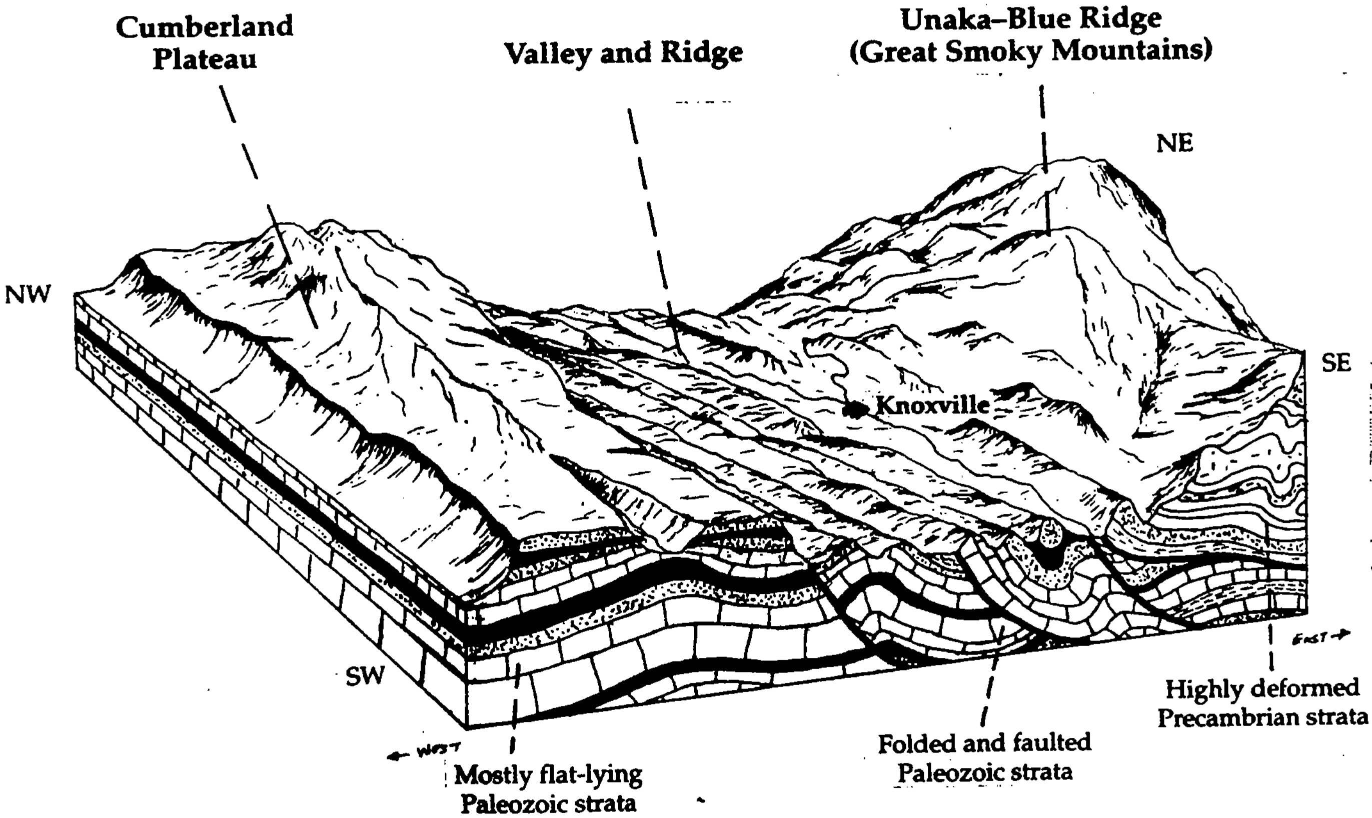
48th HIGHWAY GEOLOGY SYMPOSIUM

PROCEEDINGS
&
A FIELDTRIP EXCURSION GUIDE

Knoxville, Tennessee
May 8-10, 1997

Edited by Don W. Byerly



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STUDIES IN GEOLOGY 27

HIGHWAY GEOLOGY SYMPOSIUM

HISTORY ORGANIZATION AND FUNCTION

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then 47 consecutive annual meetings have been held in 31 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Annual Symposium moved to different locations as follows:

Year	HGS Location	Year	HGS Location
1965	Lexington, KY	1966	Ames, IA
1967	Lafayette, IN	1968	Morgantown, WV
1969	Urbana, IL	1970	Lawrence, KS
1971	Norman, OK	1972	Old Point Comfort, VA
1973	Sheridan, WY	1974	Raleigh, NC
1975	Coeur d'Alene, ID	1976	Orlando, FL
1977	Rapid City, SD	1978	Annapolis, MD
1979	Portland, OR	1980	Austin, TX
1981	Gatlinburg, TN	1982	Vail, CO
1983	Stone Mountain, GA	1984	San Jose, CA
1985	Clarksville, IN	1986	Helena, MT
1987	Pittsburgh, PA	1988	Park City, UT
1989	Montgomery, AL	1990	Albuquerque, NM
1991	Albany, NY	1992	Fayetteville, AR
1993	Tampa, FL	1994	Portland, OR
1995	Charleston, WV	1996	Cody, WY

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

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

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

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

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples, in Wyoming, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt with principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and Glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures in the greater Louisville area. The 1986 field trip was through the Rockies on recent interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

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

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W. A. "Bill" Wisner	-	1991
David Mitchell	-	1993
Harry Moore	-	1996

In 1969, the Symposium instituted an award to be presented to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

*deceased

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FUTURE
HIGHWAY GEOLOGY SYMPOSIUM
SCHEDULE

	TIME & LOCATION	COORDINATORS
1.	1997 Meeting: 48th May 8-10, 1997 Tennessee	Harry Moore Tennessee Department of Transportation
2.	1998 Meeting: 49th Arizona	Nicholas M. Priznar Arizona Dept. of Transportation
3.	1999 Meeting: 50th Virginia (Golden Anniversary)	Mr. William McKay Virginia Dept. of Transportation PH: 804/737-7731
4.	2000 Meeting: 51st Washington	Prof. Robert Holtz University of Washington and Steve Lowell Washington DOT
5.	2001 Meeting: 52nd Maine	Prof. Dana Humphrey University of Maine
6.	2002 Meeting: 53rd California	John Duffy CALTRANS
7.	2003 Meeting: 54th Massachusetts	Prof. William Highter University of Massachusetts
8.	2004 Meeting: 55th Minnesota	Chuck Howe MNDOT
9.	2005 Meeting: 56th Vermont	Alan McBean Vermont DOT
10.	2006 Meeting: 57th British Columbia*	Duncan Wylie Golder

*Meeting in Canada requires revision of By-Laws

TABLE OF CONTENTS

PROCEEDINGS: Edited by Eric Drumm and Matthew Mauldon*

Page

Karst & Environmental Factors

Internal Sulfate Attack: Wyoming's Expansive Experience
by G. Michael Hager and George S. Huntington 1

Investigation and Remediation of Service Plaza Fuel Storage Tanks in Karst by
Joseph A. Fischer and Joseph J. Fischer 14

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 by J. Brad Stephenson, W.F. Zhou, Barry F. Beck,
Tom S. Green, James L. Smoot and Anne M. Turpin 24

Stabilization of Karst Features on Major Engineering Projects by B. Dan Marks 35

Motorways in Karst of Slovenia by Dr. Stanka Sebela, Dr. Tadej Slabe, mag. Janja Kogovsek 49

Computer Analysis of Required Statistical Parameters for Sites with Multiple
Monitoring Wells Measured for Extended Periods of Time
by Terry R. West and Robert Pittenger 56

Foundations, Embankments and Pavements

Building Embankments of Coal Combustion Fly Ash-Bottom Ash Mixtures
by A. Karim, Rodrigo Salgado and C.W. "Bill" Lovell 66

Abandoned Underground Mine Inventory and Risk Assessment
by L. Rick Ruegsegger and Thomas E. Lefchik 75

Physical Distress Evaluation of the West Abutment of the SR 22 Bridge over
the Conemaugh River by Robert W. Bruhn, Bruce L. Roth, and Craig Chelednik 85

Physical and Chemical Evaluation of Soil-Tire Mixtures for Highway Applications
by Abdul Shakoor and Chein-Jen Chu 96

Case Study-Drilled Shaft Foundations-Lake of the Ozarks Community Bridge
by John F. Szturo and Wayne Duryee 106

Resistivity Field Testing of Reinforced Earth Structures
by Grant Adkins and Nanette Rutkowski 117

Potential Effects of Superpave Implementation on the Arkansas Aggregate Industry
by Kevin D. Hall 127

Soil Slope Stability

The Landslide at Waverly Street and Maryland Route 36: A Case History
by A. David Martin 136

Analysis and Remediation of the Miles Road Landslide Complex,
Cuyahoga County, Ohio by Abdul Shakoor and Mark A. Kroenke 146

A Case History on the Use of Bioengineering in Mechanically Stabilized Earth Slope
Design by Mark H. Wayne and Sean Wokasien 157

Rock Slope Stability

State of the Art for Rock Cut Slopes in Eastern Kentucky by E.M. Wright 167

Rock Slope Stability Analysis Based on Potential Energy
by Scott Arwood, Matthew Mauldon, and Harry Moore 174

The Role of a Contractor's Blasting Consultant-A Case Study by Harry L. Siebert 184

Elevated Catchment Areas: A Performance Report by Richard H. Cross 193

Selecting Shear Strength Parameters for Weathered Metamorphic Rock
by Philip C. Lambe and Margaret Sweitzer 197

Parameters Used for Design of I-26 in North Carolina by Jerome Beard 207

Fractal Geometry for the Quantification of Rock Joint Roughness
by Matt Haston, Matthew Mauldon and Don W. Byerly 217

***All proceedings citatations should acknowledge these editors.**

FIELDTRIP EXCURSION GUIDE 227

OVERVIEW OF THE GEOLOGY by Don Byerly 227

General 227
Valley and Ridge 228

BLUE RIDGE GEOLOGY by Mark Carter 229

Introduction 229
Western Blue Ridge Stratigraphy 229
Western Blue Ridge Framework and Orogenesis 231
Concluding Remarks 233

ROAD LOG by various contributors 234

REFERENCES 249

CONSTRUCTION TECHNIQUES ON U.S. 23 (FUTURE I-26), UNICOI COUNTY, TENNESSEE by Harry Moore 253

APPENDIX A - Cumulative Index of HGS Proceedings (A-1) 274

APPENDIX B - HGS Proceedings Availability List (B-1) 299

PROCEEDINGS

OF

48th

HIGHWAY GEOLOGY SYMPOSIUM

Edited By

Eric Drumm & Matthew Mauldon

Institute for Geotechnology
Department
of
Civil and Environmental Engineering

The University of Tennessee
Knoxville, Tennessee

"INTERNAL SULFATE ATTACK"

Wyoming's Expansive Experience

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Materials Research Engineer, WYDOT*

ABSTRACT

Wyoming's highways cross many geologic units that are composed of expansive soils or shales. In the last few years, two major highway projects have been disrupted by a little known process, internal sulfate attack. Research into the causes of the highway break-ups has added a new twist to expansive soils mitigation techniques, such as lime treatment, cement treatment and membrane encapsulation.

U.S. Highway 85, between Mule Creek Junction and Newcastle, Wyoming, is currently being reconstructed; and the last of the five construction projects is scheduled to be completed in 1997. The surfacing section on the first and second projects included an impermeable membrane, cement treated base, and asphaltic concrete. Soon after completion, heaves in the pavement started occurring at erratic intervals about every 500 feet. An investigation revealed that the cement treated base was expanding instead of its normal shrinkage cracking. An analysis of the base led to the conclusion that an adverse chemical reaction had occurred and internal sulfate attack caused a roughly two percent expansion of the base. A research project by the University of Wyoming and WYDOT was funded to find the exact cause of the reaction.

Another project near Rawlins, Wyoming, involved lime treating the soils before placing the surfacing section. Soon after completion of the project, a 100 foot section of the EBL cracked and heaved. Also, small boils of heaved soils appeared along the edge of the pavement. An investigation of the cause revealed that internal sulfate attack was causing expansion of the lime treated soils. Samples of the soils were tested in the lab and the swelling characteristics were duplicated.

Wyoming has used lime treated soils and cement treated bases for years with some success. Failures that have occurred in the past may have been misdiagnosed and were actually due to the formation of ettringite and other calcium sulfoaluminate minerals. From these two incidents, it is clear that there are certain soils types high in sulfates and clays that, with the addition of water and cement, undergo a chemical reaction which leads to the formation of expansive minerals. The Wyoming DOT is now checking the soil and water sources for high sulfate and clay contents before using these materials in cement treated bases and in lime treated soils.

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INTRODUCTION

The Wyoming Department of Transportation (WYDOT) has extensive experience with cement-treated base (CTB) and more limited experience with lime-stabilized soil.

"Internal sulfate attack" is the process where soil-cements, such as CTB or lime-stabilized soil, undergo damaging expansion and internal deterioration caused by sulfates in the stabilized soil or treated aggregate.

Sulfate attack in Portland cement concrete (PCC) is extensively documented. Soil-cements generally have more fines, less cement, and less water than ordinary PCC. These material differences make extrapolation from ordinary PCC to soil-cements difficult and unreliable.

This paper presents two field investigations of internal sulfate attack in Wyoming and the results of a laboratory study carried out by the University of Wyoming. Quantitative laboratory measurements are combined with qualitative and approximate quantitative field observations in an attempt to predict destructive expansion levels.

BACKGROUND

Cement-treated Base

Wyoming CTBs are rigid materials with unconfined compressive strengths of about 5 MPa (700 psi). Typically, they are constructed in areas where long haul distances make the use of high quality aggregate expensive. Generally CTB has worked well in Wyoming. However, shrinkage cracking similar to that in ordinary concrete often occurs. Also, CTBs sometimes developed swells. For a variety of reasons, including sulfate attack, WYDOT has suspended construction of cement-treated bases.

Lime-stabilized Soil

Lime-stabilized soil is used to lower the plasticity of soft, clayey subgrades. The exchange of one Ca^{2+} ion replacing two Na^{+} ions thins the diffuse double layer, thereby reducing water retention and plasticity. This makes both a more stable construction platform and a stronger subgrade.

Internal Sulfate Attack

All types of sulfate attack are believed to be caused by reactions of the expansive mineral, ettringite ($Ca_6Al_2(SO_4)_3(OH)_{12} \cdot 26H_2O$), and other similar calcium sulfoaluminate minerals. When ettringite forms from calcium ions, aluminate ions, sulfate ions, hydroxyl ions, and water, it develops very high expansive forces, possibly expanding the bulk material.

For internal sulfate attack to damage a material, two conditions must exist. First, the chemical environment must allow formation of ettringite or a similar expansive mineral. Second, expansion of this mineral must deform or weaken the bulk material in a deleterious way. If a moderate amount of ettringite forms in a porous material, the ettringite may merely fill void space without damaging the bulk material. Internal sulfate attack is controlled by complex and poorly understood physical and chemical processes. This makes prediction of when and where internal sulfate attack will occur a difficult and imprecise science.

There is a fundamental, unanswered question about internal sulfate attack in soil-cement: Is it a sulfate-cement reaction, a sulfate-clay-cement reaction, or a combination of the two? Sherwood (1962) believed that sulfate expansion in soil-cement is fundamentally different from sulfate attack in ordinary PCC. He stated that the main reason for disintegration of soil-cement by sulfate ions is the clay-sulfate reaction, which is more rapid than the sulfate-cement reaction that brings about sulfate attack in ordinary PCC. Cordon (1962) argued that sulfate attack takes place similarly in soil-cement and in ordinary PCC. Though opinions have been expressed on both sides of the argument, there does not appear to be a strong consensus on this issue.

Constructed CTB vs. Laboratory Bars

Ultimately, the goal of a test procedure is to prevent problems in real situations. The laboratory study sought a means of predicting damage to roads built with CTB. Laboratory expansion of bars cured in lime-saturated water is used to predict heave of a roadway.

One might assume that the processes controlling expansion are not significantly altered by laboratory procedures. This assumption is not necessarily valid for CTB. Kinetic controls may be critical to the expansion process. Differences in bar sizes and aggregate gradations may change the process substantially. Accelerated laboratory tests or tests using different aggregate gradations may no longer accurately simulate field conditions (Ksaibati and Huntington, 1996).

The crucial question is: How much length change of a laboratory bar reflects failure in a real, confined CTB? Some swelling of CTB is acceptable, though not desirable. Length changes in laboratory concrete mortar and cement paste specimens ranging from 0.05% to 0.4% after 28 days may be considered failed (ASTM, 1996 and Cohen and Mather, 1991).

Sulfate Extraction Methods

Two basic approaches may be taken when evaluating sulfate content. The sulfate may be extracted in an aggressive manner that measures virtually all the sulfate in the aggregate. Alternatively, a less aggressive extraction procedure simulating natural waters may be used. The more aggressive approach is most valid when one is trying to determine if there is any possibility of internal sulfate attack; it is less likely to give a false negative test result. The less aggressive approach is more likely to indicate whether pore water conditions are actually conducive to sulfate attack; it is less likely to give a false positive test result. Since the primary purpose of this study is to establish methods of preventing failures, a more aggressive extraction procedure was used.

U.S. 85, MULE CREEK JCT., WYOMING: CEMENT-TREATED BASE

In the summer of 1991, a section of U.S. 85 just north of Mule Creek Junction, Wyoming, was reconstructed. Ten inches of cement-treated base (CTB) were placed on top of a clayey, mostly A-7-6 subgrade. A geomembrane was placed between the subgrade and the CTB to prevent subgrade swelling. Four inches of plant mixed bituminous pavement (PMP) were placed on top of the CTB (see Figure 1). Shortly after placement of the PMP, swells resembling frost heaves appeared. These heaves were initially blamed on poor construction practices.

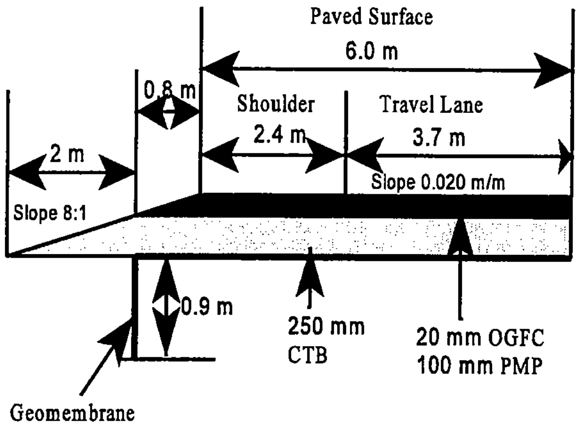

Figure 1. Typical Cross-section of U.S. 85.

The section immediately to the north was constructed during the following summer using the same pavement structure and material sources. Because of the swells observed on the adjacent section, extreme care was taken to prevent any construction flaws. This section developed similar but more severe heaves (see Photo 1). They appeared at erratic intervals, varying from about once a kilometer (half mile) to every 50 meters (150 feet) (see Photo 2). WYDOT milled the heaves repeatedly. When the milling removed all four inches of PMP, the CTB and geomembrane were removed and unbound crushed base was put in its place. In some places outside the shoulder, CTB shoved up as much as a foot. WYDOT and the University of Wyoming set up a research project to determine the cause of the heaves and to develop a method of preventing such failures on future projects.

Photo 1. Damaged section of U.S. 85 south of Newcastle, WY.

Table 1. Unconfined Compressive Strengths on US 85.			
Age, days	Date Placed	Cement %	UCS, MPa
39	6/4/92	6.9	5.10
39	6/4/92	7.0	3.02
35	6/8/92	6.9	4.72
35	6/8/92	6.9	4.55
34	6/9/92	7.1	5.38
34	6/9/92	6.8	4.77
33	6/10/92	6.9	4.33
33	6/10/92	7.1	4.28

Table 1 contains the unconfined compressive strengths of the CTB determined during June 1992. The strengths are fairly consistent, except for one of the cylinders tested on 6/4/92. The CTB with an UCS of 3.02 MPa is over two standard deviations from the mean. All others are within one and a half standard deviations of the mean. This indicates that the CTB may have occasional areas of weakness. This agrees with the field observation that the heaves are at erratic intervals, indicating that the material was intermittently flawed.

In the summer of 1994, after the swelling and heaves had damaged the road so severely that it needed reconstruction, WYDOT decided to rubblize the CTB. An impact pavement breaker reduced the CTB to fist-size and smaller chunks. This may have damaged the geomembrane. In soft spots, the CTB was removed and crushed base was put in its place. The geomembrane was not replaced. Four inches of PMP were placed on top of the rubblized CTB.

By the summer of 1996, swells had reappeared. It is not known whether these new swells are due to continuing expansion of the CTB or to subgrade swelling caused by removal of and damage to the geomembrane. The new swells, though detrimental to ride quality, are not nearly as severe as those that appeared after the original construction.

Laboratory Study

In the fall of 1993, the University of Wyoming, under the supervision of WYDOT's Materials Program, began a study into the cause of the heaves on U.S. 85. More detailed descriptions of the methods used in this study are described in Ksaibati and Huntington (1996). More detailed results are available in Huntington (1995) and Huntington and Ksaibati (1995).

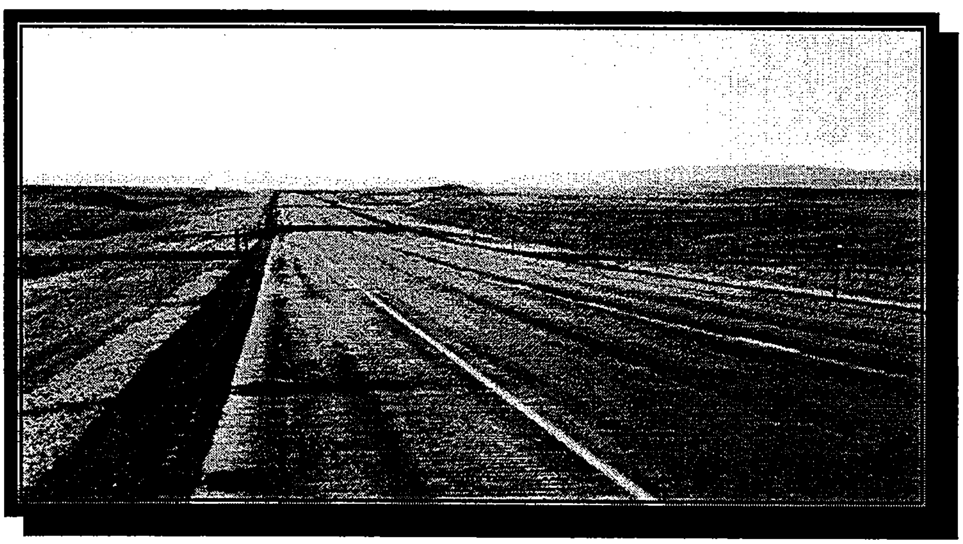

Photo 2. Sections of U.S. 85 milled to remove heaves.

Damage to the road was examined. A very rough quantitative estimate of field expansion was based on the tilting of delineators brought about by transverse expansion of the CTB. Using this approximate method, field expansion was estimated to be least 0.7%.

Materials

Samples of stockpiled aggregate used in the CTB were collected. The aggregate, referred to hereafter as LAK Pit aggregate, was from a Quaternary alluvium eroded from the Black Hills, roughly thirty miles to the northeast (see Photo 3). The Jurassic Gypsum Spring formation is a likely source of sulfates in this alluvium.

