine whether you live, work, or go to school in a lahar hazard zone. Learn about all volcanic processes that could affect your community.

Plan: Develop an emergency plan with your family so that you are prepared for natural hazards and emergencies.

Inquire: Ask public officials to advise you about how to respond during any emergency.

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by
K.M. Scott, E.W. Wolfe, and C.L. Driedger

Layout design and modification of graphics by *Christine Janda.*
Original graphics by *Lisa Faust*

Mount Rainier is an active volcano...

Reducing population growth in the paths of lahars, implementing a warning system, and planning and practicing evacuations can lower the potential loss of life and property during future eruptions and lahars. These actions can reduce the risk from lahars and provide a measure of safety for those who enjoy living, working, and playing in valleys surrounding Mount Rainier.

For up-to-date information contact:
U.S. Geological Survey
Cascades Volcano Observatory
5400 MacArthur Boulevard
Vancouver, WA 98661-7095
Telephone: 360-993-8900
World Wide Web:
http://vulcan.wr.usgs.gov

See also *Volcano Hazards from Mount Rainier, Washington* (U.S. Geological Survey Open-File Report 98–428)

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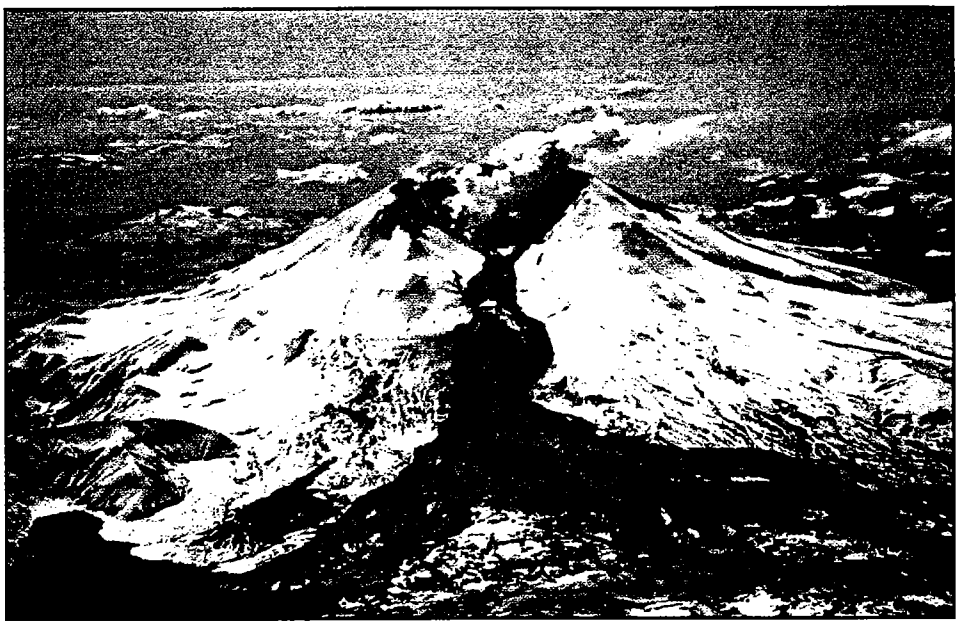
U.S. GEOLOGICAL SURVEY— REDUCING THE RISK FROM DEBRIS FLOWS

Detecting Debris Flows Using Ground Vibrations

Debris flows are rapidly flowing mixtures of rock debris, mud, and water that originate on steep slopes. During and following volcanic eruptions, debris flows are among the most destructive and persistent hazards. Debris flows threaten lives and property not only on volcanoes but far downstream in valleys that drain volcanoes where they arrive suddenly and inundate entire valley bottoms. Debris flows can destroy vegetation and structures in their path, including bridges and buildings. Their deposits can cover roads and railways, smother crops, and fill stream channels, thereby reducing their flood-carrying capacity and navigability.



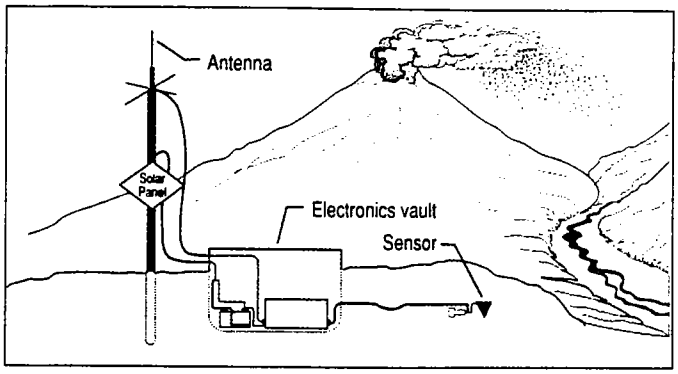
Large debris flow triggered by a small, explosive eruption at Mount St. Helens on March 19, 1992. Most of the flow went down the North Fork Toutle River, eventually reaching the Cowlitz River, 50 miles (80 kilometers) downstream. (Photo by Thomas J. Casadevall, March 21, 1982)

Detection of debris flows close to their sources can provide timely warnings to people in downstream areas if adequate communication systems exist. Unlike some other volcano hazards that are not constrained by topography, such as ashfalls and pyroclastic flows, debris flows are usually contained in valley bottoms and follow predictable paths along stream channels. In 1985, a relatively minor eruption of Nevado del Ruíz, Colombia, melted snow and started debris flows which entrained ash, soil, and debris. Within hours, the ensuing debris flows had traveled nearly 100 km down stream channels, left more than 23,000 people dead, and destroyed more than 5,000 homes. Similar, although smaller, tragedies are caused repeatedly by debris flows at volcanoes throughout the world. Thousands of lives might have been saved at Nevado del Ruíz, and hundreds around Mount Pinatubo, Philippines, had people received warning of approaching debris flows and known where to seek safety.