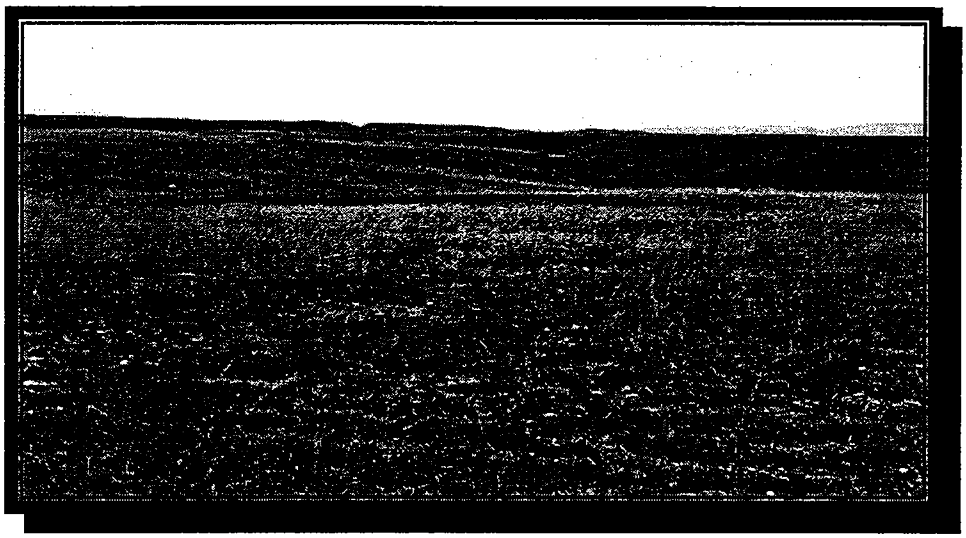

Photo 3. Site of LAK Pit with Black Hills in the background.

Samples of the mixing water from the South Fork of the Cheyenne River were collected.

The sulfate content of the LAK Pit aggregate was determined according to ASTM C 471, Section 13, *Test Methods for Chemical Analysis of Gypsum and Gypsum Products*. This aggressive method extracts the sulfate in 1:5 $HCl_{(aq)}$ (sp gr = 1.19) at near-boiling temperatures for one hour. Barium chloride ($BaCl_2$) precipitates the sulfate as barium sulfate ($BaSO_4$). The precipitate is filtered and weighed, and the sulfur trioxide (SO_3) content is calculated.

The LAK Pit aggregate used in the laboratory mixtures had a sulfate as SO_3 content of 2.7% and a PI of 10 or 11. Construction samples had PIs ranging from non-plastic to 12, typically 5 or 6. The collected Cheyenne River water had a sulfate as SO_3 content of 0.15%. USGS data from the early 1980's indicates that this sulfate content is reasonably close to that typically found in this stretch of the Cheyenne River. Holnam Type I-II Low Alkali cement containing 6.9% C_3A and 11% C_4F was used in all laboratory prepared CTB specimens.

Commercially obtained clays were mixed with some of the CTB mixtures; there were two kaolins, two ball clays (ball clays are roughly 85% kaolinite and 15% illite), and one bentonite.

Gypsum was obtained from the Mountain Cement Plant, Laramie, Wyoming. It was found to be 95% pure according to the $BaSO_4$ precipitation test.

Two additional aggregates were used. They were a quarried limestone and a quarried clinker (locally referred to as "scoria"), a lightweight, heated-expanded, red shale often used in Wyoming CTB. Both of these were mixed with 30% river run filler. Both the clinker and limestone mixes with filler were non-plastic.

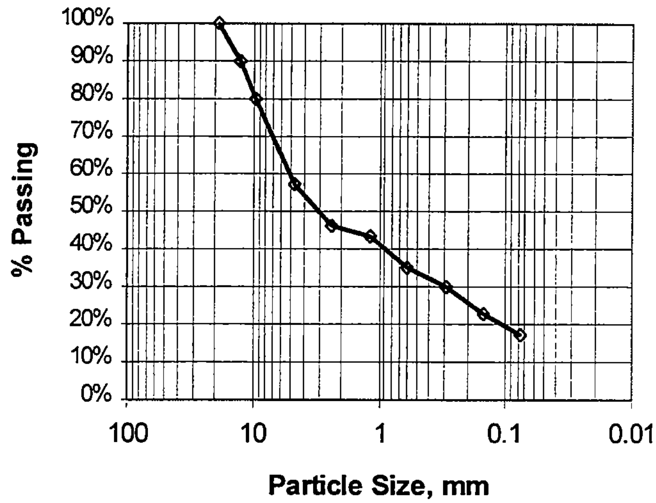

Figure 2. Laboratory Aggregate Gradation

Methods

A total of 78 bars and 78 cylinders simulating CTB were prepared. The bars' lengths were measured periodically for two years. The cylinders' compressive strengths were determined at 28 days.

The specimens were mixed at the gradations shown in Figure 2. Each had 17.5% passing a #200 (75 μm) sieve, 8% Portland cement, and water added to between one and two percent below optimum moisture content as determined by ASTM D 698 (Standard Proctor). When clay and/or gypsum were added to the mix, a corresponding amount of the aggregate fines were removed, maintaining the same percentage passing the #200 sieve for all mixes. The cylinders were compacted according to ASTM D 559. The bars were compacted according to the method described in Ksaibati and Huntington (1996). This method consists of dropping a Marshall hammer the full length of the shaft onto the CTB mix. Mixes are compacted in 3" x 3" x 11¼" molds with gage studs in each end. Three lifts of twenty blows each are applied to the mix. The bars are stored for 24 hours at 100% relative humidity at 73 °F. Then they are removed from the molds, measured, and cured in lime-saturated water at 73 °F until the next measurement.

The cylinders were tested for unconfined compressive strength according to ASTM D 1633. The bars were measured with a length comparator adhering to ASTM C 490.

Results and Analysis

The initial matrix evaluated the effect of aggregate type and mixing water concentration on expansion and strength. LAK Pit and limestone aggregates were mixed at 100% LAK Pit, at 100% limestone, and at 20% increments of each aggregate. Mixing water at natural strength and evaporated to twice its original concentration was mixed with otherwise identical aggregate and cement mixtures. Statistical analysis indicated that mixing water was not a significant contributor to expansion or weakening. The degree of expansion and weakening was controlled only by the concentration of LAK Pit aggregate.

The primary matrix consisted of limestone and clinker CTB mixtures spiked with from 0% to 5% clay and gypsum. All were mixed with Cheyenne River water at its natural concentration, 0.15% sulfate as SO_3. Statistical analysis established that plasticity index, sulfate content, and aggregate type were all significant factors in expansion.

Figure 3 shows the linear expansion of CTB bars with various amounts of gypsum. Limestone bars with no gypsum and 1% gypsum did not expand significantly. The limestone bar with 2% gypsum expanded about 0.4% in two years. Limestone bars with 3% or more gypsum expanded during the first eight months, then disintegrated between eight and twelve months. The bar with clinker and 8% gypsum expanded 0.13% during the first four months, then its expansion stopped. It expanded at least twice as much during the first two months as any of the bars made with limestone. By two years, it had expanded only half as much as the bar made with limestone and 2% gypsum. Both aggregate type and sulfate in the form of gypsum affect expansion in the absence of substantial amounts of clay.

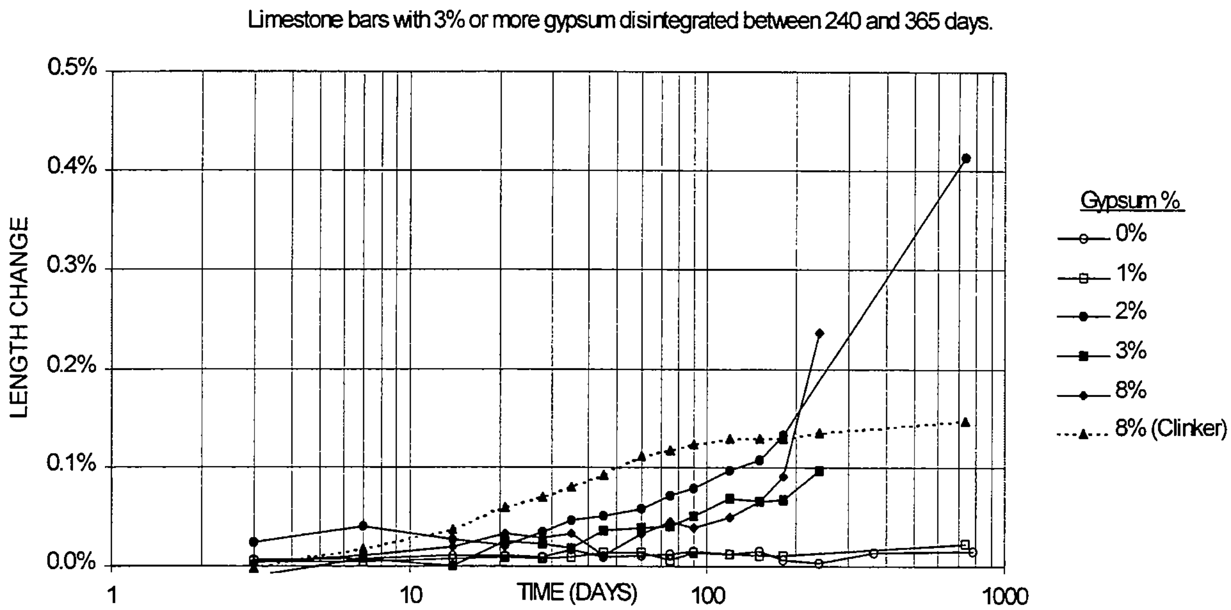

Figure 3. Limestone (except as noted) bars with gypsum.

The most perplexing outcome from this study is the striking difference in both the timing and magnitude of expansion with the different clay and aggregate types. The clinker expanded quickly during the first few months, then it stopped expanding. The limestone expanded slowly during the first few months, but this expansion continued much longer, often until the bar disintegrated completely.

Figure 4 typifies the expansion rates and magnitudes observed for the three aggregate types throughout this experiment. The most significant observation is that using 28 or 60 day expansions is not highly correlated with long-term expansion.

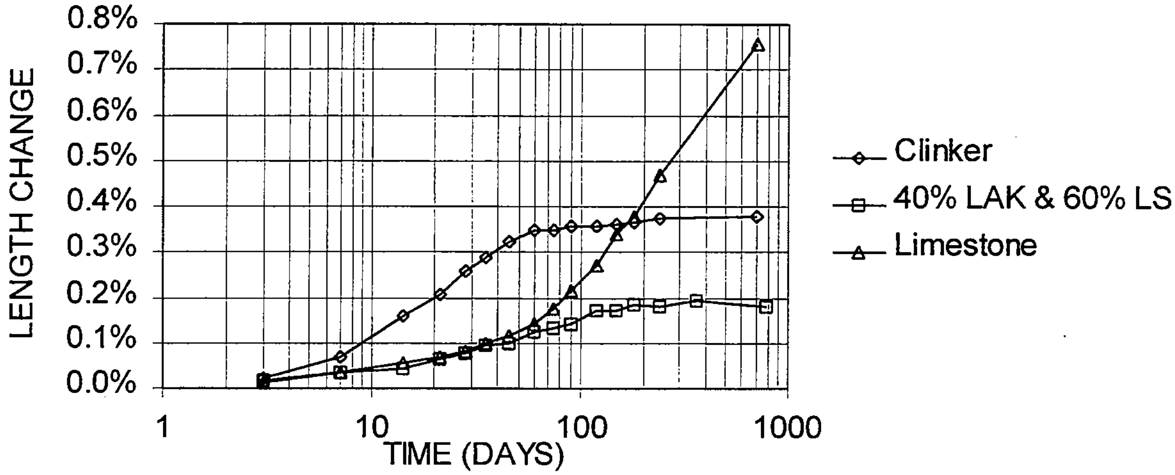

Figure 4. Bars with 1.1% sulfate.

Different clays also affected expansion differently. Figure 5 plots the expansion of limestone bars spiked with 2% gypsum and 6% clay. Expansion is similar, except for the bentonite. It expanded quickly initially, then stopped. Bars with the other clays continued expanding rapidly for at least a year, but the bentonite stopped expanding by eight months.

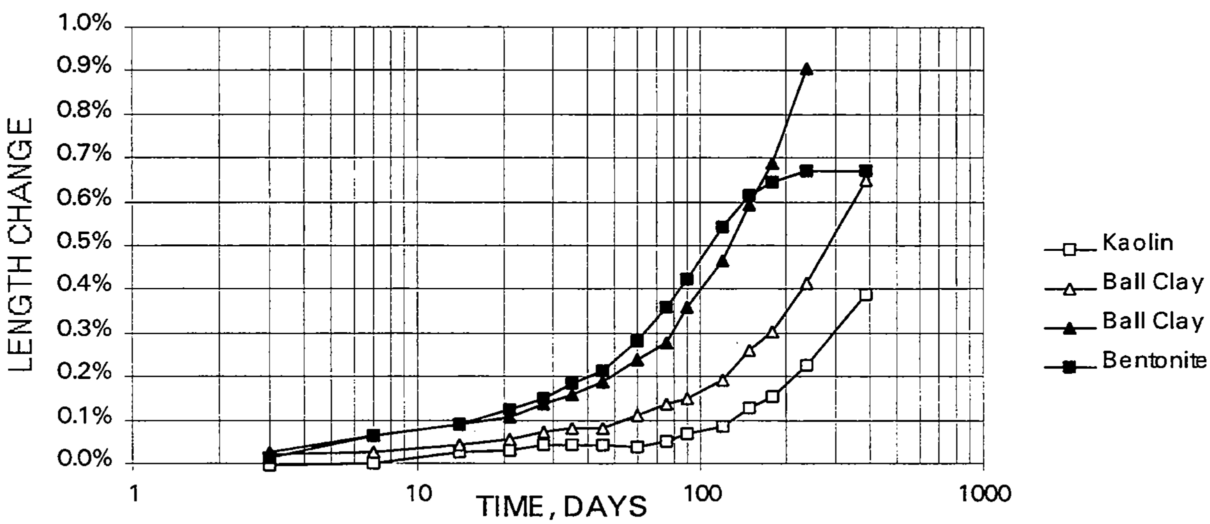

Figure 5. Limestone bars with 2% gypsum and 6% clay.

I-80 RAWLINS MARGINAL, WYOMING: LIME-STABILIZED SOIL

The Rawlins Marginal project is 3.82 miles long. Reconstruction of the westbound lane (WBL) and the eastbound lane (EBL) with 11 inches of concrete pavement were let as separate projects in 1990 and 1991. The WBL was let first, and the design for the surfacing section included 2 feet of subexcavation and separation geotextile layers. Because of the high cost for subexcavation and geotextile, the design of the EBL surfacing section was changed for the second contract to 8 inches of lime treated subgrade to raise the "R" values of the subgrade. The lime treatment consisted of a 5% lime slurry applied to the subgrade, then processed, shaped and compacted in 4 inch lifts.

By April 1994, the EBL started showing evidence of heaves and small boils in and along the concrete surfacing. The largest heave is at milepost (MP) 214 and is about 100 feet long with large diagonal cracks in the concrete, and it affects only the EBL. Two boils along the EBL ramp shoulder at the Cedar Street exit are 3 feet in diameter. The boils are 3 inches high and have created radial cracks in the asphalt shoulder and lifted the nearby concrete pavement slightly.

Geotechnical Investigation

A geotechnical drilling investigation was performed at each site in order to determine the cause of the heaves. The results of the drilling investigation revealed that the surfacing section was constructed as planned and the soils under the lime treatment were slightly clayey sand with numerous gypsum crystals. The heaved soils were confined to the lime treated zone, which appeared to have a crystalline-like structure. The subgrade soils were actually very weathered bedrock. The heave at MP 214 was aligned with the strike of the local geologic units.

The resident engineer for these projects noted two construction related problems. One was that two water sources were used to mix up the lime slurry. The Thayer Well was set up as the water source and was changed to the City of Rawlins water supply after the Thayer water caused major mixing problems in the lime slurry. The other construction problem was related to the placing of the lime slurry. There were some areas where ponding of the slurry occurred and caused the lime content to be at least twice the planned 5% by weight rate.

Laboratory Study

Bulk samples of the subgrade and lime treated soils were obtained, as were water samples from the Thayer Well and the Rawlins City water system. Sulfate contents of the soils were measured, and a series of swell tests were run at different lime concentrations using the two water sources. Samples collected of the natural ground, crushed base, and lime-treated base were submitted to the Central Laboratory for a sulfate analysis. A wide range in sulfate percentages occurred, with water soluble sulfates varying from less than 1% to 2.2% and acid soluble sulfates varying from less than 1% to 13.44%. Table 2 contains sample locations, sulfate analyses, and classification results.

Table 2. Rawlins Marginal Soil Samples.

Sample No.	TH No.	Depth (m)	MC (%)	LL	PI	#200 (%)	Soil Type	Water Soluble Sulfates	Acid Soluble Sulfates	Remarks
1	1	.46-1.2	17.4	0	0	31.9	A-2-4(0)	2.2 %	13.44%	Natural Ground
6	5	.3-.75	10.9	0	0	11.3	A-1-b(0)	0.57%	0.72%	Crushed Base & Lime Treated Base
7	5	.9-1.35	15.7	32	12	31.3	A-2-6(0)	2.23%	3.13%	Natural Ground

The natural subgrade soils were mixed with a 5% and a 10% lime slurry and used the water from the Thayer Well and the Rawlins City water system. The soils were thoroughly mixed and compacted to AASHTO T-99 standards. A control sample was also prepared with no lime added, and using distilled water. The compacted samples were prepared and placed in a consolidometer frame and allowed to expand with a slight confining pressure (0.125 Tons) equal to the natural overburden pressure. The test results are listed in Table 3.

Table 3. Rawlins Marginal Swell Tests.

Sample No.	Water Used In Testing	Percent By Weight of Lime	Swell (%)	Elapsed Time (Days)
SW94266	Distilled	0	0.20	1.3
SW94266A	Thayer Well	5%	0.55	20
SW94266B	City	5%	1.10	110
SW94266C	Thayer Well	10%	0.39	22
SW94266D	City	10%	3.78	122

CONCLUSIONS

<u>U.S. 85</u>

Heaving on U.S. 85 was caused by an expansive sulfate reaction between the aggregate and Portland cement. Intermittently deleterious aggregate was present in the LAK Pit aggregate. Both clay and sulfate contribute to expansion and weakening of soil-cements; portions of the LAK Pit aggregate high in clay and sulfate reacted with Portland cement and water to cause damaging expansion of the cement-treated base.

<u>Laboratory Study</u>

For a CTB with 17.5% passing a #200 (75µm) sieve and 8% Portland cement, damaging expansion becomes likely with over about 1% sulfate as SO_3 in the aggregate.

Based on the rather unique conditions of this study, a best estimate of the deleterious sulfate level is about 1.0% sulfate as SO_3. This estimate has very little margin for error. Sampling error or unusual conditions not addressed in this study might allow damaging reactions to occur at this sulfate concentration.

Twenty-eight or 60 day expansions represent a highly variable proportion of the total expansion a soil-cement or stabilized soil will undergo.

Rawlins Marginal

Summarizing Table 3, swell of the 5% mixture was two times greater using City water, while swell of the 10% mixture was ten times greater using City water. The time for both the 5% and 10% mixtures to stabilize using City water was approximately five times longer than mixtures using Thayer Well water. The Thayer Well water is very hard and is high in dissolved solids. The laboratory tests duplicated the field problems by using this well water to mix the lime slurry. The Thayer Well water tends to kill any chemical reactions and it reduced the effectiveness of lime treating the subgrade. This was noticed during construction, and the water source was changed to the Rawlins City water. This change also had a negative effect in increasing the swell potential of soils with high sulfate contents. The exact chemical product of the expansion was not identified (possibly ettringite), however all the components necessary for expansion are present in the clay, sulfate, and lime.

RECOMMENDATIONS

When considering a soil or aggregate for cement or lime treatment, determine its sulfate content using an aggressive extraction procedure, such as the one used in this study. This determines if there is sufficient sulfate to damage the soil-cement or stabilized material. The issue of whether the reaction will actually occur is not directly addressed by this approach, but the possibility of internal sulfate attack is established. Less than 0.25% sulfate as SO_3 appears to be a safe level. Even if internal sulfate attack occurs, it won't be of a damaging magnitude. It is better to have many false positives than one false negative test result. Savings from using soil-cement are far less than the damage caused by a failure. There are alternative technologies that may be used instead of chemical stabilization. If sulfate expansion is even a remote possibility, an alternative technique should probably be chosen. It is better to err on the side of conservatism; the cost of failure is too high.

Sulfate content of any soil or aggregate under consideration for cement- or lime-treatment should be determined using Section 13 of ASTM C 471. Samples to be tested for sulfate should be collected from the lowest energy depositional deposits within the material being considered for chemical stabilization. Low energy deposits are likely to concentrate both clay size particles and precipitated sulfate. Depositional concentration of these two components may lead to extremely damaging, expansive sulfate reactions when the soil or aggregate is combined with water and Portland cement or lime.

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INVESTIGATION AND REMEDIATION OF SERVICE PLAZA FUEL STORAGE TANKS IN KARST

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Abstract

Sinkhole repairs were accomplished at the site of three fuel storage tanks at a Pennsylvania Turnpike service plaza in the spring of 1996. The first sinkhole occurrence was observed in 1993 and attempts were made to fill the sinkholes with gravel backfill. As sinkhole occurrence continued, the service plaza's refueling operations ceased and the tanks were emptied to prevent a possible environmental catastrophe in February 1995. Subsequent investigations using "BAT" (best available technology) were conducted in an attempt to find a more logistically satisfactory location, and later to better define the subsurface below the exiting tank site.

Tilted, fractured and solutioned Ledger Formation dolomite underlies the sites investigated. An exploratory drilling/grouting program using a conventional sand, cement, water, and bentonite grout with the selective use of an accelerator apparently sealed the subsurface voids below the tanks. As predicted, more sinkholes formed outside the repaired tank area. Eventually, the likely source of surface water accelerating sinkhole formation was identified and remediated and sinkhole activity in the locale of interest seems to have ceased. The tank farm area has functioned in a completely satisfactory manner for about one year and motorists can once again refuel vehicles as well as their appetites.

Introduction

The Pennsylvania Turnpike crosses and then parallels Route 202 through a highly developed section of Montgomery County, Pennsylvania, west of Philadelphia. Subsequent to the construction of a fuel storage area at a busy Turnpike service plaza, near its convergence with Route 202, environmental studies were performed as a result of leaking tank and supply line concerns. It is assumed that the field investigations were performed under the technical direction of geotechnical personnel. The geologic information in these studies indicated that the fuel storage area was underlain by siltstone, with areas of dolomite nearby. No mention of solutioning or fracturing within any of these rocks was noted on the boring logs.

In the fall of 1993, the first sinkhole appeared. Subsequently, through 1995, additional sinkholes opened and were filled with gravel. One of these sinkholes was located immediately adjacent to three fuel storage tanks. The plaza operator hired a local consultant to investigate the area and remediate this sinkhole. The results of this adventure were; 1) the subcontracted drillers apparently know more about karst than the consultant, and 2) the sinkhole shortly reappeared, together with another, both near the tank pad.

At the request of the plaza operator, the senior author visited the site. Two sinkholes were evident, one extending below the tank cover slab and one within several feet of another edge of the slab, both some 6 to 8 feet in diameter. Apparently, the first sinkhole had grown in size over the previous two days. Solutioned limestone was reported on the boring log prepared by the driller from the aforementioned sinkhole investigation/remediation work.

It was decided to immediately empty the fuel tanks and close the service plaza's fuel operations before a tank rupture occurred with the likely consequence of an environmental catastrophe resulting from the solutioned nature of the underlying dolomite and the populated nature of the locale. Remedial grouting was planned to start almost immediately.

Some seven months later, a geotechnical investigation was initiated with the intention of moving the tanks to a new location within the service plaza. The new area appeared to have even more potential problems. Thus, installation of new tanks, which had been scheduled for the week following the start of the investigation, was canceled. Subsequently, a geotechnical investigation of the original tank area indicated that remediation was both possible and economically feasible. An exploration/grouting program was initiated approximately 13 months after the tanks were emptied.

It was recognized that additional sinkholes could form outside the tank area as a result of the remedial work, which changed subsurface water flows without removing the unknown source of (sinkhole-triggering) water. New sinkholes started to form outside the tank area within weeks. Two separate repairs were made. At the time of the second repair, a significant leak in a buried water line crossing west of the tank area was found and repaired. Some 12 months have passed without sinkholes forming within the tank area, so there is hope that the initial remediation work was effective.

Geologic Conditions

The site area lies in a valley that extends from the vicinity of Lancaster, PA to the eastern edge of Montgomery County, just to the west of Philadelphia, PA. The valley itself is an inlier of folded and faulted Paleozoic-age sedimentary rocks in the Piedmont Province of Pennsylvania. The valley is underlain by Cambrian and Ordovician limestone and dolomite, and by Cambrian quartzite and quartz schist of the Lebanon Valley sequence (Berg, et al, 1908).

The susceptibility to chemical erosion of the carbonate rocks is responsible for the subdued topography of the valley. The hills to the north of the site locale are cored by Precambrian crystalline rocks (at depth) and by quartzite and schist. The service plaza is on a fault-bounded block of Ledger Formation dolomite.

This formation is described by the Pennsylvania Geological Survey as: "moderately well-bedded; massive, light gray, locally mottled, coarsely crystalline dolomite, cherty, with siliceous beds. Joints have a blocky pattern and are moderately well developed; moderately abundant; irregularly spaced, having a wide distance between fractures; open and steeply dipping."

Geotechnical Studies

Some seven months after closing the service plaza, after political and financial negotiations between the plaza's owners and service providers, a geotechnical study was performed in a new fuel storage area adjacent to the fuel delivery area. This study included the examination of aerial photographs, the available geologic information (from the State and from previous studies performed at the site), a geologic reconnaissance of the site and locale, and the drilling of three rotary-wash test borings using techniques particularly established for carbonate rock sites (see Fischer and Canace, 1989 for a description of the drilling and sampling techniques used). The most important aspects of these investigative procedures are the need for drillers experienced in karst, the monitoring of drilling fluid losses and the use of split-double tube coring devices.

This study revealed that in this area the bedrock had been subject to fracturing and thus increased solutioning in the past. Numerous open channels and soil-filled cavities were noted during the drilling and coring operations. The log of Borings 2 (see Figure 1) and 3 evidenced more cavities than sound rock. Rock depths varied from 35 to 65 feet below grade in a ±50 foot square area. On the basis of the recovered core, a small fault was believed to trend through the westerly portion of the proposed tank site.

The grouting of the boreholes for purposes of both sealing the boring to prevent future sinkhole occurrence and as a diagnostic tool for cavity size also revealed the solutioned nature of the subsurface. In excess of one cubic yard of grout was introduced into the boreholes with no evidence of the grout column at the surface. Other means were then used to seal the borings from surface water infiltration.

As this area would require extensive remediation for tank support, an investigation incorporating two additional borings was performed at the location of the original tanks (see Figure 2 for A log from this location). Again, solutioned dolomite was found. The results of the second investigation indicated essentially similar subsurface conditions to that in the initial area studied with the exception that rock was much shallower (21 to 26 feet below grade) in the original tank area. During this study, a constant flow of water into a nearby storm sewer was noted. The source of this water was not obvious as it flowed near the base of a steep man-made slope adjacent to the site. Reconnaissance of the site locale during a heavy rain storm did not indicate any obvious relationship between the surficial drainage of storm water and the relatively heavy flows observed in the storm sewer.

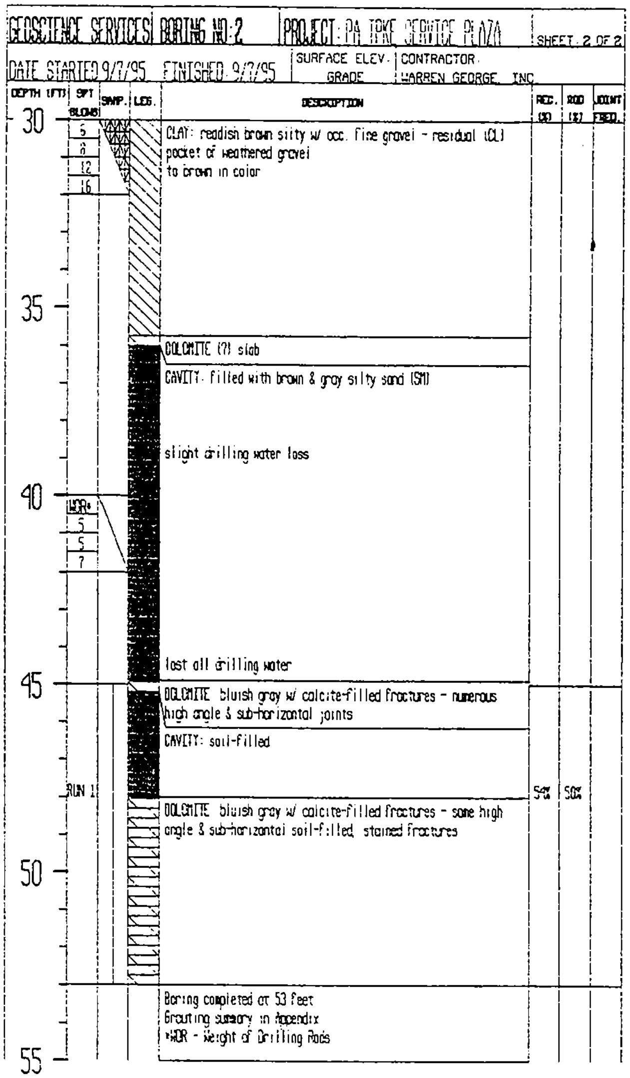

Figure 1 -- Boring Log

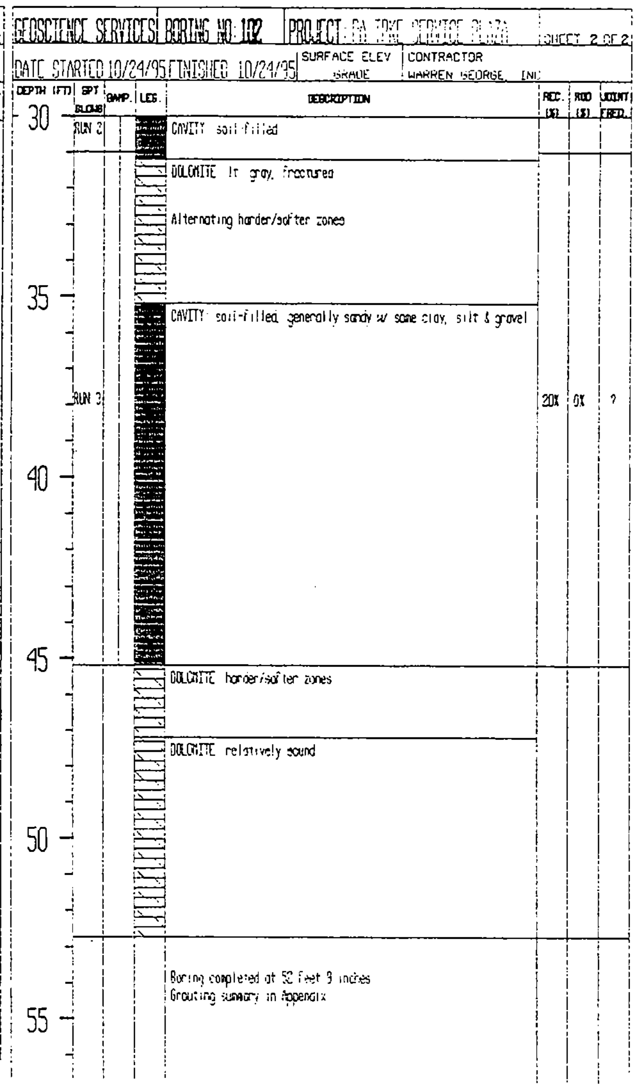

Figure 2 - Boring Log

Foundation Alternates

As the two studies revealed conditions hazardous for the support of the fuel tanks by conventional means, four foundation alternates were discussed. One alternate was to support the tanks on a thick, reinforced concrete slab. This would be the simplest approach, but also the most risky. It is probable that the tanks would settle in the future as a result of the continuing migration of overburden soils into rock openings. Monitoring would be required to reduce the possibility of catastrophic failure. However, it would be possible to repair any incipient or observed sinkholes by grouting, if the monitoring system worked the way it was supposed to.

Pile or caisson foundations were considered for areas of deeper rock, obviously an expensive operation. In addition, driving piles or installing caissons in areas of fractured, solutioned and pinnacled carbonates would likely require some form of drilling at foundation locations to ensure that they were either supported by relatively sound rock or perform grouting at suspect locations.

Excavation to competent rock, then filling any revealed cavities with a fluid grout was an alternate for areas of shallow rock. The cost of excavation to the expected depths (20 to 30 feet below grade in the better area) would be high. In addition slope stability and the continuing use of the service area would be a concern for any excavation of this depth.

The final and most cost-effective alternate for use in areas of deep or shallow rock was remedial grouting, limited to the area immediately below the fuel storage tank area. This would entail drilling into the rock and sealing any open joints or cavities so that the supporting soils below the tank could not be eroded into void areas. The form of remediation suggested was a karst exploration/grouting program as detailed in Fischer, et al, 1992.

After much deliberation by the owners and operators of the service plaza, this alternate was chosen. As a result of the protracted negotiations and discussions between the service plaza's owners and operators, the period of time between our first site visit, which precipitated the closing of the fuel supply services at the plaza, and the initiation of the tank support remedial program was about one year.

Remedial Grouting Program

Subsequent to the contract award and prior to the initiation of our work at the site, sheet piles were installed in a 41 by 36 foot rectangular area just outside the proposed tank locations. The existing tanks were removed and the area backfilled with 3/8-inch stone. Both the placement of the stone backfill and site work being performed on nearby storm sewers complicated drilling and grouting operations at the tank site. As subsequently discussed, the stone backfill resulted in changes in the drilling and grouting procedures and increased costs as a result. It was originally planned to use one rotary-wash drilling rig and one air percussion (airtrack) rig for the drilling operations. However, as result of the placement of the stone backfill prior and during the start of the drilling and grouting operations, it was necessary to install drill casing to limit grout flow into the stone, a function that is difficult and slow for an airtrack. As a result, two conventional truck-mounted drilling rigs were used. In addition, owing to the weight of the truck-mounted drilling equipment and the sloped nature of the incomplete stone fill, it was necessary to occasionally employ heavy site equipment to facilitate the movements of the drilling rigs.

The first two operating days were primarily spent drilling exterior borings just inside the sheet pile wall and installing 2-inch PVC "tremie" pipe for subsequent grouting. As a result of the free-flowing nature of the gravel backfill, 4-inch steel casing was installed through the gravel layer (about 17 feet deep). In the initial boreholes, the outer steel casing was removed leaving only the 2-inch PVC tremie pipe in-place.

Occasional soil samples were taken in the natural overburden soils to assure that they could support the tank loads. A combination of rock coring at certain locations and drilling "feel" were used to develop an understanding of the conditions of the rock at each boring location. This information was used to continuously update our geologic model of the site and plan the future grouting operations.

The exterior grout holes were placed about 2 feet inside the sheet pile perimeter as a result of a 14-inch wide whaler extending along the interior of the sheeting. Grouting was initiated three days after the drilling began. In general, the initial drilling and grouting pattern followed a ten foot spacing (see Figure 3 for Plan).

At the completion of each boring, a fluid grout containing sand, cement, water, and bentonite (to control shrinkage) was tremied into the probe hole, to seal any open rock or soil passages that might cause sinkhole occurrence. The proportions of these ingredients were varied depending upon the conditions encountered during the drilling and grouting operations. To limit grout movement to areas outside that being remediated, calcium chloride was added to the grout (to accelerate set time) introduced into exterior probe holes with excessive grout-takes. Grout was mixed in a double-tub mixer and delivered to the grout hole with a single-piston pneumatic pump.

On the first day of grouting, it was determined that we could not effectively "jack" the PVC out of the ground (as is normally accomplished) because of the gravel backfill settling around the pipe. Where possible, a drill rig was used to remove the PVC pipe. However, this procedure slowed drilling operations and resulted in pipe breakage in some instances, necessitating the redrilling of some of the grout holes, as well as resulting in the introduction of grout into the stone backfill in one hole. To eliminate this problem, it was necessary to leave the outer drill casing in place while the grouting operations were conducted through the PVC "tremie" pipe which could be removed as grout filled any cavities and line pressures increased.

As the exploration/grouting operations continued, secondary grout holes were drilled and, in general, these holes exhibited little grout-take indicating the effectiveness of the primary pattern. Secondary holes were also drilled in areas of larger grout takes to both check the effectiveness of the primary grout holes and to add grout at these locations.

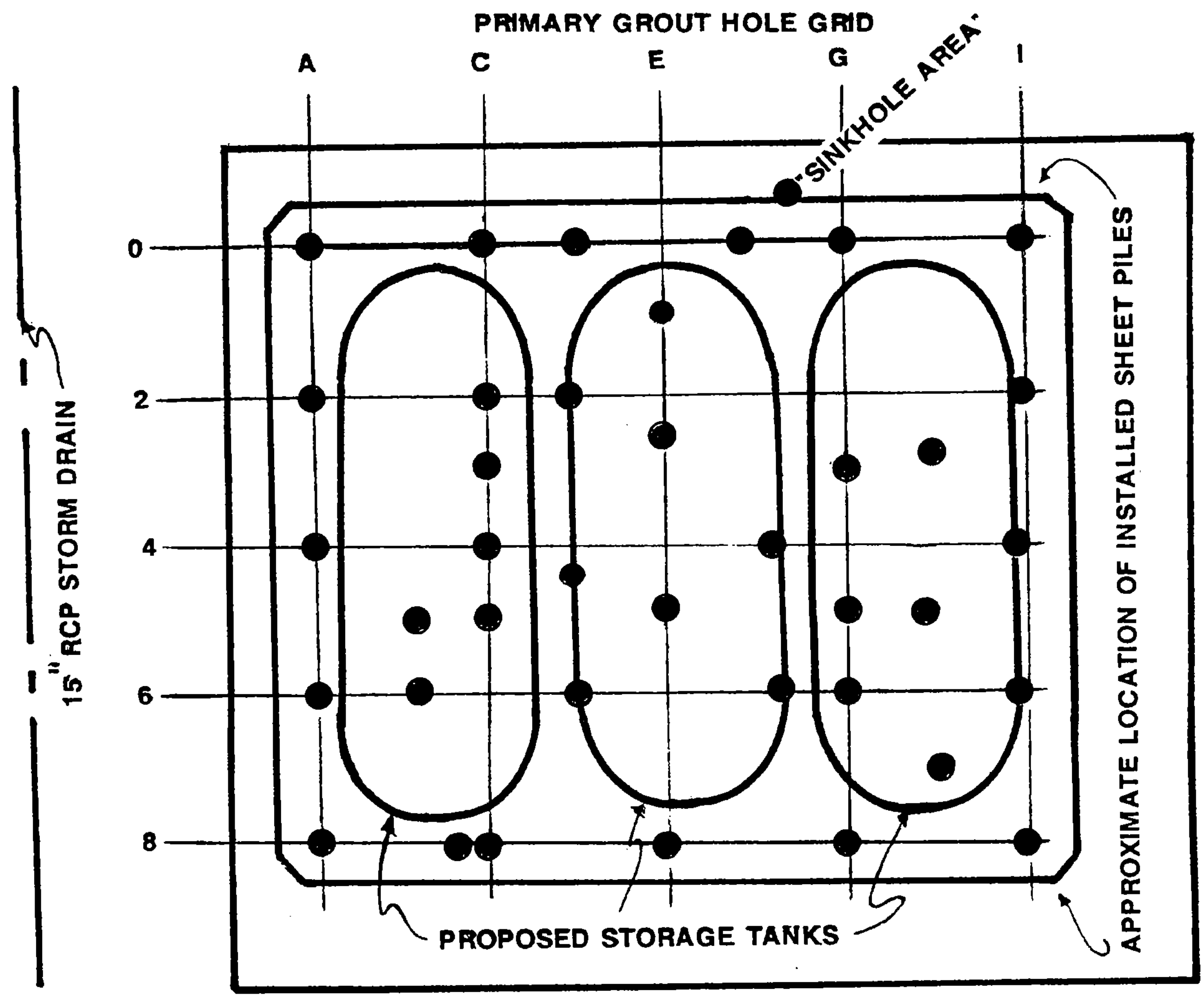

Figure 3 -- Grouting Plan

An additional probe hole was added exterior to the sheet pile wall (see Figure 3) at the location of a sinkhole which started to form several times over the course of the drilling/grouting operations. The activity of this sinkhole seemed to be precipitated by the drilling and grouting operations.

Thirty nine exploratory/grout holes were drilled. The volume of grout placed in each probe hole was monitored and was provided in the report. Table 1 is typical of the information provided to the client.

TABLE 1 - DRILLING AND GROUTING SUMMARY							
PROBE #	DATE		DEPTH OF (IN FEET):			GROUT-TAKE	
	DRILLED	GROUT-ED	TOP OF ROCK	WATER LOSS	PROBE HOLE	TOTAL (CU FT)	RATIO TO HOLE VOLUME
A-0	2/19	2/22	27.5	NONE	32	3.49	1.25
A-2	2/21	2/23	43	43	48	4.98	1.19
A-4	2/21	2/26	31	34	54	6.64	1.41
A-6	2/22	2/28	40.5	NONE	43	4.20	1.12
A-8	2/22	2/28	36	41	51.5	7.00	1.56
B-5	2/27	2/27	31	NONE	31	5.97	2.21
B-6	2/23	2/26	27	43	46	191.16	47.66
C-0	2/19	2/22	29	25	36	6.84	2.18
C-2	2/22	2/23	33	19	36	53.80	17.14
C-3	2/28	2/28	28	NONE	30	11.94	4.56
C-4	2/23	2/28	31	32	36	27.62	10.22
C-5	2/28	2/28	33	33	33.5	9.46	3.24
E-3	2/26	2/27	29	29	40.5	179.28	50.77
F-4	2/29	2/29	35.5	27.5	36.5	28.46	8.94
F-6	2/29	2/29	22.5	NONE	23.5	4.73	2.31

The field exploration and grouting program was performed under the technical direction of experienced field personnel. Each drill rig had a crew of two with the grout crew generally consisting of three men. The grouting engineer was responsible for: 1. Identifying and describing the conditions and materials encountered; 2. Specifying the depth of each borehole; 3. Specifying the grout mix for each hole drilled; 4. Determining tremie pipe and casing depth; and 5. Ordering material for use when needed (so as not to be left with a glut of materials) while keeping operations running smoothly and efficiently. Obviously, an impossible task.

A total of 57 cubic yards of grout was placed below the 41 by 36 foot tank area during this remediation program. Placement of what amounts to six truck loads of transit-mix concrete is evidence of the solutioned nature of the tank area subsurface and reinforces the earlier concerns for tank support. Spilling petroleum products into a cavernous subsurface such as this would certainly result in rapid contaminant transport in unknown directions.

Chasing Sinkholes

As previously mentioned, water flows from an unknown source were noted in the storm water sewer on-site even during periods of dryer weather. These flows dissipated with distance along the sewer line, indicating some leakage from the sewer system. A number of cracks were found in the storm sewer during site work in the tank area. This water likely found passage into fractures and openings, carrying soil into the bedrock, thus causing some of the sinkholes on-site. The remediation work performed to prevent sinkholes directly below the fuel storage tanks would have changed these subsurface flows, likely causing the sinkholes to appear elsewhere on the site. This possibility was discussed with the plaza operators and owners. The operators of a high pressure natural gas line crossing the site were notified of our concern.

As envisioned, sinkholes started to appear outside the tank area within a month of completing the initial grouting. A quick repair was made on the first one. Additional sinkholes appeared within a short time, some rather large. One sinkhole exposed the aforementioned gas line traversing the site as well as a sanitary manhole. When informed about the sinkhole exposing the gas line, the gas company was again apparently unconcerned. Later site work to repair the manhole caused it to tip and breach the gas line. Naturally, all site work stopped and the gas supply to the plaza's restaurant was interrupted, causing a scarcity of Big Macs.

On two separate occasions, a program of exploration grouting was affected to repair the sinkholes, just to have them appear elsewhere. The sinkhole repair program entailed very similar concepts and used the same grout mixes as those utilized below the tank field. The sinkhole area was ringed with grout holes and then the center filled. In an effort to fill as many rock and soil voids as possible, no accelerators were used in the last repair program so as to allow the fluid grout to flow freely. Some 11 cubic yards of grout was placed in 5 boreholes in one sinkhole area.

Just prior to the second repair program, additional site work had revealed a leaking water line. This water line was repaired prior to our last remediation activities and several subsequent "windshield reconnaissances" of the site have indicated no additional sinkhole occurrence to date.

Summary and Conclusions

If a highway or related facility is located in karst, remediation will likely be necessary in at least a portion of the site area, either prior to or after construction. The judgement as to the type of remediation requires an understanding of the geologic conditions and the nature of the foundation support or remediation economically feasible. Having a P.E. or P.G. after a name does not necessarily equate to having knowledge and experience in karst.

An exploration/grouting program, as discussed herein, conducted with appropriate insight and equipment, is often a technically viable and economically feasible approach to remediation in karst. Any remedial action must consider the source(s) and cause(s) of sinkhole occurrence and where possible, the causative source should also be remediated.

Solutioned carbonates are an extremely challenging engineering geologic problem. Answers are often difficult to find and their efficacy difficult to quantify.

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Management of Highway Stormwater Runoff in Karst Areas—Baseline Monitoring and Design of a Treatment System for a Sinkhole at the I-40/I-640 Interchange in Eastern Knoxville, Tennessee

by

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Abstract

Groundwater is particularly vulnerable to highway-derived contamination in karst areas. The primary goal of this project is the development and evaluation of practical remedial measures for treating highway runoff draining into sinkholes.

A sinkhole draining the I-40/I-640 interchange in eastern Knoxville, Tennessee, was selected as one of two sites for testing the contaminant-removal effectiveness of a prototype treatment system. (The other is adjacent to I-70 in Frederick, Maryland.) Based on results of laboratory testing of various materials, the pilot system is being designed to use peat, sand, and limestone to remove contaminants by sedimentation, filtration, and adsorption.

The 60-acre (24-hectare) drainage basin of the sinkhole includes the interchange, where the average daily traffic exceeds 86,000 vehicles, as well as adjacent grassy and wooded areas. Quantitative dye tracing and hydrograph analyses indicate that water draining into the sinkhole passes through a phreatic cave passage and resurges at Holston Spring, which lies approximately 420 feet (128 m) away. Stormwater quantity has been monitored for more than 1.5 years, and runoff quality has been monitored during a storm event. For most of the contaminants analyzed, peak contaminant loading at Holston Spring occurred approximately 1 hour after the peak at the sinkhole.

Introduction

P.E. LaMoreaux & Associates, Inc. (PELA) is conducting an assessment of potential groundwater contamination by highway stormwater runoff in karst areas under contract to the Federal Highway Administration (FHWA). The primary goal of this project is to develop and test a method for treating highway runoff draining into sinkholes. Potential sites for pilot testing of runoff treatment technology have been evaluated in each of the fifteen states participating in the research: Arkansas, Florida, Illinois, Indiana, Kentucky, Maryland, Minnesota, Missouri, New York, Oregon, Pennsylvania, Tennessee, Texas, Virginia, and Wisconsin.