Previous Instrumental Monitoring of Debris Flows

Several approaches using electronic instruments have been used in the past to detect and monitor debris flows at active volcanoes. Trip wires are lengths of wire that are broken by a passing flow, and have been used with only limited success. If installed at different levels up the side of a valley, they can detect the maximum depth of a flow according to the level of the highest wire that has been broken. If installed at successive downstream sites, they can detect the progress and speed of the flow front. Such trip wires are difficult to install and are subject to vandalism and accidental breakage by animals or falling rocks. They cannot provide information about subsequent flows after they have been broken, and they must be reset each time a flow passes.

Debris flows have been observed with conventional seismographs that were deployed for earthquake detection. However, because conventional seismographs record earth tremor signals in the range of 0.5 to 20 Hz, they do not detect the strongest vibrations of debris flows and floods which occur at higher frequencies, and cannot pinpoint the exact location where a debris flow is flowing. Furthermore, conventional seismographs cannot distinguish debris flows from other sources of persistent noise at volcanoes, such as high wind, heavy rain, or fumarole activity.



A typical installation of an automated, debris-flow sensing station includes an antenna with a solar panel attached, a buried electronics vault, and a sensor buried 50 to 500 meters above a stream channel.

Researchers in Japan and China used high-frequency accelerometers in separate studies of debris flows. They reported that debris flows with dissimilar composition and dissimilar channel characteristics caused similar ground vibrations, in particular, the frequency, amplitude, and duration of vibration.

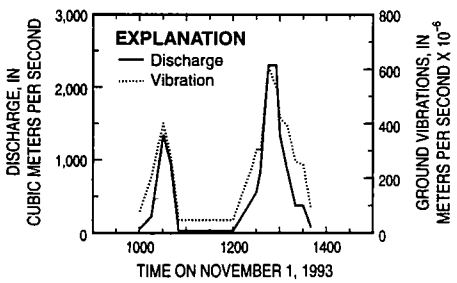
The Acoustic Flow Monitor

Scientists at the U.S. Geological Survey Cascades Volcano Observatory have developed an inexpensive, durable, portable, and quick-to-install system to detect and continuously monitor the arrival and passage of debris flows and floods in valleys draining active volcanoes. This automated system, known as the Acoustic Flow Monitor (AFM), senses and analyzes ground vibrations with a compact, solar-powered unit that is installed near specific channels. It uses an inexpensive geophone and an on-site microprocessor to continuously analyze vibration signals and detect debris flows and floods on the basis of frequency composition, amplitude, and duration of the vibration signal. A two-way radio system communicates between each sensing unit and a base station.

The geophone can sense ground vibrations with frequencies ranging from 10 Hz to 300 Hz. Most debris flows cause the ground to vibrate at a peak frequency of 30 Hz to 80 Hz, whereas watery floods cause ground vibrations with a peak frequency of more than 100 Hz.

The microprocessor-controlled instrument measures the amplitude of the vibration signal every second and sends data by radio to the base station at regular intervals, typically 30 minutes. If the instrument measures vibrations that exceed a certain threshold value (adjustable for each individual site) for longer than 40 seconds, the AFM transmits immediate alert messages. It continues to send alert data at 1-minute intervals for as long as the signal remains above the threshold level. When the signal drops below the threshold level, the AFM resumes normal operation, transmitting at less-frequent intervals. The AFM can detect increased ground vibrations several minutes (as much as a kilometer) before a debris flow or flood reaches the station, and continues to record vibrations above background level until the tail of a debris flow or flood has passed. The speed of approaching flows can be determined by detecting arrival times at sequential sensing sites along stream channels. In addition, continuous monitoring of the passage of floods and debris flows allows estimation of the volume of water and debris that is passing the sensing unit. At Pinatubo Volcano, Philippines, AFM signals have been used to estimate debris-flow discharges ranging from 10 to 1,800 cubic meters per second.

The AFM system has distinct advantages over previous debris-flow detection systems: (1) the geophone and microprocessor are set to analyze the peak vibrations typically produced by debris flows and floods, and screen out other noise or tremor that would affect normal seismographs; (2) flows are monitored as they approach, and recede from individual monitored sites, and (3) the equipment is ready to detect subsequent flows immediately without any maintenance.



Graph showing the relation between discharge and ground vibration during passage of two lahars on the Sacobia River near Pinatubo Volcano, Philippines on November 1, 1993.

This new technology is available and ready to deploy at the next sign of unrest at any volcano in the United States. The AFM system already has been installed and tested at Mount St. Helens and Mount Rainier, Washington; Redoubt Volcano, Alaska; Unzen Volcano, Japan; Cotopaxi Volcano, Ecuador; and Pinatubo Volcano, Philippines. The system can be used to alert emergency officials to the approach of debris flows and large floods emanating from volcanoes or other steep terrain. If properly calibrated to local conditions, it can also provide estimates of the magnitude of the flows. The AFM has the potential to save many lives from one of the most dangerous hazards posed to people who choose to live near these volcanoes.

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