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

Stephenson and Beck (1995) present a review of literature regarding the quality of highway runoff and its potential impacts on karst groundwater. Stephenson and others (1995) discuss the evaluation of candidate test sites and the selection of sites in Knoxville, Tennessee, and Frederick, Maryland. Stephenson and others (1996) describe the Knoxville site and general investigation plan, and Beck and others (1996) discuss the results of preliminary baseline monitoring of runoff quantity and quality in Knoxville. This paper presents a more detailed account of baseline monitoring in Knoxville, as well as a summary of laboratory-based testing of treatment alternatives conducted by the UT Department of Civil and Environmental Engineering (Smoot and others, 1997).

A pilot treatment system will be installed and tested at the I-40/I-640 sinkhole in eastern Knoxville, Tennessee. Water samples collected before and after treatment will be analyzed to evaluate water quality improvement. The hydrologic performance will also be monitored to assess the long-term viability of the system and projected maintenance requirements.

Site Description

Approximately 86,000 vehicles pass through the I-40/I-640 interchange each day. The site is underlain by sinkhole-forming Ordovician limestones of the Holston Formation which have been thrust-faulted over younger sandstones of the Chapman Ridge Formation (Cattermole, 1966). Originally, stormwater runoff from approximately 60 acres (24 hectares) of the interchange and adjacent land was directed through ditches and culverts to a small detention basin designed to treat runoff flowing into the “old sinkhole.” However, all the highway runoff now enters the ground via a large “new sinkhole” which has collapsed within the basin. Water from this site passes through two small karst windows and is subsequently discharged at Holston Spring approximately 420 feet (128 m) from the site. Dye tracing indicates that the spring discharges groundwater from a narrow, strike-oriented drainage basin encompassing a variety of urban land uses southwest of the interchange (Ogden, 1994).

Stormwater drains into the new sinkhole by three routes (Figure 1). The most significant by volume is a 42-inch (107-cm), concrete culvert, which receives drainage from several inlets throughout the interchange. This pipe is referred to as the "large culvert." In addition, a shallow, 6-foot-wide (1.8-m-wide), concrete-lined ditch (referred to as the "ditch") carries runoff from the highway surface and an adjacent grassy area to the discharge end of the large culvert. From that location, runoff flows along a 12-foot-wide (3.7-m-wide), concrete channel (referred to as the "channel") into the sinkhole, which collapsed beneath the end of the channel. Finally, an 18-inch (46-cm), corrugated-metal culvert transmits drainage directly from the paved surface in the vicinity of a concrete median barrier.

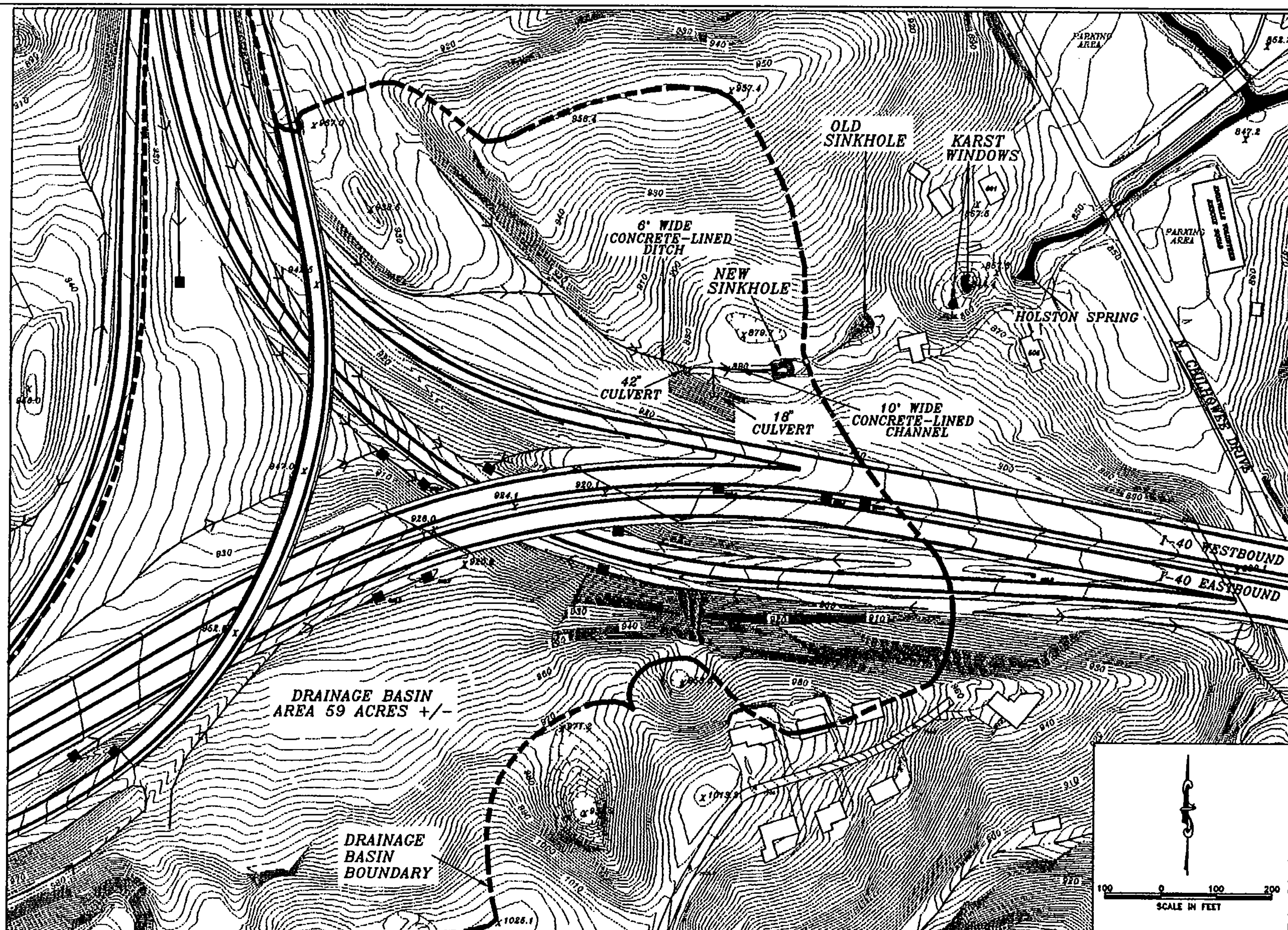

FIGURE 1. Site map for pilot highway stormwater runoff treatment system test site at the I-40/I-640 interchange in eastern Knoxville, Tennessee. (Base map from City of Knoxville.)

Referred to as the "small culvert," this pipe discharges stormwater from the embankment adjacent to the highway (approximately 15 feet [4.6 m] above the level of the channel). Runoff from the small culvert drops approximately 3.5 feet (1.07 m) to the ground and flows down a steep, vegetated slope and joins runoff from the large culvert and ditch in the channel before flowing into the sinkhole. The channel ends abruptly in a 15-foot (4.6-m) precipice at the edge of the sinkhole.

Runoff Quantity

The discharge of highway runoff entering the sinkhole and of water resurging from Holston Spring have been monitored continuously since July 21, 1995, to assist with the design of the pilot treatment system and to assess the data from quantitative dye-tracing experiments (Figure 2). In the future, discharge records will permit calculation of pollutant loads before and after treatment.

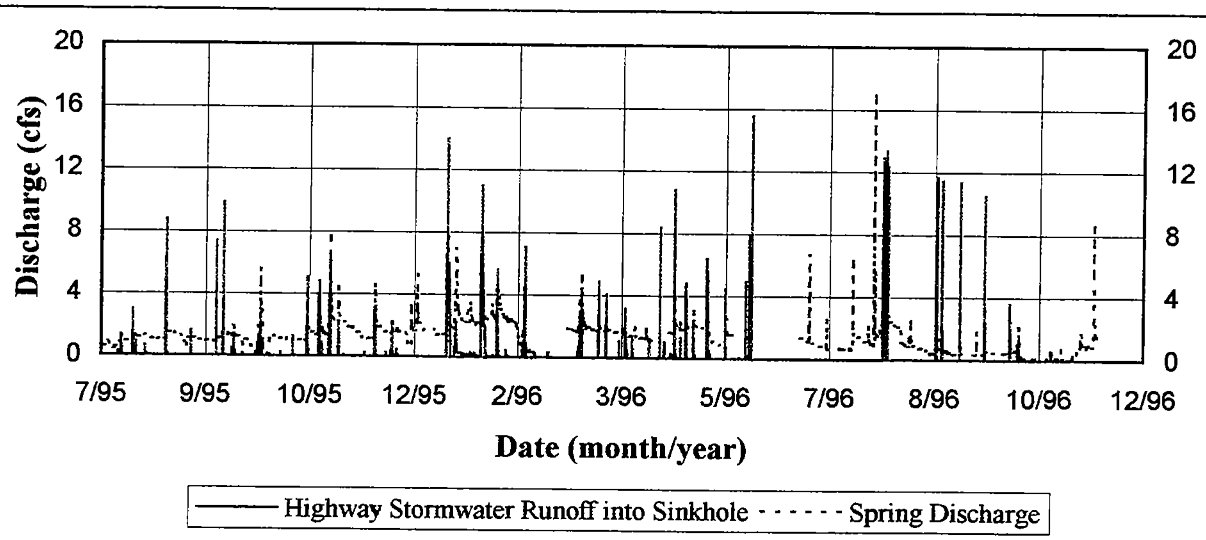

FIGURE 2. Hydrographs of highway stormwater runoff into sinkhole and discharge from Holston Spring.

Hydrographs from the sinkhole and spring indicate that water draining into the sinkhole resurges at Holston Spring and that the hydrologic connection between them is very direct. Stormwater runoff draining into the sinkhole from the interchange produces a sharp rise in the water level (and discharge) at the spring. This increase occurs almost immediately after the drainage begins flowing into the sinkhole, which suggests that these features are connected by a phreatic conduit—i.e., a cave passage that is completely filled with groundwater. While the actual stormwater runoff does not arrive at the spring immediately, it displaces water already flowing through the conduit from other parts of the groundwater drainage basin.

The peak discharge value at the spring is sometimes higher than the peak discharge at the sinkhole for the same storm. This confirms that the drainage basin for the spring extends well beyond the site, as indicated by Ogden (1994). However, storm peaks at the spring are only associated with runoff into the I-40/I-640 sinkhole, suggesting that the flow of stormwater entering the aquifer from other sinkholes in the drainage basin is regulated by partial blockage of the conduit and/or its tributaries by urban development of the karst terrane in eastern Knoxville. Flooding problems in upgradient sinkholes provides further evidence for this hypothesis.

Runoff Quality

Highway stormwater runoff and spring flow were sampled during a rainfall event on January 18, 1996. Sample collection occurred between 16:13 and 22:35 (Eastern Standard Time) during the first flush of runoff from an event which continued throughout the night. A total of 16 samples were collected at the sinkhole site, including six at the small culvert, five at the large culvert, and five at the edge of the sinkhole. A total of 11 samples were collected at the spring. Each sample was analyzed for the following constituents: dissolved Pb and Zn, total Pb and Zn, PAH, TPH, TDS, TSS, and TVS. Samples to be analyzed for dissolved Pb and Zn were filtered immediately after collection (and before acidification) using high-surface-area 0.45-micron filters.

Figure 3 presents hydrographs for drainage entering the sinkhole. The maximum runoff was 8 cfs (0.23 m^3/s) from the large culvert (including the ditch), which contributed 17,913 feet^3 (507 m^3) of runoff to the sinkhole. The peak runoff was only 0.4 cfs (0.01 m^3/s) from the small culvert, which contributed 1,044 feet^3 (29.6 m^3) of runoff to the sinkhole. Runoff from the small culvert accounted for only 5 percent of the highway drainage flowing into the sinkhole. Even though runoff from the small culvert contained the highest contaminant concentrations, its volume and contaminant load were minor, relative to runoff from the large culvert. The total volume of water flowing into the sinkhole exceeded 18,957 feet^3 (537 m^3). During this storm event, the maximum spring discharge was 0.18 m^3/s (6.5 cfs), and the volume of water discharged was 44,373 feet^3 (1.257 m^3).

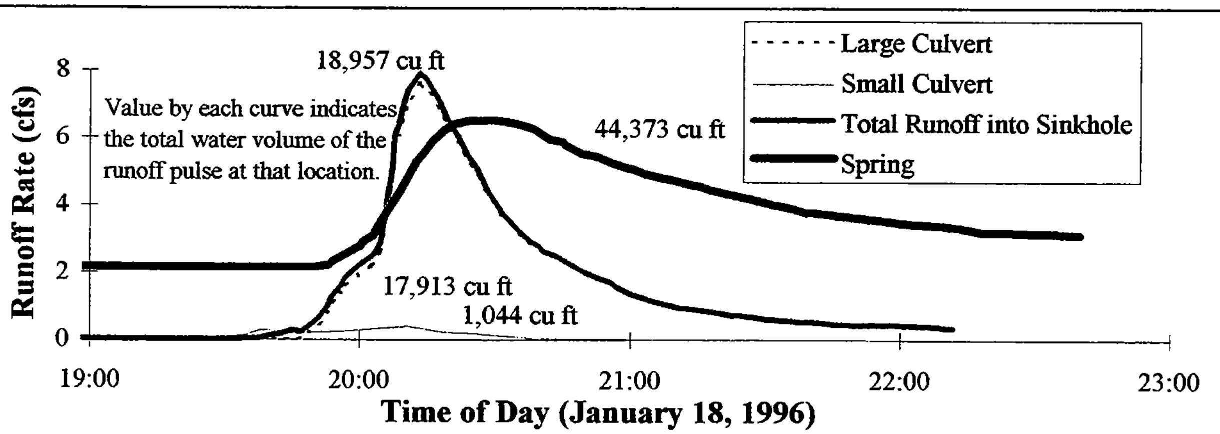

FIGURE 3. Hydrographs of highway runoff into sinkhole and discharge from Holston Spring during sampling on January 18, 1996.

Table 1 presents analytical results from a stormwater sampling event. At the sinkhole site, runoff from the small culvert generally contained higher concentrations (but lower loads) of the constituents than runoff from the large culvert. At the small culvert, the first sample had the highest contaminant concentrations, while the second sample had the highest concentrations at the large culvert.

TABLE 1. Contaminants in Stormwater Runoff and Spring Discharge, I-40/I-640 in Eastern Knoxville, Tennessee (January 18, 1996).

	Total Zinc (μg/L)	Dissolved Zinc (μg/L)	Total Lead (μg/L)	Dissolved Lead (μg/L)	TPH (mg/L)	PAH (μg/L)	TDS (mg/L)	TSS (mg/L)	TVS (mg/L)
Sinkhole	7900	2400	374	5	12.6	178	3584	1485	490
Spring	208	46	31	BDL	1.6	46	404	336	96

At the spring, concentrations of dissolved and total Zn, total Pb, TPH, and PAH were very low or non-detectable before 20:30. Dissolved lead was not detected at the spring. Total dissolved solids concentrations remained high during the entire sampling event, with the highest concentration occurring in the first sample. The concentrations of TSS and TVS were identical to each other until 20:30, after which TVS constituted only a portion of the TSS load. The peak concentrations for most of the constituents were observed in the sample collected at 21:15.

Contaminant loading rates (mass per unit time) were calculated by multiplying flow rates and concentrations for each sampled interval. For most contaminants, peak contaminant loading at the spring lagged behind the peak at the sinkhole by approximately 1 hour. For TDS, however, the peak lagged only 20 minutes behind. Generally, peak loading rates were lower at the spring than at the sinkhole, with the exception of dissolved Zn and TDS, which were higher at the spring. At the sinkhole, loading rates decreased rapidly after peaking; the decrease was slower at the spring. Contaminant loads (masses) for the sinkhole and spring were calculated by integrating loading rates over time for each sampled interval. Analysis of the data from the sinkhole shows that contaminant load is more closely related to runoff volume than contaminant concentration.

Dye Tracing

Quantitative dye tracing conducted during a storm event on November 2, 1995, confirmed that water draining into the new sinkhole resurges at Holston Spring. Rhodamine WT was introduced with stormwater runoff, which was flowing into the sinkhole at a rate of 4 cfs (0.11 m^3/s).

Dye recovery was monitored at Holston Spring, where grab samples were collected and analyzed in the field with a fluorometer. Field analysis was conducted to permit adjustment of the sampling interval during the dye breakthrough period. Sampling frequency ranged from one sample every 30 seconds around the time of the peak concentration to one sample every 15 minutes during the recession of the dye concentration curve. During a 6-hour period, 170 samples were analyzed on site. However, because field conditions are not ideal for fluorometric analysis, 129 of the samples were also analyzed in the laboratory the following day. This permitted more precise calibration of the fluorometer and provided time for temperatures to stabilize and turbidity to settle in the samples.

Based on accepted methodologies (e.g., Mull and others, 1988), aquifer and solute-transport characteristics were determined using data obtained from the dye-tracing experiment. However, most dye-tracing literature addresses tracing under constant-discharge conditions. Because this project deals specifically with the variable discharge conditions associated with stormwater runoff events, the published methods were modified accordingly. Also, it should be noted that the distance between the new sinkhole and Holston Spring may be too short to permit reliable estimation of some parameters because it does not allow for adequate mixing of dye and water.

Figure 4 shows the dye breakthrough curve, as well as the spring hydrograph for the period of the dye trace. Low levels of dye began to arrive at the spring approximately 20 minutes after it was introduced. The peak concentration (39.9 mg/L) was measured approximately 69 minutes after dye introduction. The spring water was slightly discolored, but the dull red color was obscured by turbidity resulting from the storm event. Dye concentration had dropped to 0.301 mg/L when the last sample was collected 5.5 hours after dye introduction. Analysis of a grab sample collected during storm flow five days later (November 7, 1995) indicated that the dye concentration was only 0.034 mg/L above the background level measured at the beginning of the tracer test, indicating that most of the dye had been flushed from the aquifer.

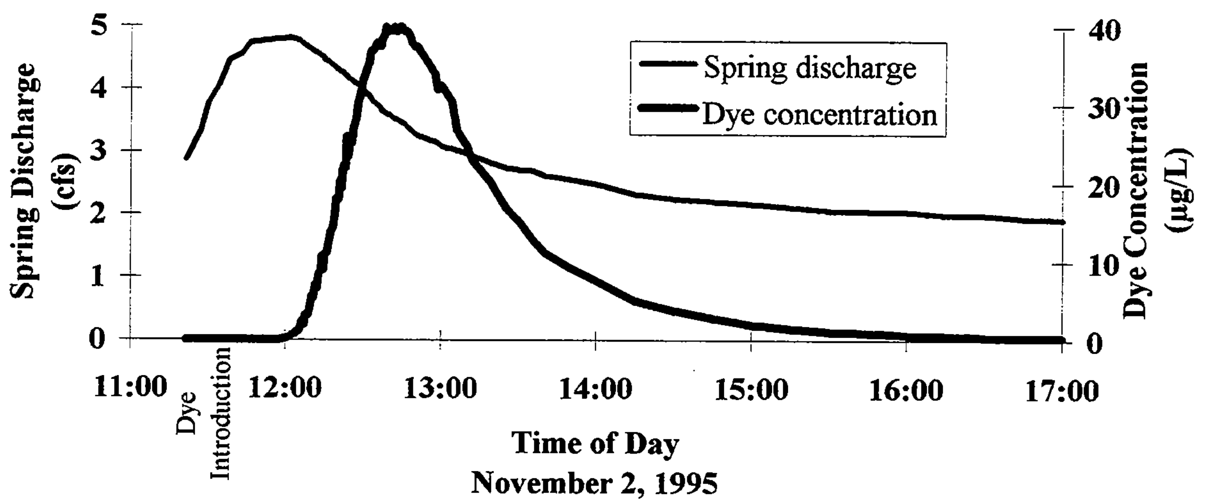

FIGURE 4. Dye breakthrough curve and hydrograph for Holston Spring during quantitative dye tracing on November 2, 1995.

The increase in discharge at the spring does not necessarily correspond with the arrival of highway runoff from the sinkhole. As mentioned previously, the increased spring flow probably results from the displacement of water already present in the karst conduit by the stormwater runoff draining into the sinkhole. This scenario is supported by two observations: 1) the peak discharge at the spring occurs at approximately the same time as the peak highway stormwater runoff into the sinkhole (Figure 2); and 2) the peak dye concentration did not arrive at the spring until approximately 40 minutes after the maximum discharge value (Figure 4).

The mean traveltime of the dye was 1.7 hours, with a standard deviation of 0.7 and a skewness coefficient of 0.14. In other words, half the dye (by mass) had passed through the spring about 1 hour and 40 minutes after dye introduction. Assuming that the sinkhole and spring are connected by a straight conduit, the apparent mean velocity of dye transport was 0.077 feet/second (0.024 m/s) or 1.26 miles/day (2.07 km/day). This value should be considered a minimum velocity because it is based on the assumption that the conduit is straight. The actual velocity is probably higher.

Design of a Pilot-Scale Treatment System

The treatment system will involve a combination of sedimentation, filtration, and adsorption designed to treat the first flush of stormwater runoff, which is the most heavily contaminated. It should be easily installed using readily available materials, resulting in a cost-effective, low-maintenance treatment system. Challenges involved include limited available land area; difficult terrain conditions; susceptibility to additional sinkhole collapse; periodic, large runoff events; daily and seasonal temperature extremes; and numerous other considerations. The design of the pilot treatment system addresses these challenges and is based on results of laboratory experiments conducted at UT.

The first phase of laboratory research involved batch studies to determine the relative removal characteristics of candidate materials proposed as sorption/filtration media (peat moss, activated carbon, crushed limestone, and calcined clay). Synthesized stormwater was used for the experiments. Well water from a karst aquifer in Oliver Springs, Tennessee was used as base from which the synthetic stormwater was developed. It was selected to avoid chlorination effects and to provide typical concentrations of major ions and other constituents which would be present in stormwater. Three heavy metals commonly found in stormwater runoff (copper, lead, and zinc) were used as indicators representative of typical contaminated stormwater. Metal standards or metal salts were added to bring their concentrations up to nationwide average stormwater levels. The final concentrations in the simulated stormwater water were approximately 3.1 mg/L zinc, 0.46 mg/L copper, and 3.7 mg/L lead. Hydrocarbons were not included in the batch studies.

After known amounts of simulated stormwater and treatment media were mixed and equilibrated, the solutions were analyzed for the selected metals. The concentrations were used as indicators of the relative contaminant-removal efficiencies of the materials. All media tested removed more than 90 percent of the copper in solution, and similar results were found for the other metals. Activated carbon was the most effective, removing more than 10 times as much copper per unit mass of media than the other materials. However, it may not be practical for use in a field application because of its cost. Peat moss also removed more than 90 percent of the dissolved lead.

The second phase of laboratory research involved column studies to simulate the field use of several possible designs. This phase was conducted to provide an indication of operating characteristics and contaminant-removal efficiencies, as well as to provide an indication of the field life for the system. Three 8.5-foot (2.6-m) vertical columns were constructed of 3-inch (7.6-cm) diameter Plexiglas tubing. Sampling ports were installed near the bottom for collecting effluent. Column 1 was packed with inert crushed stone and peat moss; column 2 included crushed limestone and peat moss; and column 3 included crushed limestone, inert crushed stone, and activated carbon.

For the column tests, the simulated average stormwater concentration for each contaminant was increased by an order of magnitude and doubled to simulate two first-flush events, reducing the amount of water and time needed to produce the equivalent of several years of storms. Concentrated stormwater was prepared in a 5.3-gallon (20-liter) plastic carboy. Prior to each sequential test, 6.76 fluid ounces (200 ml) of the concentrated stormwater was adjusted to pH 8 and diluted to 5.0 gallons (19 liters) with base water. Four grams of used motor oil was added to the mixture and stirred continuously for a minimum of 4 hours. After agitation, the synthetic stormwater was poured into the top of each column. The influent and effluent from each column were sampled. The raw simulated stormwater (influent) and column effluents were analyzed for lead, copper, zinc, total organic carbon (TOC), pH, conductance, tannin-lignin composition, alkalinity, polyaromatic hydrocarbons (PAH), total dissolved solids (TDS) and oil and grease using standard methods and instrumentation. Additionally, quality assurance samples were analyzed by the PELA laboratory. Each of the columns received the equivalent of 60 first-flush stormwater runoff events.

Results of the column testing showed that the contaminant-removal efficiency of the system generally ranged from about 90 percent to more than 99 percent for the selected indicator metals throughout the simulated expected life of the system. Similar removal efficiencies were found for total polyaromatic hydrocarbons—an indicator of contamination from used motor oil.

Following the column testing, each column was dissected and sectioned into 10 layers. The mass was determined of each metal in each layer of each column to evaluate breakthrough. The bulk of the metal mass introduced into each column did not penetrate very far into the column during the testing, suggesting that they had significant additional life remaining after testing. Copper penetrated each column the most, while zinc penetrated the least. The responses were similar for all three columns.

Although the pilot-system design is still being finalized, a preliminary version is presented in Figure 5. The pilot system is being designed to treat the first flush of highway stormwater runoff. To prevent flooding at the site, the structure will be designed to permit subsequent runoff to bypass the treatment process and flow directly into the recently-collapsed sinkhole and/or the pre-existing sinkhole. The treatment media (peat, sand, and limestone) will be placed in horizontal layers in a lined retention basin within the sinkhole. A perforated drainage pipe beneath the peat will collect treated drainage and conduct it into the sinkhole. The surface of the treatment system will consist of a layer of stone to minimize clogging and to facilitate maintenance.

The drainage conduit from the treatment structure into the sinkhole will be designed to permit sampling of the treated runoff as it recharges the karst aquifer. The load (mass) of each indicator contaminant in the treated runoff will be compared with the load upstream of the treatment system to evaluate its effectiveness. Because of the time lag resulting from detention and treatment of the runoff, the total load (not concentration) of each contaminant will be used. Samples may also be collected to determine event mean concentrations (EMCs) at Holston Spring to assess changes in water quality at the aquifer scale.

Preliminary Conclusions

- The susceptibility of groundwater to contamination by highway stormwater runoff is enhanced in karst areas because surface drainage may flow directly into karst aquifers with little or no natural attenuation and because it may travel rapidly through the aquifer, transporting highway-derived contaminants from sinkholes to distant cave streams, water wells, springs, and surface streams.

- Stormwater runoff from the I-40/I-640 interchange in eastern Knoxville, Tennessee, drains into a recently-collapsed sinkhole and flows rapidly to Holston Spring through a phreatic conduit. During a storm event with a peak spring discharge of 5 cfs, the apparent mean velocity of dye transport between the sinkhole and Holston Spring was about 1.26 miles/day (2.07 km/day), although the actual velocity was probably higher.

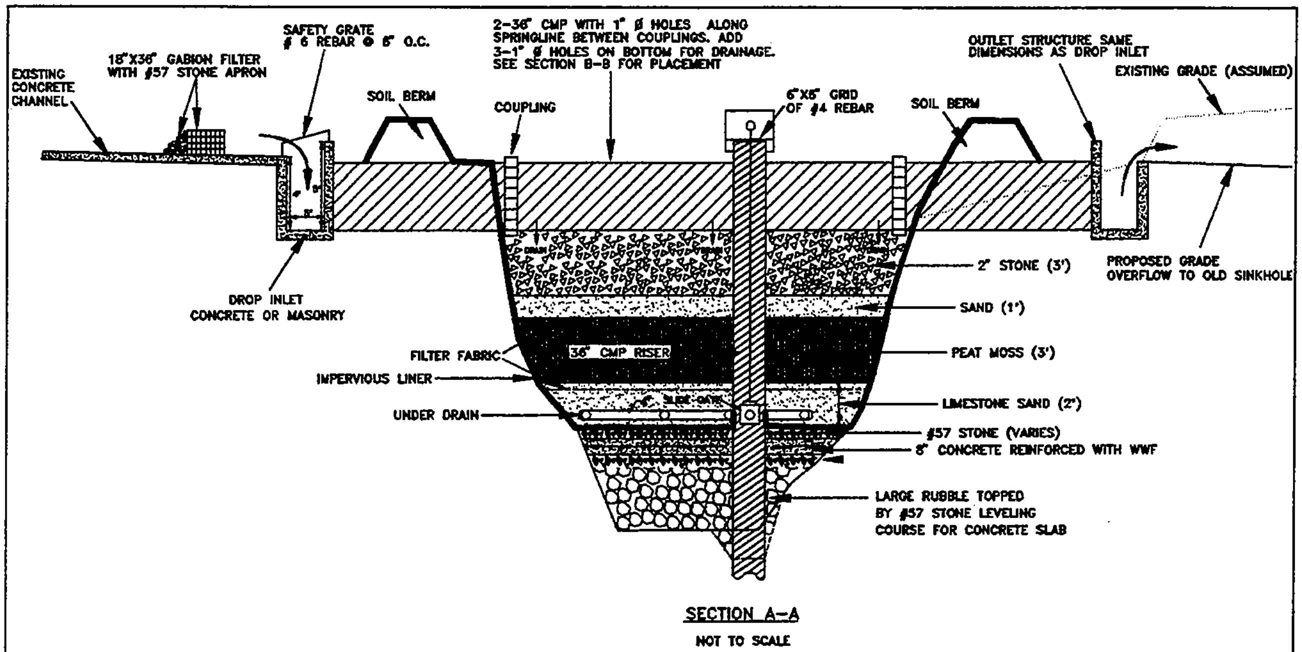

FIGURE 5. Preliminary design (cross section) of a passive system for treating highway stormwater runoff at the I-40/I-640 interchange in eastern Knoxville, Tennessee.

- For most of the contaminants analyzed, peak contaminant loading at Holston Spring lagged behind the peak at the sinkhole by approximately 1 hour. For TDS, however, the peak lagged only 20 minutes behind.

- Generally, peak loading rates were lower at the spring than at the sinkhole, with the exception of dissolved Zn and TDS, which were higher at the spring.

- Contaminant load is more closely related to the volume of runoff than contaminant concentration.

- The movement of stormwater from other sinkholes in the drainage basin to Holston Spring is regulated by partial blockage of the conduit-dominated flow system. Urban development of the karst terrane in eastern Knoxville may be responsible for this observed phenomenon.

- The tested sorption/filtration systems can achieve contaminant-removal efficiencies in the range of 90 to 99 percent.

- Peat moss may be the best material for many stormwater treatment applications because of its low cost, ease of use, hydraulic conductivity, and removal efficiency. However, a crushed-limestone component should also be considered to minimize acidity and loss of efficiency.

Acknowledgments

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STABILIZATION OF KARST FEATURES ON MAJOR ENGINEERING PROJECTS

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Presented By:

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STABILIZATION OF KARST FEATURES ON MAJOR ENGINEERING PROJECTS

B. Dan Marks, Ph.D., P.E.[1]

ABSTRACT

The presence of sinks, grikes, sinkholes and other karst terrain features present on otherwise desirable land tracts has historically resulted in reduced property value and/or complete avoidance of affected property relative to development of major engineering projects. However, continued demands for accessible land and changes in cost-benefit ratios now make major projects on previously undesirable tracts affected by karst features economically feasible. As such, site development in karst areas often involves stabilization and treatment of sinks, grikes, sinkholes, and other karst features.

For the purpose of this paper, limestone sinks are defined as karst features having low to moderately active internal drainage and do not exhibit evidence of recent collapse. Limestone sinks might be considered inactive sinkholes. Sinkholes are defined as recent collapses in residual soils above calcareous bedrock or karst features which continue to have active internal drainage. Grikes are open joints in calcareous bedrock exposed at the surface which are typically linear features as opposed to caves or caverns.

This paper presents a comparison of three methods of stabilization and/or treatment of sinks and sinkholes at four major project sites. The four sites are located within three distinctly calcareous bedrock (karst) geologic settings from south-central Pennsylvania to central and southwest Virginia. The three methods of stabilization and/or treatment include: 1) excavation or undercutting of organic material from sinks and backfilling with properly compacted, low permeability clay fill; 2) excavation of sinkholes and installation of inverted filter drainage systems; and 3) excavation of sinkholes and stabilizing with a combination of concrete grouting and installation of inverted filter drainage systems.

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STABILIZATION OF KARST FEATURES ON MAJOR ENGINEERING PROJECTS

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PROJECT DESCRIPTIONS

The four project sites presented herein are located at Lancaster, Pennsylvania; Roanoke, Virginia; and Abingdon, Virginia. The project near Lancaster, in Lancaster County, Pennsylvania consisted of the construction of a new parallel taxiway with two connector taxiways and improvements to the stormwater drainage system adjacent to and beneath the new taxiways at the Lancaster Municipal Airport. Two projects in Roanoke, Virginia involved major improvements to the Roanoke Regional Airport and construction of a large regional mall immediately south of the airport. Improvements to the Roanoke Regional Airport involved an approximately 275 meter (900 feet) extension of Runway 23 and associated parallel taxiway. Extension of the runway and taxiway required the relocation of Virginia Highway 118 through a vehicular tunnel beneath the runway and taxiway, relocation of a major stream, and construction of soil/rock fills with maximum heights of 38 meters (125 feet). Valley View Mall is a large regional shopping mall with seven anchor stores. Grading of the site for the mall and associated parking areas required maximum cuts of five to six meters (16 to 20 feet). The primary drainage system consists of a 1.8 meter (72 inch) diameter, concrete lined, corrugated metal pipe which empties into an unlined permanent stormwater detention basin. The project in Abingdon, Virginia consists of the reconstruction of the existing runway with changes in grade that required maximum fills of about four to five meters (thirteen to sixteen feet) at the east end of the runway and installation of a stormwater drainage system to drain the infield and apron areas. Borrow material for the fill at the east end of the runway was obtained from an area north of the existing runway which was used for construction of a new runway.

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The project locations form a northeast to southwest line which parallels the strike of the Appalachian Uplift. The line connecting the projects lies within the valley and ridge physiographic provinces of Pennsylvania and Virginia.

GEOLOGIC SETTINGS

Lancaster, Pennsylvania is located within the Great Valley and Northern Piedmont Physiographic and Geologic Province of Pennsylvania. Limestone formations in the Lancaster area are of Ordovician Age and are within the Lebanon Valley Sequence. The Lancaster Municipal Airport is located north of Lancaster, Pennsylvania in the extreme north-central portions of Lancaster County just east of Pennsylvania Highway 501. The airport is located very near the contact of the Stonehenge Formation with the Epler Formation. These formations are relatively thinly bedded, or laminated, crystalline limestones that exhibit very gentle to moderate dips. The Stonehenge Formation is a gray finely crystalline limestone containing dark gray silty laminations and numerous edgewise conglomerate beds. The Epler Formation is a very fine-grained crystalline light gray limestone interbedded with gray dolomite and coarsely crystalline limestone lenses. Residual soils overlying the limestone bedrock are typically fine sandy silty clays, silty fine sandy clays, and fine sandy clayey silts classified by the Unified Soil Classification System as CL, CH, CL-CH, ML, MH, and ML-MH soils. Residual soils vary in thickness from about one to four meters (three to thirteen feet) with no evidence of limestone outcropping in the proposed construction area. Figure 1 illustrates a typical subsurface profile at this site.

Roanoke, Virginia is located near the boundary of the Blue Ridge and the Valley and Ridge Physiographic and Geologic Province of Virginia. However, both the Roanoke Regional Airport and the Valley View Mall projects are located within the Valley and Ridge Province. The airport project is located very near the Salem Thrust Fault which is reflected by steep bluffs and stream channels that form the transition from limestone to shale bedrock formations. The airport project is influenced by three prominent Ordovician to Precambrian Age geologic formations. The Beekmantown Formation is a laminated gray limestone and thick-bedded dolomite, and thin-bedded sandstone. The Elbrook Formation is a thin to thick-bedded limestone, dolomite, and shaley dolomite. The third formation at the airport project is a shale unit which is the Liberty Hall facies of the Edinburg Formation; however, this unit has also been identified as the Athens Shale Formation. The shale unit is exposed at the toe of the steep limestone bluff within the floodplain of the stream near the limestone bluff at the north end of the airport property. The Valley View Mall project is south of the Roanoke Regional Airport and is further removed from the Salem Thrust Fault. As such, the mall project is influenced primarily by the Elbrook Formation. Residual soils at both the airport and mall projects range in depth from three to twelve meters (ten to forty feet) with limestone outcrops visible on steep limestone bluffs at the airport site. Residual soils at both sites are fine sandy silty clays, silty fine sandy clays and fine sandy clayey silts. These residual soils are classified as CL, CH, CL-CH, Ml, MH, and ML-MH soils according to the Unified Soil Classification System. Figure 2 illustrates a typical subsurface profile at both the airport and mall projects in Roanoke, Virginia.

Abingdon, Virginia and the Virginia Highlands Airport, south of Abingdon and just north of Interstate 85, are located within the central portion of the Valley and Ridge Physiographic and Geologic Province of Virginia near the border of Tennessee. The Virginia Highlands Airport project is influenced by the Conococheque Formation which is of Ordovician to Cambrian Age. The Conococheque Formation is a relatively massive formation of limestone, dolomitic limestone, and dolomite limestone with interbeddings of calcitic sandstones and sandy dolomitic limestones. Residual soils at the site are typically four to eight meters (fifteen to 26 feet) in depth except along prominent ridges where residual soils are typically two to three meters (six to ten feet) in depth. Residual soils at the Virginia Highlands Airport are typically fine sandy silty clays, fine sandy clayey silts and silty fine sandy clays. These residual soils are classified according to the Unified Soil Classification System as CL, CL-CH, CH, ML, ML-MH, and MH soils. A typical subsurface profile at this site is presented in Figure 3.

DESCRIPTIONS OF KARST FEATURES

The four project sites were known to be located within areas of both ancient and recent karst development activity. As such, detailed site reconnaissances were conducted at each site both prior to and during the subsurface exploration program phases of geotechnical investigations for these sites. The character and extent of karst features at each of these project sites were quite different as related to variations in the limestone formations at the sites.

Site reconnaissances at Lancaster Municipal Airport revealed the presence of numerous ancient and active karst features in the infield area between the existing runway and new taxiway alignments. Most of the ancient (inactive) karst features have been filled with limestone rip-rap stone, crushed stone, and random soil fill. Some of these ancient karst features retained active internal drainage as evidenced by the development of voids beneath adjacent stormwater drainage lines and development of new collapses adjacent to ancient features subsequent to heavy rains which occurred during grading of the new taxiway. Newly developed collapses or sinkholes in the infield area were found to be circular in shape and one to three meters (three to ten feet) in diameter with typical depths of 0.6 to 1.2 meters (two to four feet). Inspection of the sinkholes revealed that they were directly associated with joints in the existing corrugated metal pipe stormwater drainage system.

The magnitude of the Roanoke Regional Airport project and complexities of site geology required that an extensive site reconnaissance and geologic mapping program be incorporated into the geotechnical investigation program for this project. Site reconnaissances revealed that one major limestone sink and one major sinkhole existed within the alignment of the taxiway extension. Both karst features had previously been filled with debris and rubble fill. The active sinkhole was near the alignment of the vehicular tunnel to route Virginia Highway 118 beneath the runway and taxiway extensions. Both features were oval-shaped with dimensions of approximately 45 to 91 meters (150 to 300 feet) by approximately 24 to 46 meters (80 to 150 feet) with depths of three to six meters (ten to twenty feet). The limestone sinks and sinkhole features at this site were found to be near the contact of the Beekmantown and Elbrook Formations with the Liberty Hall shale unit of the Edinburg Formation. Only one minor sinkhole collapse adjacent to the large sinkhole feature observed in site reconnaissances occurred during grading of this site.

Site reconnaissances for the Valley View Mall project site revealed the existence of a large sinkhole feature near Hershberger Road north of the mall site. This sinkhole was protected from sedimentation or siltation and had apparently been used for drainage of stormwater runoff from Hershberger Road for many years. In addition to this feature, an old limestone sink was observed within a portion of the mall site which was to be raised in grade by the placement of two to three meters (seven to ten feet) of fill. The old limestone sink was approximately 45 by 91 meters (150 by 300 feet) and one to two meters (three to seven feet) in depth. This feature had little active drainage as evidenced by the ponding of surface water for extended periods of time. The old limestone sink contained one to two meters (three to seven feet) of topsoil and organic silt above six to eight meters (20 to 25 feet) of firm residual soil overlying parent limestone bedrock. No other limestone sinks or sinkholes were apparent on or adjacent to this project site during site reconnaissances. However, two sinkhole collapses occurred in the vicinity of the permanent stormwater detention structure and stormwater drainage system during construction. Subsequent to excavation and construction of the stormwater detention basin, a sinkhole collapse approximately three to four meters (ten to thirteen feet) in diameter and four to five meters (twelve to fifteen feet) in depth occurred near the bottom of the east side of the basin in close proximity to the outlet of the 1.8 meter (72 inch) diameter stormwater system. This sinkhole feature eventually became filled with silt and eroded soil and exhibited no further active internal drainage. A large sinkhole collapse occurred beneath the third joint of the corrugated metal pipe system during excavation and installation of the first three sections of the stormwater drainage system. The new sinkhole collapse eventually extended over an area of about five by 21 meters (fifteen by 70 feet) and was six to eight meters (twenty to 26 feet) in depth.

Site reconnaissances at the Virginia Highlands Airport project site revealed that the site had experienced the formation of sinkholes over a long period of time. Two old limestone sinks were apparent north of the existing runway. These features were typically circular in shape with diameters of 46 to 61 meters (150 to 200 feet) and depths of three to four meters (ten to thirteen feet). These features exhibited moderate to relatively inactive internal drainage as evidenced by ponding of surface runoff. One relatively large sinkhole existed within the infield area. This sinkhole feature was previously excavated and backfilled with random stone fill to provide drainage of the infield and parking apron. During recent years the sinkhole had become less active relative to internal drainage and several smaller sinkhole collapses had occurred along the corrugated metal pipe drainage system leading to the sinkhole. The sinkhole apparently became less active relative to internal drainage as a result of siltation and clogging of the stone backfill which was not designed to provide filtration during drainage. The more recent sinkholes in the infield and apron areas were three to four meters (ten to thirteen feet) in diameter and 2.4 to four meters (eight to thirteen feet) in depth. Soil lost by ravelling at these sinkhole features apparently was transported into the large sinkhole feature.

STABILIZATION AND TREATMENT PROCEDURES

Limestone sinks and sinkholes were stabilized and treated at the four project sites by the application of three procedures. The method selected for stabilization or treatment was dependent upon: 1) activity of the internal drainage of the sink or sinkhole feature; 2) depth of fill to be placed above the feature after stabilization or treatment; 3) type of structure to be supported above the feature after stabilization or treatment; and 4) the extent of excavation into the limestone formation possible with conventional excavation equipment.

Two methods of sinkhole stabilization and treatment were utilized at the Lancaster Municipal Airport. The first method, utilized in treatment and stabilization of the larger sinkhole which was previously filled with random soil and rockfill, was excavation into the limestone formation to isolate the solution joint or "throat" of the sinkhole and installation of an inverted filter drainage system. Since this procedure was utilized at both Roanoke, Virginia sites, the filtration drainage system utilized at Lancaster, Pennsylvania will not be expounded upon further.

However, the second procedure utilized at Lancaster Municipal Airport was unique to this site and was employed as a direct result of the characteristics of the limestone formation at this site. The procedure of concrete grouting and installation of a filter drainage system was utilized in stabilization and treatment of a sinkhole collapse which occurred in the edge of a connector taxiway after a heavy rainfall caused runoff to pond in the excavation for the pavement box of the taxiway. During excavation of the sinkhole, the laminated character of the limestone formation prevented access to a solution cavity that was exposed by a slot approximately 0.75 to one meter (2.5 to three feet) in diameter. Examination of the cavity revealed that it extended approximately four to five meters (thirteen to sixteen feet) and was 2.4 to 3.7 meters (eight to twelve feet) wide. The cavity terminated at more closely spaced joints in limestone rock and was partially filled with very fine silt. Since the cavity could not be opened without blasting, which could have accelerated other minor sinkholes, the cavity was grouted with a slurry concrete. The sinkhole was then backfilled with an inverted filter drainage system to above the elevation of limestone rock and sealed with a highly plastic, well-compacted clay blanket one to 1.5 meters (three to five feet) in depth. A cross section of the sinkhole treatment is presented in Figure 4.

Since the large sinkhole identified at the Roanoke Regional Airport was in very close proximity to the alignment of the vehicular tunnel and the sinkhole was accepting large volumes of stormwater runoff without ponding of water, provisions were made for treatment and stabilization of this feature during construction. The original ground surface elevation in the sinkhole was approximately 341.5 meters (1120 feet) with the finished subgrade elevation for the floor slab of the tunnel at about elevation 339 meters (1112 feet). After excavation to near finished grade of the tunnel subgrade, the sinkhole was undercut or excavated to a total depth of 8.5 meters (28 feet) below finished subgrade elevation. Limestone boulders and weathered rock were encountered at a depth of about four to five meters (thirteen to sixteen feet) below finished subgrade elevation. The limestone formation at Roanoke Regional Airport is characterized by the formation of blocky to oval-shaped limestone boulders above the more closely spaced limestone units. Excavation of these boulders allowed access to the solution channel or drainage feature of the sinkhole. The sinkhole excavation was backfilled to above the limestone rock elevation with successive layers of rip-rap size shot rock choked with No. 2 stone, No. 4 stone, No. 57 stone, C-33 concrete sand and a clay blanket to seal the sinkhole area. A cross section of the sinkhole treatment at this project is presented in Figure 5.

Excavation of the sinkhole that developed beneath the 1.8 meter (72 inch) stormwater drainage system at the Valley View Mall revealed the presence of as many as four solution channels and joints which exhibited very active internal drainage. Once the sinkhole area was excavated, large volumes of water were used to expose and clean the limestone rock and identify the locations of solution channels. Since this sinkhole feature had developed with such activity of internal drainage, the inverted filter drainage system used to treat and stabilize this sinkhole was modified by the placement of a geotextile filter fabric between the contact of the limestone rock and filter drainage system. The geotextile filter fabric was placed in a loose condition to allow the initial layer of No. 4 stone to be compacted or forced into solution channels and joints. A cross section of the sinkhole stabilization and treatment at the Valley View Mall project is presented in Figure 6.

Sinkhole features exhibiting very active internal drainage, to include the old sinkhole used for drainage, at the Virginia Highlands Airport were excavated and stabilized by installation of an inverted filter drainage system. However, the limestone sinks which were observed to have very inactive internal drainage were excavated to remove topsoil and organic silts and backfilled with a well-compacted clay soil fill. This same procedure was used in the treatment and stabilization of limestone sinks at the Roanoke Regional Airport and the Valley View Mall projects. A cross section of the treatment procedure used for stabilization of the limestone sink features is presented in Figure 7.

SUMMARY AND CONCLUSIONS

The treatment and stabilization of karst features at these projects, coupled with previous experience with detection and treatment of sinkholes in the Ordovician to Cambrian Age limestones of east Tennessee and southwest Virginia have led to the formulation of the following conclusions.

1. Karst features can be effectively treated and stabilized by proper methods to provide foundation support on major engineering projects.

2. Installation of inverted filter drainage systems is considered to be the best method for treatment and stabilization of karst features exhibiting active internal drainage characteristics.

3. Although the installation of inverted filter drainage systems is considered to be the best treatment for karst features with active internal drainage, provisions must be made to allow alternation and adjustment of the treatment method once the karst feature has been excavated and the characteristics of the individual feature are evaluated by inspection.

4. On major engineering projects where karst features are present, the owner and contractor must be made aware of the potential for development of sinkholes during construction. Additionally, provisions must be made to

allow proper treatment and stabilization of karst features developed during construction.

5. The treatment and stabilization of karst features must be supervised by a geotechnical engineer or geologist experienced with limestone formations and karst feature development. Continuous inspection of the excavation of karst features often provides information needed for evaluation of the characteristics of the features and the corrective action required.

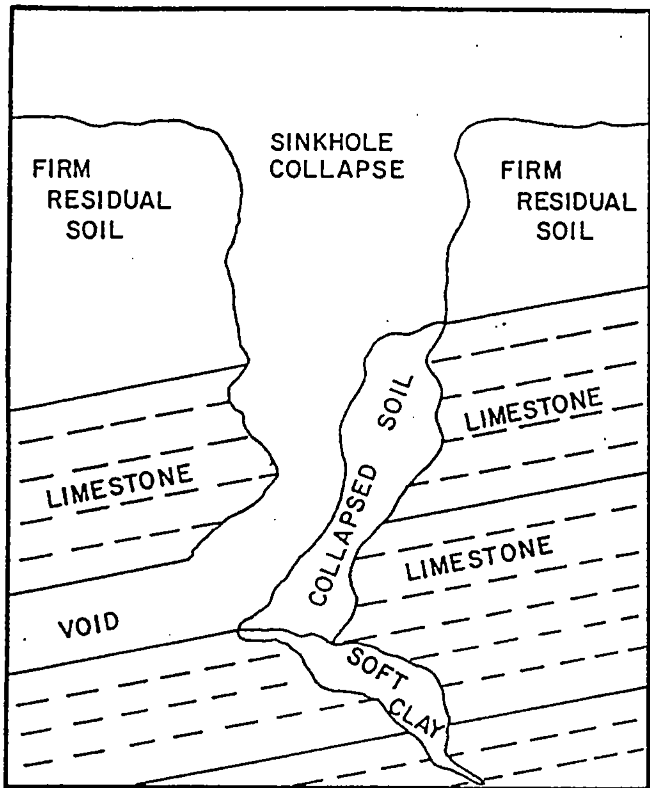

FIGURE 1 Subsurface Profile at Lancaster, PA.

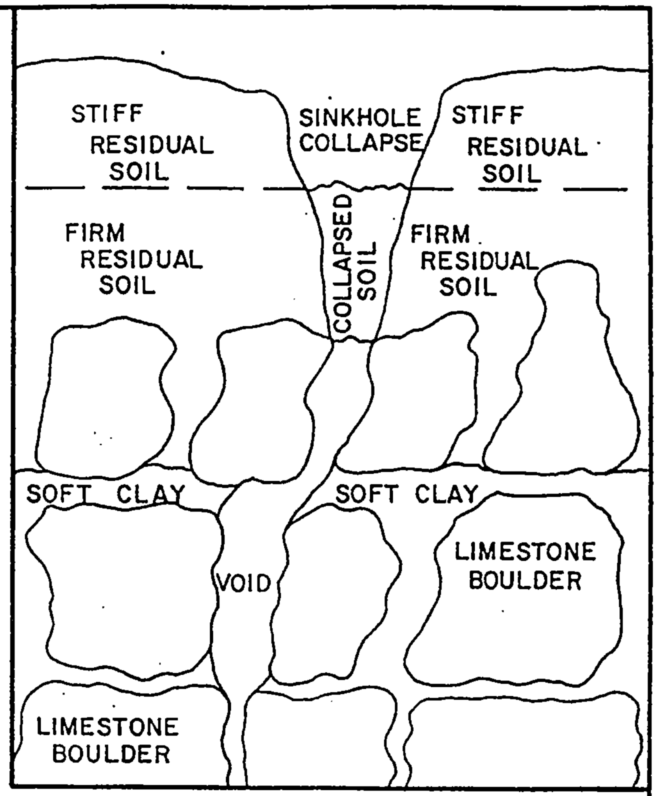

FIGURE 2 Subsurface Profile at Roanoke, VA.

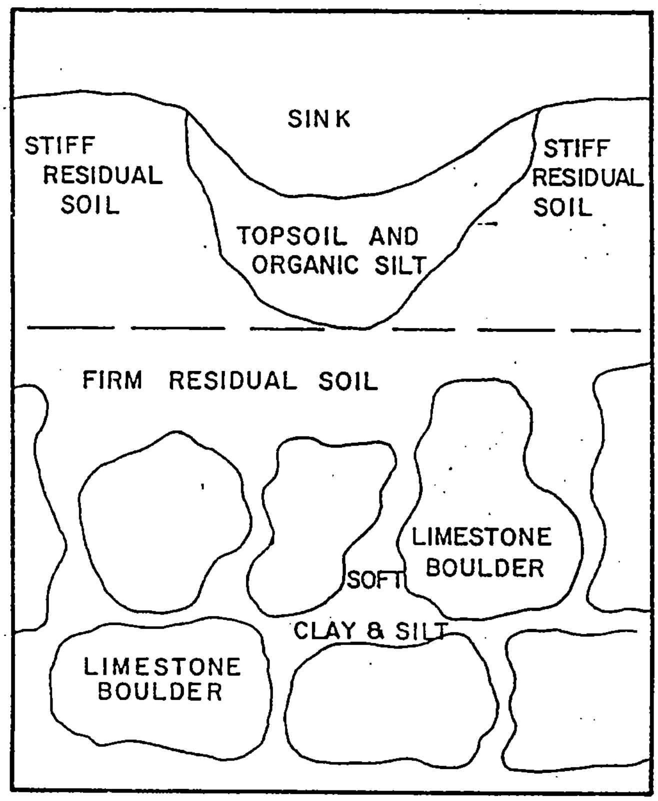

FIGURE 3 Subsurface Profile at Abingdon, VA.

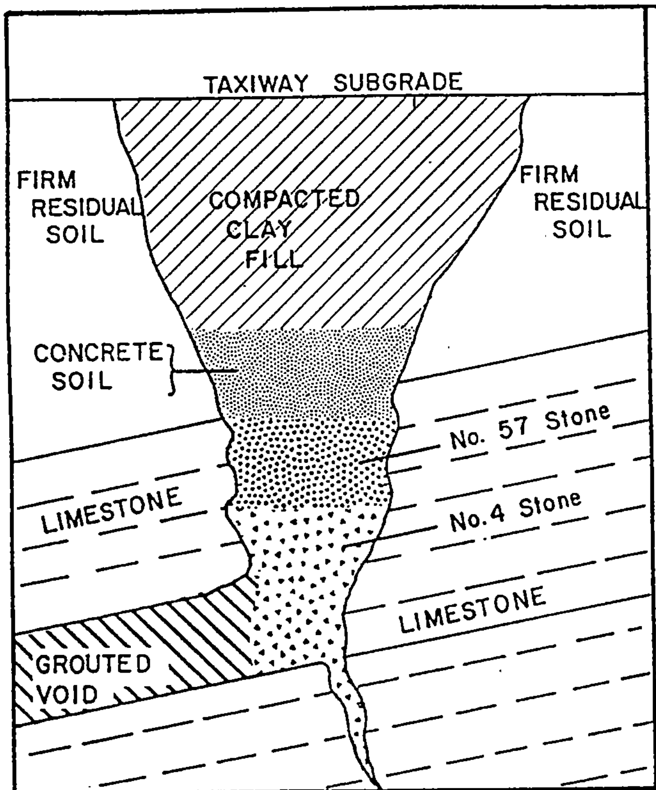

FIGURE 4 Sinkhole Treatment at Lancaster Municipal Airport (Grouting & Filter)

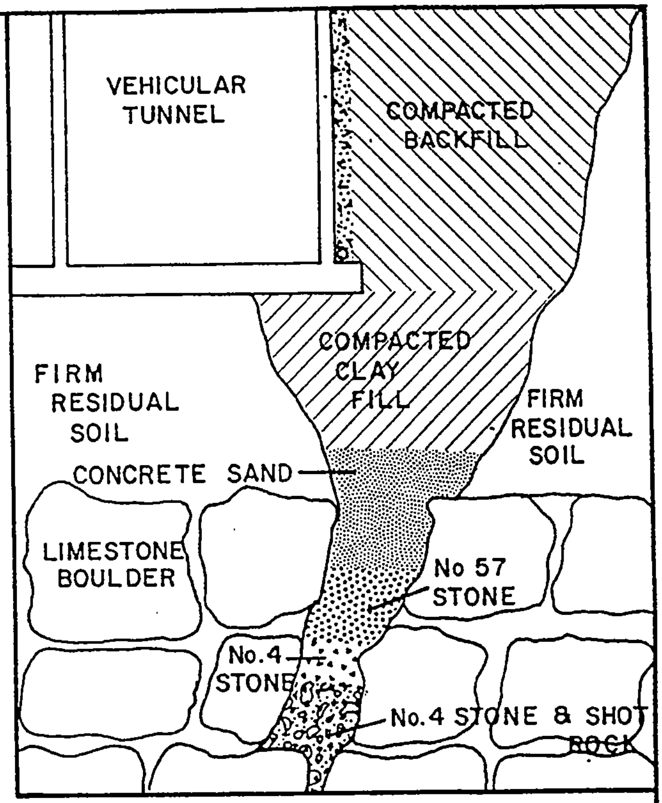

FIGURE 5 Filter Drainage Treatment of Sinkhole at Roanoke Regional Airport

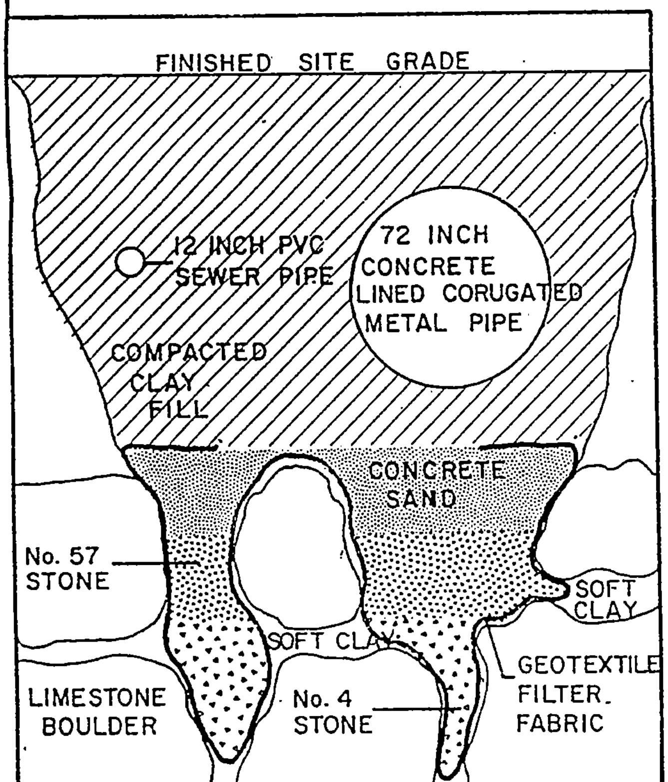

FIGURE 6 Filter Drainage Treatment of Sinkhole at Valley View Mall, Roanoke, Virginia

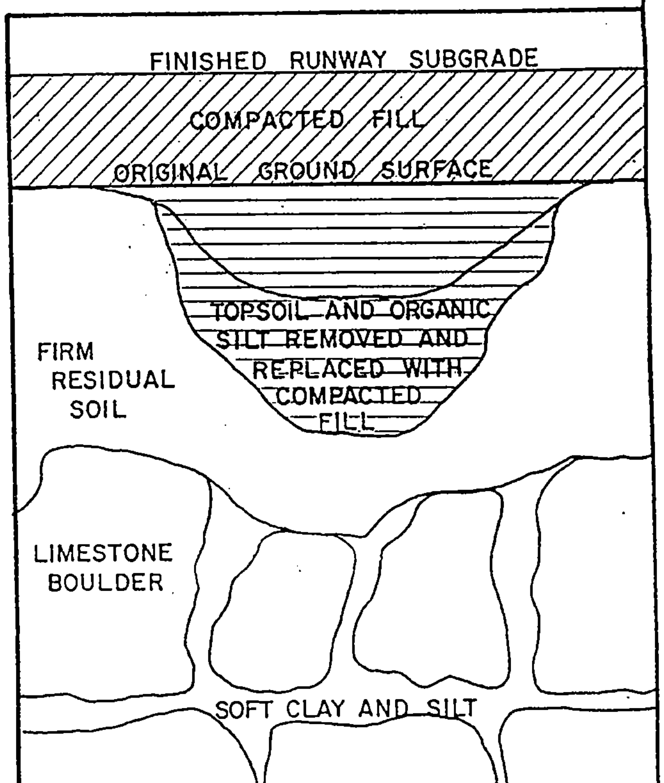

FIGURE 7 Limestone Sink Treatment at Virginia Highlands Airport

Motorways in Karst (Slovenia)

by

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Abstract

By preliminary speleo-karstological researches we overviewed the known caves located in the routing of the future motorway Divača-Kozina and also the superficial karst features, including several caves without roof. By such a preliminary inspection the control over the opening of new caves during the earthwork is easier as many caves may be predicted. In fact several new caves opened during the construction. These are either shafts draining the water from the surface underground or old caves that are either empty of filled up by sediments and flowstone. Newly discovered caves are very illustrative in terms of cavernosity and aquifer development. Karst studies showed that karst may be extremely permeable, the water may drain through 100 m thick limestones in an hour already. We analysed the chemical composition of water that flows off the roadway after every rainfall and we recorded organic pollution, lead, cadmium, zinc and copper and in winter a high level of chloride. This pollution due to traffic at normal conditions augments by accidental spills of harmful substances at eventual road accidents.

Introduction

The Karst Research Institute ZRC SAZU is in control of speleo-karstological researches during the motorway construction in Slovenia from the preliminary researches to construction itself when new, unknown caves are opening; the quality of water flowing off the roadways is monitored also.

Preliminary karstological researches in the laying out Divača - Kozina

By detailed speleological-geological researches at the scale 1:1000 we checked out 6,7 km long section of future motorway Divača-Kozina. The shafts are prevailing in the area of future motorway and nearby but superficial karst phenomena are also frequent.

By preliminary field researches we resurveyed all the known caves lying directly in routing or nearby. We also explored all the dolines and collapse dolines interpreting the karst loam fills. Altogether 15 karst caves were registered, 6 of them were already known in the Register of the Slovene Caves. But we must say that newly discovered caves are substantially shorter than already known caves. Average density of caves is 2,238 cave/1 km.? In the laying out of the future motorway and nearby we traced some deeper shafts (up to 130 m in depth) and shorter horizontal caves (the longest horizontal passage is in Jama nad Škrinjarico, 190 m long).

The entrance into Jama nad Škrinjarico (Cad. no. 1102) lies 35 m westwards from the laying out (1,8.1,84 km) of the motorway. The cave is 270 m long and 130 m deep. As a roadcut is foreseen just in this section of the motorway the thickness of the roof above will diminish from 95 to 85 m. The height of the horizontal passage below the road is from 5 to 10 m.

The cave developed in the limestones of the Liburnian formation (Pc). The above sea level altitude of the entrance is 482 m. In the list of natural heritage it figures as a natural monument (NS 882). The horizontal passage is richly decorated, on the walls there are old Italian signatures. The cave survey was already published in the book "Il Timavo" (Boegan 1938).

From the natural karstological point of view the cave is very important and must be preserved. Extreme care must be dedicated to the pollutants, like spilled oil from a construction site, and even later when traffic will circulate by the road. The main drainage direction is towards the springs of Timavo but a high probability exists that one part of water flows towards the Rižana or its wider background (Mihevc et al. 1994). As the motorway is foreseen directly above the horizontal passage of this cave the stability of the roadway must be assured above the empty space and later eventual changes in the cave must be monitored (for example breakdown due to considerable structural load).

Superficial karst phenomena can be divided into dolines, big dolines, collapse dolines, caves without roof, karren, grooves, kamenitzas and grikes. The bottoms of most of dolines in the laying out of the future motorway are filled by karst loam and this must be removed during the road construction. There are also some wilder dolines having breakdown boulders and poor karst soil at the bottom.

Close to Kozina the motorway passes between the dolines Petrovec and Lešnjak. Regarding the volume these are two biggest space anomalies within the laying out and they must be treated accordingly. The diameter of the upper lip is up to 170 m and of the bottom 90 m. The bottoms of dolines are filled by karst loam. A collapse doline is a superficial karst feature that rather reliable indicates fast transport of surface material into underground and later underground transport by water flow.

In the profile no. 142 (from 5,6 to 5,68 km) a collapse doline is located entirely in the laying out. Its upper dimension is 55 x 60 m across and at the bottom it is 30 x 20 m. Blocks of limestone had crushed from the steep walls, thus breakdown boulders and partly loam prevail at the bottom. As this doline lies in the middle of a future road it must be treated carefully.

Caves without roof (Šebela & Mihevc 1995) are old, mostly horizontal passages which remained inactive after the last infill by flysch and other sediments. The processes of uplifting and lowering of the terrain and in particular erosion and dissolution of karst caused that these caves remained roofless. The caves without roof may morphologically resemble to dolines filled with karst loam. Within the future motorway we discovered four roofless caves. When construction works will start numerous new caves and roofless caves will be discovered without doubt.

Typical superficial features, very frequent in the laying out of Divača-Kozina motorway are karren, grooves, solution basins and grikes. They indicate high limestone solution, grooves and grikes also high cavernosity favourable for vertical water infiltration. Along numerous grikes, up to 2 m deep, developed in fissure-broken zones, during the construction one may expect appearance of vertical shafts or corrosionally widened fissures. Speleo-karstological researches of the Divača-Kozina motorway revealed beside known karst phenomena numerous new ones.

Fissure and broken zones (Čar 1982) are extremely well permeable for vertical water percolation which consequently means that these zones are very favourable for karstification in horizontal and vertical direction. Along these zones the cavernosity is increased and mostly shafts may be expected as well as fast percolation of rainfall and pollutants from the road underground.

Newly discovered caves during the motorway Čebulovica-Fernetiči construction

In planning new motorways in Kras we tried to avoid important caves. Though it was impossible to avoid two smaller shafts and one old cave located in the laying out of the motorway Čebulovica-Fernetiči. They were blasted during the road cut works. Without doubt some other smaller caves were blasted also during extensive earthworks as flowstone and sediments remained mixed with blasted rubble at the surface.

Quite a lot of caves opened during removal of vegetation and soil from the karst surface and during digging the road cuts and tunnels. Some of new discovered old caves are empty, more of them are filled with sediments and rubble. These are the oldest traces of underground flow through the aquifer and therefore bearing an extreme karstological importance. Many old caves are roofless already due to lowering of karst surface. Frequently they look as impermeable corrosion notches on the rocky surface. Caves had developed in phreatic zone when the aquifer was still bounded by flysch and they were later, due to underground water table lowering, transformed by faster water flows and during floods filled up by sediments. In the periods when the water did not flow through the caves flowstone deposited. We may conclude that these sediments although found on the surface are cave sediments. A high cavernosity of this part of karst is indicated by numerous shafts through which the water flows from the surface underground. The shafts become visible when upper layers are removed; most of them were found at the bottom and in the slopes of dolines from where the soil was removed, or they even intersect old caves. Shafts are most frequently found in the Cretaceous limestone but also in Paleogene limestone and dolomite. The deepest newly discovered shaft was 110 m deep. Due to dispersed permeability over the karst surface most of the shafts are small and inaccessible; deeper below the surface the shafts are more spacious as water converges.

Empty old caves usually opened because blasted roof collapsed or they occurred in sides of road cuts (Annex 1). Smaller caves below the road were blasted and filled up, the caves in the sides of road cuts were walled as their rims were unstable. The walls of the caves crushed in a distance of 15 m from the blasting point. Nearby rocks are very crushed and disintegrated into rubble, further on the cave walls are fissured but stalactites mostly remained on the ceiling. The cave exploring was a very demanding task and in tunnels, where the caves opened in the ceiling, even impossible. A bigger cave in the laying out Dane-Fernetiči (Profile 1048) was constantly collapsing due to extremely crushed rocks. Above it the roadway must be specially strongly consolidated. Fissure-shaped shafts with thin roof were also blasted, filled up and often strengthened by a concrete plate. However, larger caves must be preserved. Thus a deep shaft was preserved below a concrete plate and a very thick fill. The water drainage through it remained through the rocks. We suggested to make an artificial entrance into the deepest shaft; a waste water collector is located just above it (Annex 2).

Due to high cavernosity of karst aquifers a constant warning against the roadway subsidence due to roof collapse is accentuated. Sinkholes that appeared during consolidating the rubble on the roadway by a machine roller confirmed the danger. Once again we suggest to examine the terrain by georadar. By such a method all spacious cavities close to the surface would be detected. The caves filled up by fine-grained sediments must be cleared completely. Later erosion of sediments may cause the subsidence of the roadway. Even below old caves younger shafts might exist.

High cavernosity is one of the most reliable factors of permeability and demands a special care during the construction but also during the use of a road. Only treated waters are allowed to flow over a karst surface. But karst must also be protected against accidental spills of harmful substances.

Composition of water that flows off the roadway and how it threatens karst waters

The karst is widespread in Slovenia, it covers 43%. The key to karst is the development of its unusual subsurface hydrology with predominantly vertical and underground drainage. The researches that lasted for two decades have already shown that any pollution on the surface is reflected deeper underground.

A part of pollution due to traffic deposits on nearby vegetation and soil and its penetration underground is slower. Pollutants that deposit on the roadway are washed by every rain and they flow either directly or by various collectors into karst. Water chooses the most permeable natural way or it flows by canals provided by the construction of road. After some time this permeability increases due to carbonate rocks dissolution.

The first analyses of water that flows off the roadway near Postojna showed the type of pollution and its extent (Kogovšek 1995a, 1995b). But we do not yet know how these pollutants affect karst deeper underground. Ecological awareness of people in Slovenia improves in particular if the essential living substance, water, is concerned. We must study in detail the extent and dynamics of pollution due to traffic and its impact on karst water as only good understanding of these problems provides efficient and suitable way of solving this kind of pollution and thus preserving good drinking water.

The results of water composition that flows off the motorway

On the observed section of motorway Ljubljana-Razdrto near Postojna we analysed water that flows off the roadway through an oil collector directly into karst; we took samples at different climatic conditions, at first rain after a long drought and after intensive rainfall. The first measurements showed essential variations of most of the parameters and this is why we wished to ascertain the composition of water during a single climatic event - during water pulse, when we sampled four times. In the Tables 1 and 2 are shown average, minimal and maximal values of single parameters of all the measurements and also those, observed during water pulses.

The highest variations demonstrated the levels of chloride due to intensive salting of roadway in the wintertime. Maximal recorded value is surprisingly high, almost 14 g/l. It seems that chlorides do not harm the growth of grass along the road but they damage the pine trees nearby where drying of branches may be seen. Average levels of chloride correspond to the measurements made in America and elsewhere in Europe (Ellis & Revitt 1991). But we recorded higher levels of lead which is probably due to only partial use of unleaded fuel in Slovenia. We also recorded higher variations of BOD_5 and only slightly lower average values. Much more bothering are levels of non-degradable organic pollution (COD) which is from 5 to 10 times higher compared to BOD. We also studied dependence of COD and BOD on turbidity; it shows that organic pollution mainly derives from solid particles on the roadway (Kogovšek 1995 c).

Table 1. Average, maximal and minimal values of measured parameters of all measures

	T	SEP	pH	Turbidity	COD	BOD	Chloride	Sulphate	Pb	Cd
	0C	µS/cm		FTU	mgO_2/l	mgO_2/l	mg/l	mg/l	µg/l	µg/l
AVERAGE	11	2504	7,9	158	136	19,5	938	35	1636	96
MAXIMUM	19	33100	8,8	780	480	84,0	13900	158	11100	250
MINIMUM	4,6	50	7,3	21	19	2,3	1	3	180	11

Table 2. Average, maximal and minimal values of measured parameters during water pulses

	T	SEC	pH	Turbidity	COD	BOD	Chloride	Sulphate	Pb	Cd
	0C	µS/cm		FTU	mgO_2/l	mgO_2/l	mg/l	mg/l	µg/l	µg/l
AVERAGE		1059		191	132	17	335	35,5	1671	105
MAXIMUM		7810		780	480	70	1980	258	11100	250
MINIMUM		50		23	19	4	2	3	180	11

The highest pollution was recorded after the initial rinsing of a roadway after a long, dry period; a high amount of accumulated solid impurities on the roadway was recorded. In the wintertime the runoff water demonstrates increased concentrations of chloride and sulphate levels and also specific electric conductivity compared to summer time when increased levels of metals (lead and cadmium) were mostly recorded. The degree of road pollution depends on density of traffic; the quantity of pollution flowing off the roadway might be calculated by contemporaneous measurements of quantity and composition. In the area of Postojna from 1600 to 1700 mm of precipitation falls annually, mostly in heavy showers meaning a huge amount of water in short time intervals.

We saw that if a lot of rain falls in one shower, the roadway is entirely washed and it shows a better quality of consecutive samples. It means that the highest amount of pollutants is washed at the beginning of rain. The figures 1 and 2 show that about 10 mm of rain wash a major part of pollution and it would be appropriate to treat this water. However the values of parameters even after good rinsing of roadway during intensive and abundant rainfall do not decrease below a certain value; this cannot be even expected as permanent traffic produces continuous pollution.

The water that flows off the roadway cannot be considered as a clear, meteoric water. It is notably true for permeable karst areas where many roads exist and new roads are under construction. Very sensitive are water catchment areas above the springs that are captured for water supply. Such case is Malni spring near Planina which is only 2 km far and 150 m lower than the motorway, local roads and railroad. Analysing heavy metal levels we recorded in average 2 mg/l of lead, up to 0,25 mg/l of cadmium and once 0,5 mg/l of zinc and 0,02 mg/l of copper. According to the Official Gazette of the Republic of Slovenia (1995) waste waters values permitted by the law, allowed to drain into a water flow where dilution is expected,

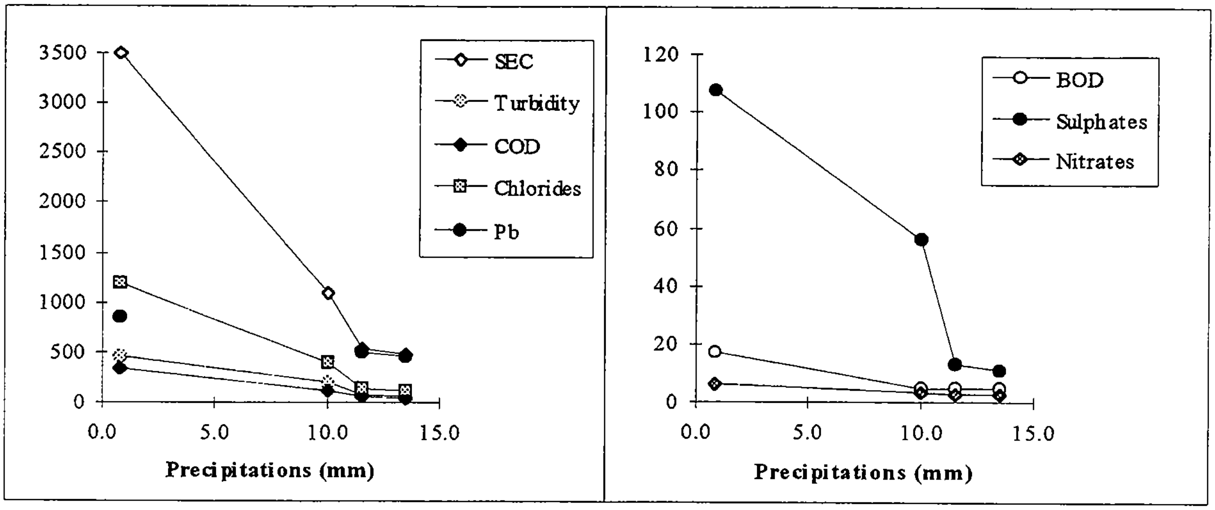

Fig. 1: Conductivity (μS/cm), turbidity (FTU), Chemial oxygen demand - COD (mgO_2/l) and lead(μg/l) in water flowing off the roadway in relation to rainfall quantity.

Fig.2: Biochemical oxygen demand - BOD_5 (mgO_2/l), sulphate (mg/l) and nitrate (mg/l) in water flowing off the roadway in relation to rainfall quantity.

may contain up to 0,5 mg/l of lead, copper and cadmium and up to 1 mg/l of zinc. If water flows directly into karst there is no dilution, and much lower limit values are planned to be implemented for karst regions. In our case we cannot overlook the fact that the same metals are present in the sediment of the Malni spring which is captured for water supply of Postojna commune inhabitants and high risk exists that their origin also lies in the polluted road at the surface. Maximal permitted value in drinking water is for lead 50μg/l and for cadmium 5 μg/l. Permeability of this region is very high; by detailed studies of vertical water percolation we stated that through 100 m thick carbonate rocks the water penetrates in one up to some hours already, depending on intensity of rainfall (Kogovšek 1995 b). It would be necessary to find out how are particular components of determined pollution widespread underground; such type of studies does not exist yet.

It is absolutely clear that these waters require, in particular if this is the case of captured springs, a special treatment. Detailed researches of quantity and type of pollution would allow detailed estimation of pollution which could serve as a basis for planned treatment of these waters. At the observed case we stated that in dry period single components in a solution contained in the oil collector were substantially enriched (chloride levels up to 7 times, sulphates 2 times). In the outflow from the collector compared to its inflow lower values were recorded related to turbidity and less degradable organic pollution (COD). The oil collector played in this case a role of a settling where degradable processes are present in smaller extent and this is already a useful fact at searching the way how to treat these waters. The second such fact is the composition of deposit as the analyses showed a waste that may be deposited at communal waste disposal sites (tight disposal sites with collecting and treatment of infiltrated waters).

Conclusion

In karst, in particular in the catchment areas of springs captured for water supply it would be helpful to find out how the most risky components of pollution due to traffic, this is heavy degradable organic pollution, lead, cadmium, copper and zinc travel underground; till now such studies were not yet done in the Slovene karst. Monitoring the quality of karst springs in Slovenia we established that they are in a larger extent already polluted by heavy minerals. Such knowledge would help solving pollution at normal conditions but also in case of eventual spills of harmful substances which may occur during road accidents; such a case we had when in October 1993 near Kozina 18 t of mineral oils was spilt (Knez et al. 1994) and a year later when 16 m^3 of gas oil was spilt near Obrov polluting Rižana, the most important water source of Littoral (Kogovšek 1995 c).

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Computer Analysis of Required Statistical Parameters for Sites with Multiple Monitoring Wells Measured for Extended Periods of Time

by

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Abstract

Environmental regulations for solid and hazardous waste facilities and other construction sites require comprehensive, long-term groundwater monitoring. Rigorous programs are used to determine the impact of disposal sites, landfills and embankments on groundwater quality. Many monitored parameters are naturally occurring, so a sophisticated statistical analysis is needed to accomplish sound evaluation of down-gradient effects. Highway embankments today are sometimes constructed from waste materials or they may contain acid forming soils and rocks from on-site borrow areas.

Depending on state requirements, up to 64 naturally occurring constituents may require statistical analysis. Data are also collected extensively on volatile organic compounds and sometimes semi-volatile organic compounds. Testing may be semi-annually for each constituent, or perhaps quarterly, during the active life of a facility, followed by up to a 30 year post-closure period.

Statistical analysis of large volumes of data cannot be done manually. Many statistical methods for data analysis are highly complex, involving extensive calculations. Selection of proper statistical procedures is also difficult. Although statistical procedures are outlined in two USEPA guidance documents, commonly this work falls outside the capability of many environmental professionals. Users have also claimed that USEPA guidance documents are unclear, containing inconsistencies in the recommended methods.

Currently we are developing a computer procedure to track and statistically analyze groundwater data from various facilities including disposal sites and special embankments. This will allow environmental professionals to perform easily and quickly the required statistical data analysis. Because of the widespread availability of Microsoft Windows, it will be used along with many of its user-friendly features. The procedure will emphasize ease-of-use, yet have a sufficiently comprehensive set of features to accommodate groundwater data from typical, monitoring situations. The complete set of statistical procedures as outlined in the USEPA statistical analysis guidance documents will be considered, focusing on easy data entry and management, as wells as proper statistical analysis.

The application will also allow for easy data reporting; based on analyte, sampling date, or well designation. Because of its increased popularity, the GIS system data files will be included, to allow linkage to programs that accommodate extensive querying and the visualization of groundwater data.

Introduction

Environmental regulations for solid and hazardous waste facilities and other construction sites require comprehensive, long-term groundwater monitoring. Rigorous sampling and analysis programs are used to determine the impact of disposal sites, landfills and embankments on groundwater quality. For non-naturally occurring chemical parameters, such as volatile organic compounds (VOCs), the presence of the parameter in down-gradient wells and the absence in up-gradient wells is often sufficient to show that the source of the contamination is the facility. Other monitored chemical parameters, such as metals, are naturally occurring and may appear in both up-gradient and down-gradient wells even if there has not been a release from the monitored facility. Such contamination may not always appear at consistently elevated levels in down-gradient wells. Therefore complex statistical procedures may be required to determine if down-gradient levels are statistically significant.

A large amount of data can be generated for such a monitoring program. Depending on state requirements, facilities may be required to monitor for up to 64 chemical parameters. A minimum of one up-gradient and three down-gradient monitoring wells are required, although sites with over thirty total monitoring wells are not uncommon. Sampling is usually conducted semi-annually for the life of the facility plus a thirty year post-closure period. If contamination is detected, periods of quarterly monitoring for an extended list of parameters, including semi-volatile organic compounds, may be required.

USEPA statistical analysis guidance documents (USEPA 1989, 1992) detail procedures for the statistical analysis of groundwater monitoring data. To compare down-gradient water quality to background, recommended analyses include parametric and non-parametric analysis of variance (ANOVA) procedures, prediction limits, and tolerance limits. It may also be useful to monitor trends in a single well to determine if concentrations are increasing. Prediction limits and Shewhart-CUSUM control charts may be appropriate for such analyses. Also recommended by USEPA (1989, 1992) are procedures for the graphical display of data, testing for normality of data, and testing for equal variance between wells.

To simplify the statistical analysis of such a large amount of data, we are currently developing a specialized statistical application for personal computers. Although many other statistical analysis programs such as SAS™, SPSS™, Statistica™, or MiniTab™ are available, they may not be suitable for the unique situations and specialized statistical requirements of evaluating environmental monitoring data. Such programs also may require extensive and specialized statistical training for proper use.

Our software will be developed for the popular Microsoft Windows 95 operating system and include many of the user-friendly features common to such programs. The program will also include the most popular graphs and statistical methods recommended by USEPA (1989, 1992). Specialized features for environmental data analysis, such as unlimited parameter name length, and options for the representation of non-detects will be included.

The purpose of developing the program is to allow for fast and accurate analysis of data by the different recommended statistical methods. The primary focus of the research is to determine how the results of the various statistical methods vary under different monitoring situations. Statistical methods will be used on groundwater monitoring data from more than 30 actual hazardous waste facilities in Ohio.

Research Objective

The objective of this study is to evaluate USEPA recommended statistical procedures for groundwater monitoring data at hazardous waste facilities. To aide in the investigation, ChemStat™, a computer program for the statistical analysis of groundwater monitoring data is being developed. Areas of potential investigation include; determination of the best statistical methods to minimize or eliminate the effects of natural spatial variability in groundwater monitoring design; determination of the best method of representation of non-detect values; determination of ways to reduce the cost of groundwater monitoring while still protecting human health and the environment; and evaluation of the application of the USEPA recommended procedures to a variety of actual monitoring situations.

To aid in the research, a computer program will be developed to automate the statistical analysis of groundwater monitoring data. The program will include many USEPA recommended statistical procedures, and be flexible enough to apply them to a variety of monitoring situations.

Program Design

The program uses "Calculation-at-once" design, a method whereby most of the statistical calculations are performed when the data are entered into the program. The results of the calculations are then stored in memory until needed. When the user selects a statistical analysis method, much of the computational work will have been already performed, and the results can be displayed quickly. Advantages involve the fast display of results. Also, since calculations are performed at only one place in the computer program, it is easier to ensure that all calculations are correct. Any calculation that is incorrect will appear at many places throughout the program and is more likely to be detected. The disadvantage is a significant increase in memory usage needed to store the calculation results. It is also more difficult to perform customized calculations based on user preferences because any change in preference requires that all calculations must be re-done.

Data are input into the program from specially formatted ASCII text files. A converter is available to import data from other popular data management programs such as USEPA's GRITS/STAT, or created in spreadsheet applications such as Microsoft Excel™ or Lotus1-2-3™.

Statistical Methods

A complete analysis of groundwater monitoring data requires a series of computations. Figure 1 outlines a flowchart for the selection of statistical methods based on characteristics of the data set.

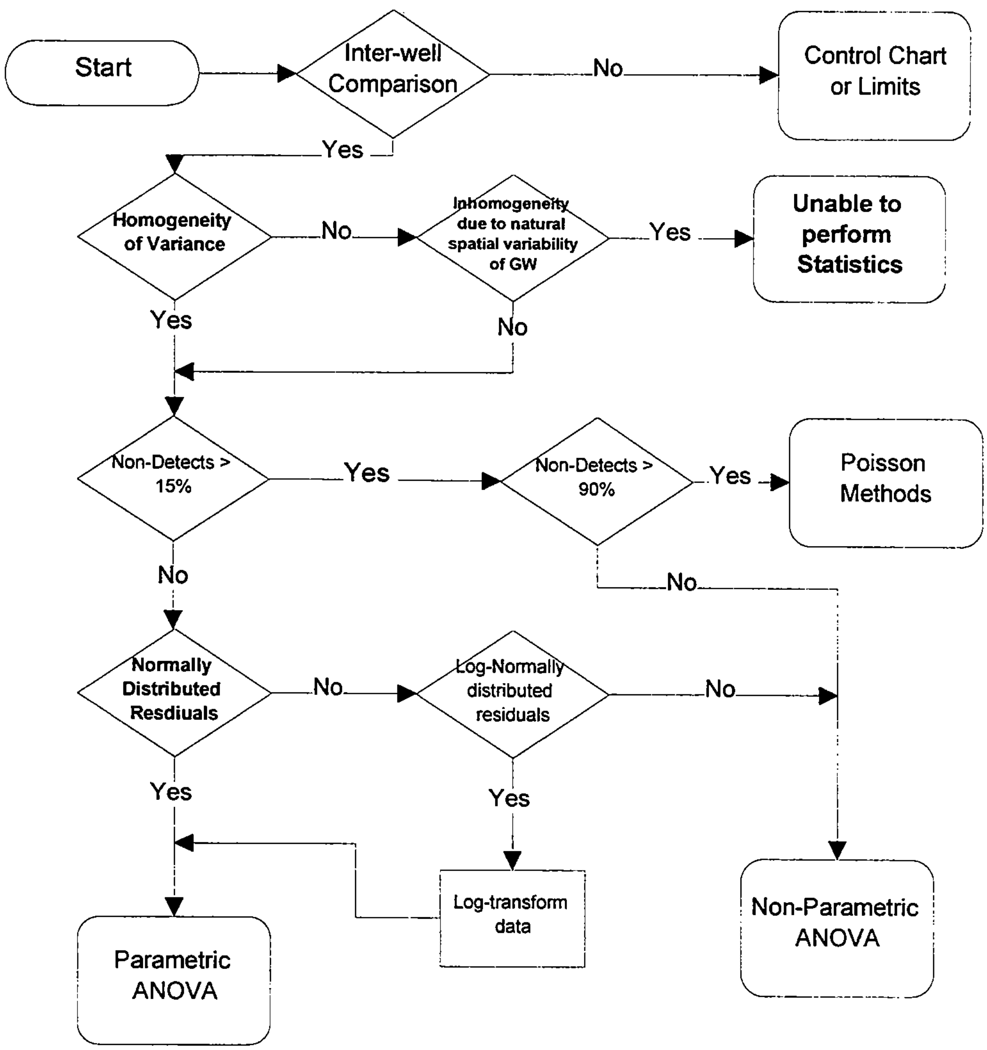

Figure 1. Flowchart for selection of statistical methods.

Because of data distribution, data set size, or subsurface geology, some analysis methods may not be suitable for a particular parameter. To be useful in most monitoring scenarios, the program includes statistical methods to test for normal distribution of data or the residuals, test for equal variance between the wells, and display informative presentation quality graphs. To determine if there is a statistically significant increase in contamination, parametric, non-parametric, and Poisson methods for large percentages of non-detects are included.

The standard method of detecting contamination is by inter-well comparison whereby the concentrations of a parameter in down-gradient wells are compared to concentrations of the same parameter in up-gradient wells. However such a comparison may not be possible if down-gradient and up-gradient wells monitor hydrogeologically isolated zones, or may be undesirable if extensive natural variability would invalidate the statistics. In such a situation, an intra-well comparison is required where for a single well, recently collected data are compared statistically to historical data to determine if levels of the chemical parameter are increasing. The program will include statistical methods to perform intra-well comparisons including prediction limits and control charts.

Normality Tests

Normal distribution of the data or the residuals (the difference between the observed value and the mean for the well) is a requirement for some parametric statistical methods. ChemStat includes several tests for normality. The most common normality test is the Shapiro-Wilks test. However this test is not suitable for more than 50 values. A modified version of this test is the Shapiro-Francia test suitable for data sets with more than 50 values. Another test for normality includes D'Agostino's test that is presented by Gibbons (1994), and is suitable for data sets with up to 1000 values. Also presented by USEPA (1989) is the coefficient of variation method. Although its use is discouraged by USEPA (1992), the method is included here because it is very fast, easy to implement, and suitable for an unlimited data set size. ChemStat also includes the probability plot, a popular graphical method to test for normality. Any normality test in ChemStat can be performed either on the data, or on the residuals of the data.

Equal Variance Tests

Homogeneity of variance, or "Equal Variance" between distributions is a requirement for most parametric tests. If the wells have equal variance, this indicates that data for each well follows the same distribution.

The Box plot or Box/Whiskers plot is a graphical method to test for equal variance and interpretation is therefore somewhat subjective. Bartlett's test and Levene's test are analytical methods and are therefore more objective. If data from the wells do not have equal variance, non-parametric tests may be required.

Graphs

The Shewhart-CUSUM (Cumulative Sum) control chart is a graphical method recommended for intra-well comparisons. The method has the advantage of providing a visual representation of changes in concentrations over time.

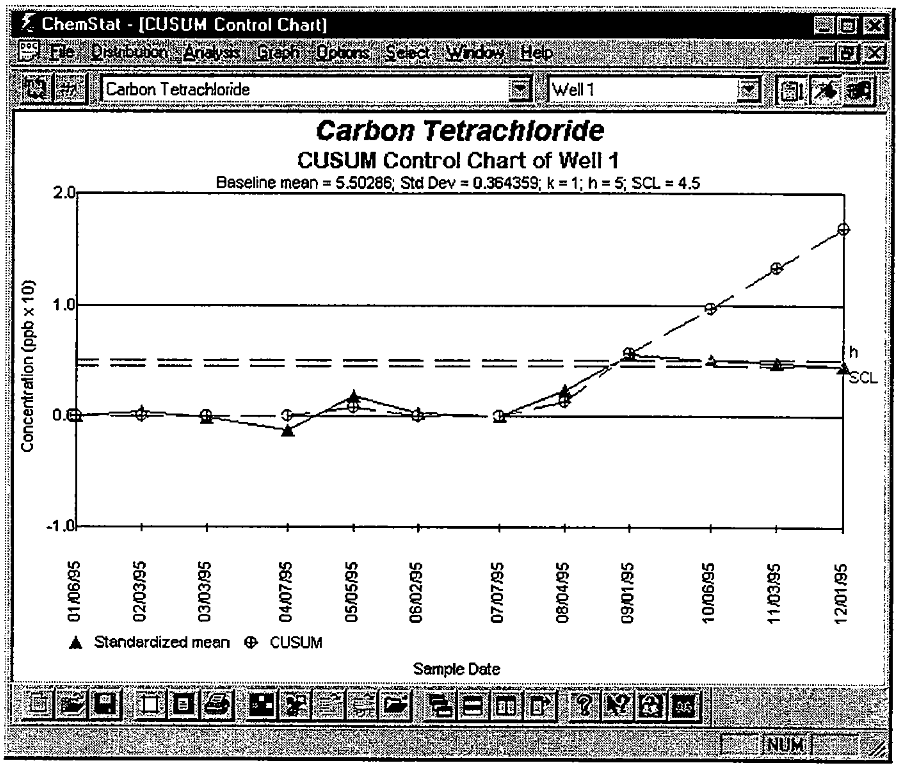

Figure 2. Shewhart-CUSUM control charts are used for intra-well comparison to track changes in concentration over time.

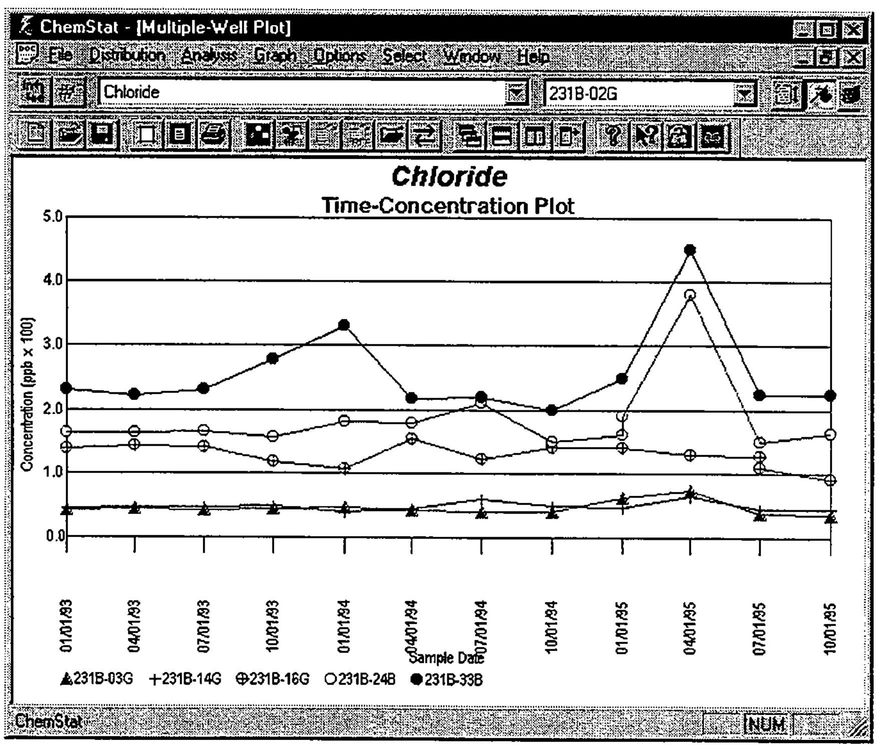

Figure 3. Concentration vs. time graphs comparing concentrations of multiple wells.

In addition to control charts, concentration vs. time graphs can be used for trend analysis of contaminant levels. ChemStat includes capabilities to graph multiple well concentrations over time. The time axis is scaled proportional to the time between each sampling date. A variety of symbol choices are available to distinguish the individual wells.

Parametric Analyses

Parametric analysis methods are the most commonly recommended analysis methods. These include parametric one-way ANOVA (analysis of variance), parametric prediction limits, and parametric tolerance limits. Parametric methods require that the data are normally distributed with equal variance between wells. If the data are not normally distributed, parametric methods may be applied to the natural logarithm of the data if the log of the data is normally distributed.

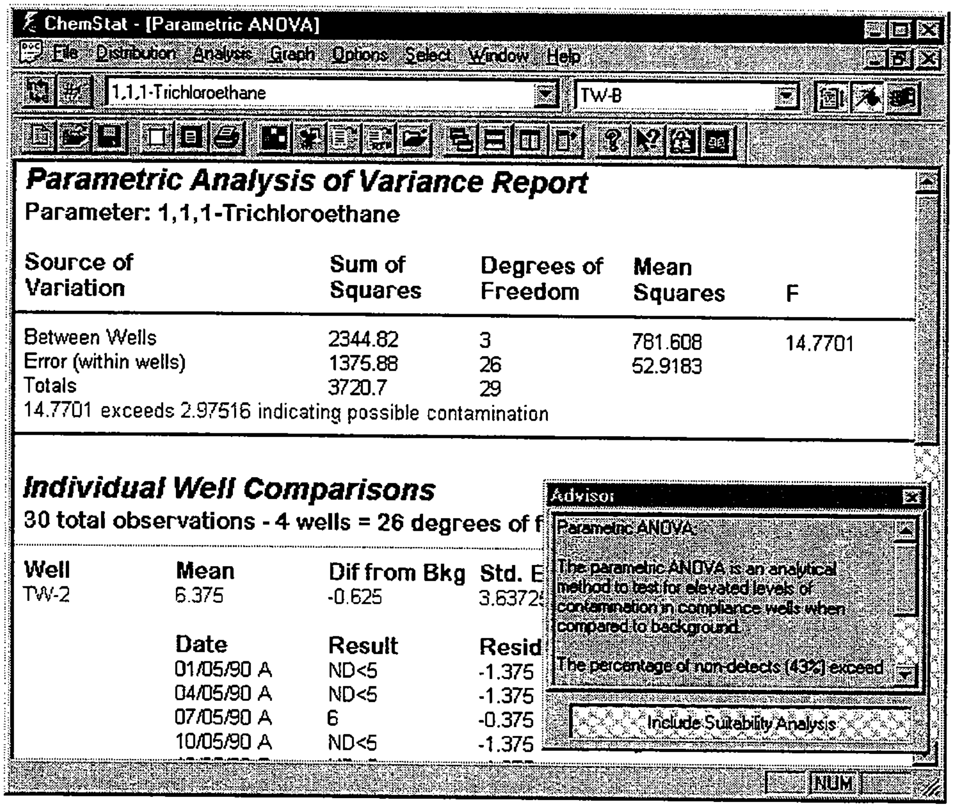

Figure 4. The parametric one-way ANOVA is the most commonly used analysis method. ChemStat provides the complete ANOVA table as well as individual well comparisons.

ANOVA methods are typically used to compare the pooled concentration of down-gradient wells to the pooled concentration of background wells. If the test indicates that down-gradient wells are statistically elevated as a group, the Bonferroni t-test can be used to determine which specific wells are statistically elevated.

Prediction limits can be used to both intra-well comparisons (comparing a single well's recent data to its own historical values) and inter-well comparisons (comparing down-gradient wells to up-gradient wells). Intra-well comparisons can eliminate the effects of natural spatial variability from the analysis. However intra-well comparisons require a series of historical baseline samples known not to be impacted by the facility. Inter-well comparisons involving prediction limits or tolerance limits are useful for situations where there are a limited number of samples from down-gradient wells.

Non-Parametric Analysis Methods

Non-parametric methods include non-parametric ANOVA methods, non-parametric prediction limits, non-parametric tolerance limits, and Poisson methods. Non-parametric methods are required if data are not normally distributed or do not exhibit equal variance between wells. They are also recommended for data sets with a high percentage of non-detects (> 25%) as would be common with organic compounds at contamination levels near the detection limit as well as heavy metals such as chromium, nickel, and selenium.

Non-parametric ANOVA methods such as the Kruskal-Wallis method and the Wilcoxin Rank-Sum method, usually involve ranking the concentration values from lowest to highest. ANOVA analyses are then performed on the ranks. Concern regarding the use of the non-parametric ANOVA is that the magnitude of the detections is not considered.

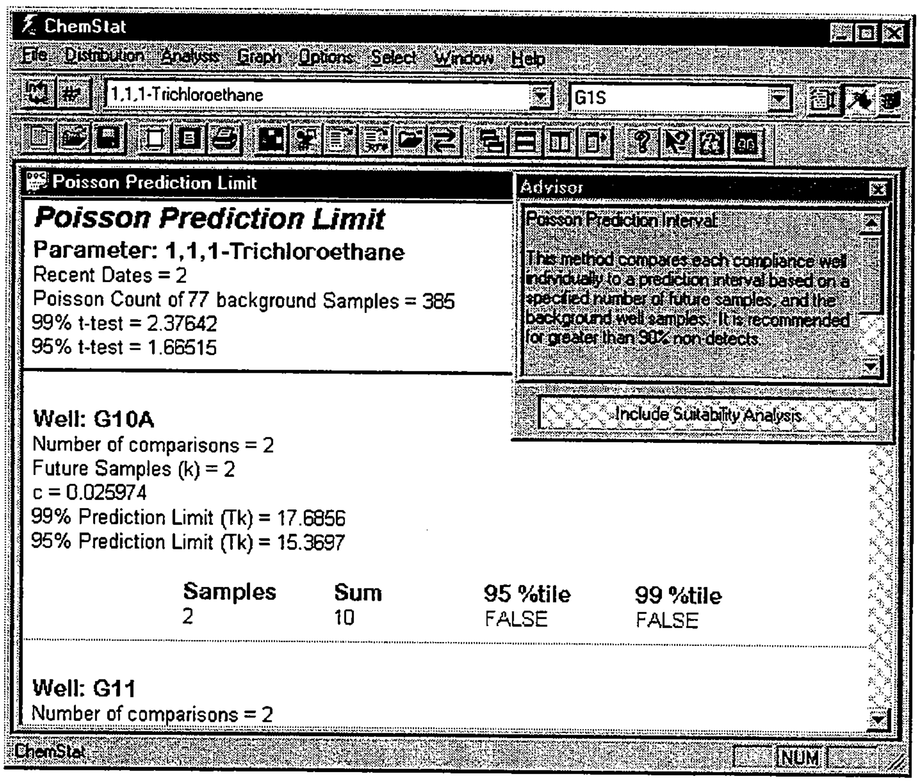

Figure 5. The Poisson Prediction Limit is a non-parametric method recommended for data sets with greater than 90% non-detects. The "Advisor Window" provides information about each test and makes recommendations for the suitability of the selected test regarding the current data set.

Non-parametric prediction limits and non-parametric tolerance limits usually consist of comparisons of down-gradient concentrations to the maximum background concentration. The power (or level of significance) of the test is then calculated. The disadvantage of these methods is that many samples (about 20) may be required to reach the desired 95% level of significance.

Poisson methods include parametric prediction limits and parametric tolerance limits. Poisson methods are recommended by the USEPA for data sets with greater than 90% non-detects. Poisson methods for groundwater statistics were developed by Gibbons (1987), but since that time, no further studies have been conducted regarding their effectiveness.

Program Verification

With any computer program that performs calculations, there is always a serious concern about the inclusion of incorrect calculations. For our application, sources of potential calculation errors can be divided into three primary categories.

1. Calculation errors occur when the calculation is programmed incorrectly. For example, multiplying when one should have divided.

2. Display errors are errors that occur when the value displayed is not what was intended. For example displaying the average value in the category labeled for standard deviation.

3. Interpretation errors occur as a result of incorrectly translating mathematical equations into computer code.

To help verify the program, calculations will be performed on examples provided with USEPA guidance, and the results compared to those in the guidance document. The disadvantage of this method is that USEPA examples are usually idealized situations, and the program may not behave as intended if actual field data are considered.

Verification using hand calculations, or by applying other computer packages will also be performed. This works well to ensure that values such as sample means, standard deviations, and percentiles are correctly calculated. However it is not effective in guarding against interpretive errors.

Program Usability

The program will include features to accommodate the requirements of real-world data.

Because much attention has been given to the inability of many computer applications to correctly represent dates past 1999, the program will be designed to properly represent dates including the year 2000 and beyond.

Many sampling events involve the collection of duplicate and replicate samples for a single date, and many statistical methods (i.e., control charts) are designed to include multiple samples. To accomplish this, the program will accommodate multiple samples for a single date. Additionally, the user will have the option of averaging all values with the same date, and performing statistics on the average values.

The program will have the ability to advise the user on the suitability of a selected statistical method in regard to the current data set, providing a warning if insufficient samples, or excessive non-detects would invalidate the selected method

To enable evaluation of various statistical scenarios, any statistical method can be performed on the original data or the natural logarithm of the data. Also, non-detect values can be represented by the detection limit, 1/2 detection limit, 1, or 0.

Summary

Currently we are developing a computer procedure to track and statistically analyze groundwater data from various facilities including disposal sites and special embankments. This will allow environmental professionals to perform easily and quickly the required statistical data analysis. Because of the widespread availability of Microsoft Windows, it will be used along with many of its user-friendly features. The procedure will emphasize ease-of-use, yet have a sufficiently comprehensive set of features to accommodate groundwater data from typical, monitoring situations. The complete set of statistical procedures as outlined in the USEPA statistical analysis guidance documents will be considered, focusing on easy data entry and management, as well as proper statistical analysis.

The application will also allow for easy data reporting; based on analyte, sampling date, or well designation. Because of its increased popularity, the GIS system data files will be included, to allow linkage to programs that accommodate extensive querying and the visualization of groundwater data.

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BUILDING EMBANKMENTS OF COAL COMBUSTION FLY ASH-BOTTOM ASH MIXTURES

By

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ABSTRACT

Coal combustion by-products of fly ash and bottom ash may be disposed separately, or may be commingled for disposal. Markets for any product, except high calcium Class C fly ash, are extremely limited. However, the other fly ashes and the bottom ashes have been demonstrated to be excellent embankment materials. The research reported in this paper makes the economic use of these presently wasted materials more technically feasible.

Where waste fly ashes and bottom ashes are disposed separately, there is a need to know the geotechnical behavior of explicit mixtures of them. Mixtures of optimal performance are thus identified and recommended. Where the products are commingled for disposal, it usual to find that the fly/bottom mixture varies widely within the deposit. Thus, materials excavated for use are implicit mixtures, whose composition must be determined, and parameters measured. Compaction control of these mixtures becomes an issue because of their variability.

The paper affords guidelines for the selection of fly/bottom ash mixtures, as well as the technology for controlling their compaction and predicting their behavior.

INTRODUCTION

Coal burning power plants in the U.S. produce enormous amounts of coal combustion by-products (CCBP) in the form of fly ash and bottom ash or slag. If there is no market for these materials, they are treated as solid wastes and are safely disposed. The bottom materials are sandy-gravely in size, while the fly ashes are of silt and clay size. They are collected separately and may be disposed separately. On the other hand, they are often disposed hydraulically in a pond, which then has varying amounts of the two materials, both areally and with depth.

Disposal costs are borne by the generators, but these are passed along to consumers as a part of the charge for services rendered, e.g., cost of electrical power. Alternates to disposal are desirable for a number of reasons: (1) reduced costs; (2) reduced disposal space; (3) substitution of these materials for natural materials; and (4) adherence to the positive principles of reuse/recycle.

A number of geotechnical/material uses of CCBP bottom and fly ash are possible. The one of emphasis in this paper is as a soil substitute in transportation embankments. Both environmental and geotechnical concerns must be addressed prior to routine use of these materials in embankments. In the environmental category are: possible contamination by leachates, dust, and potential erosion. Limits are placed on the heavy metal concentrations in the leachates. In addition, soil encasement is used to reduce leachate quantities from the ash fill and to control erosion. Geotechnical concerns center on the potential variability of co-mingled ashes, appropriate compaction control, and prediction of shear strength.

MATERIALS STUDIED

Samples of CCBPs were gathered for experimental study from two power plants. Both plants were producing Class F fly ash and bottom ash during this period. However, Plant A disposed these ashes separately, while Plant B hydraulically combined disposal in a pond. Sampling of the former was straightforward; however, sampling for the combined material had to be accomplished at a number of locations. The sampling sites were selected to allow a range of textures present in the pond area to be obtained. Samples were examined visually and microscopically and were character-terized as to grain size and specific gravity. Fly ash and bottom ash components were separated, as necessary, and were

recombined in prescribed proportions, as described in Karim, Salgado, and Lovell (1997), prior to study of compaction and shear strength characteristics.

Material from Plant A was combined at fly ash contents (F_1%), of 0 to 100%, as shown in Table 1. These are termed "explicit" mixtures since separately disposed materials were combined. Mixtures from Plant B exist at percentages finer than the #200 sieve (F_2%) of roughly 20% to 75%, as shown in Table 2. These are termed "implicit" mixtures, since the percentage is implicitly determined by the hydraulic mixing of the two components in the disposal process and varies with location in the disposal area.

COMPACTION

Relationships defined by standard compaction effort for Plant A are shown in Figure 1, where ($F_1 = 0\%$) represents all bottom ash and ($F_1 = 100\%$) is for all fly ash. As fly ash content is increased from 0% to 10% to 25%, the maximum dry unit weight increases and the optimum moisture content decreases. However, further increase in fly ash content to 50% and then to 75% produces a decrease in maximum unit weight and increase in optimum moisture content.

For Plant B, $F_2 = 22\%$ to 74%, the maximum unit weight decreases and optimum moisture content decreases as F_2 increases. See Figure 2. Data from both sources indicate that the optimum % fly ash for maximum unit weight is a relatively small value (about 25%). Interestingly, the availability of these products is roughly the reverse of the above, viz., about 80% fly and 20% bottom ashes.

The difference in compaction curves for Plant B (Figure 2) shows how important unrecognized variability in the disposed deposit might be. Unless the correct target maximum unit weight and optimum moisture content range is identified, difficulties in achieving the compaction specification and in correctly predicting the compacted properties will result.

Table 1. Composition of Explicit Mixtures for Compaction (Plant A)

Explicit Mixture	Fly Ash Content F_1 %
CA1	0.0
CA2	10.0
CA3	25.0
CA4	50.0
CA5	75.0
CA6	100.

Table 2. Composition of Implicit Mixtures for Compaction (Plant B)

Implicit Mixture	Passing #200 F_2 %	Sampling Location From Plant B
CB1	22	Location 1
CB2	53	Location 2
CB3	48	Location 3
CB4	74	Location 4

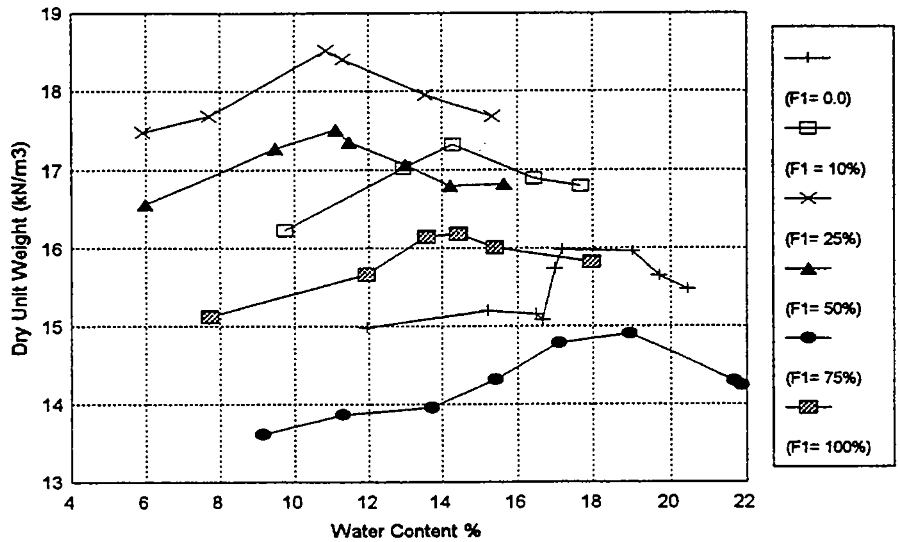


Figure 1. Compaction Curves for Fly/Bottom Mixtures. Plant A.

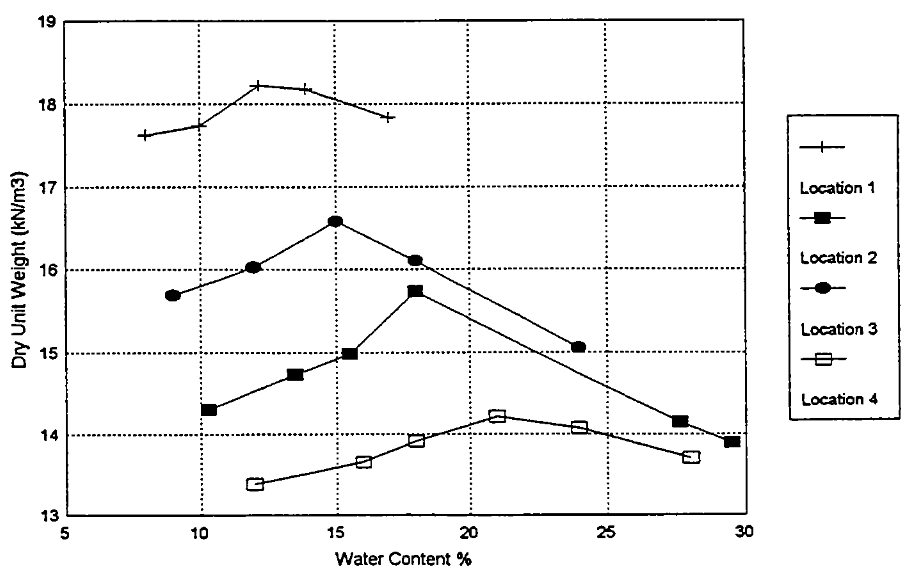


Figure 2. Compaction Curves for Fly/Bottom Mixtures. Plant B.

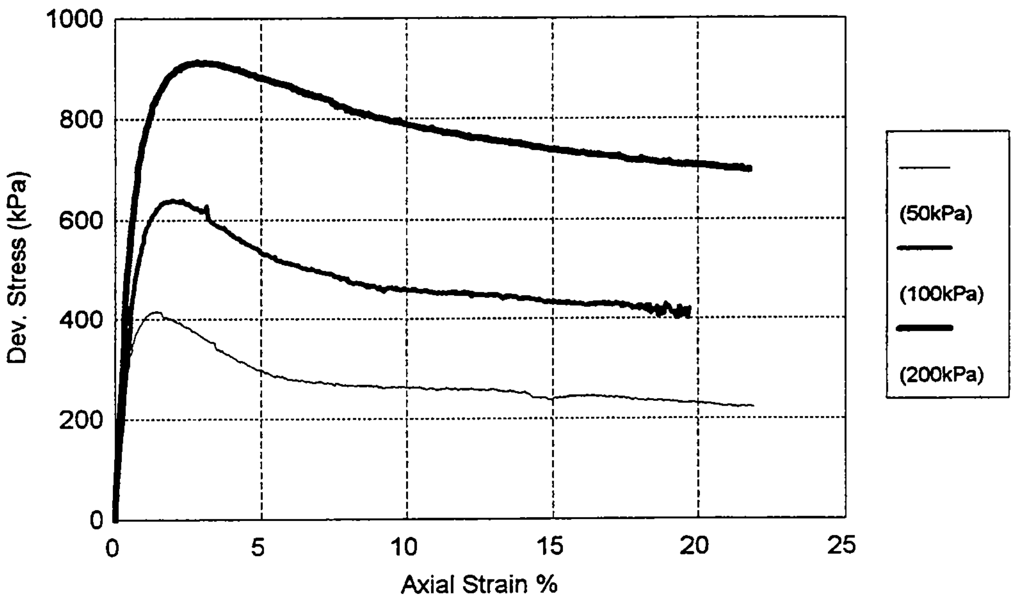
SHEAR STRENGTH

Compacted ash samples may in time become saturated in place and either compress or swell under the confinement effected by their position in the embankment. The procedure elected for prediction of shear strength parameters was triaxial testing with isotropically consolidated drained (CID) conditions. Two compaction levels were tested (90% and 95% of maximum unit weight at standard compactive effort), and three confining pressures were used. Details of the testing, including back pressure saturation, are given in Karim, Salgado, and Lovell (1997). Volume change was also measured. Bottom ash is texturally a sand, and its shear behavior can be qualitatively predicted from those of natural sands. As shown in Figure 3, the deviatoric stress increases to a peak and then decreases. At very small axial strains, there is volumetric contraction, followed by expansion (dilatation). A constant volume is approached at large strains. Peak stress increases with confinement, while dilatation decreases with increased confining pressure. As shown in Figure 4, the peak value of stress decreases with reduced level of compaction (R). Similarly, dilatation is decreased as R value decreases and may even become negative (contraction).

The value of the angle of shearing resistance at peak stress (Φ'_{max}) varies with the state of compaction (R), the amount of fly ash, and the confining pressure. As percent fly ash increases, (Φ'_{max}) decreases at first, then becomes little affected at higher values. While shearing resistance may vary significantly for common ranges of the above variables, the (Φ'_{max}) values are commonly above a value of 30°. Therefore, shearing resistance is unlikely to a controlling factor in building ash embankments.

APPLICATIONS TO HIGHWAY EMBANKMENTS

The optimum mix of ash components for both unit weight and strength would seem to be approximately 20% fly ash and 80% bottom ash. Where materials are separately disposed, they can be mixed in a proportion to take advantage of such an optimal condition. Since the byproducts are produced in approximately an inverse ratio (80% fly - 20% bottom), it may be desirable to use higher percentages of fly ash. Laboratory testing will reveal what



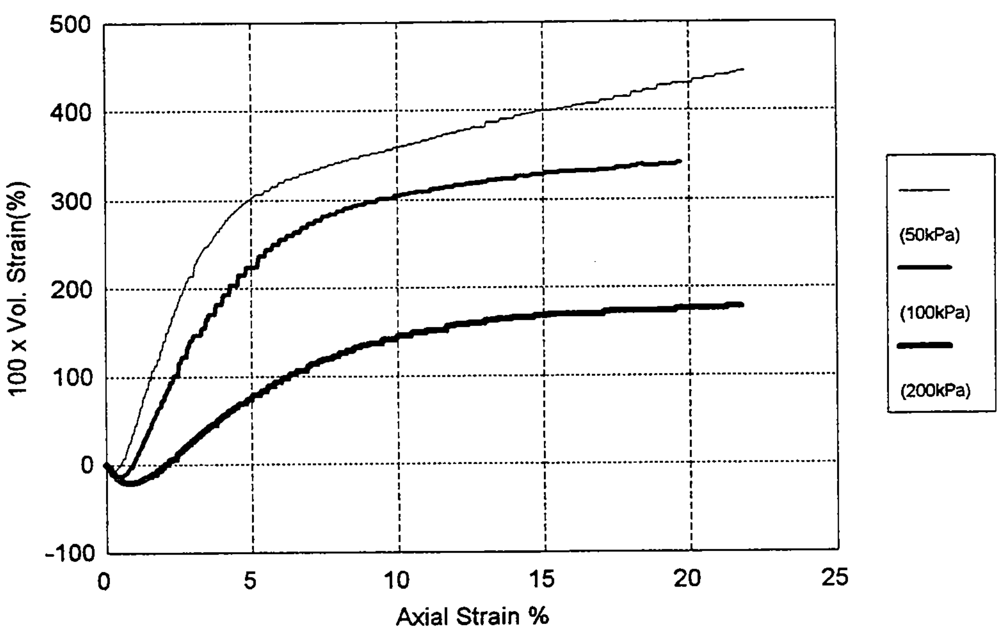
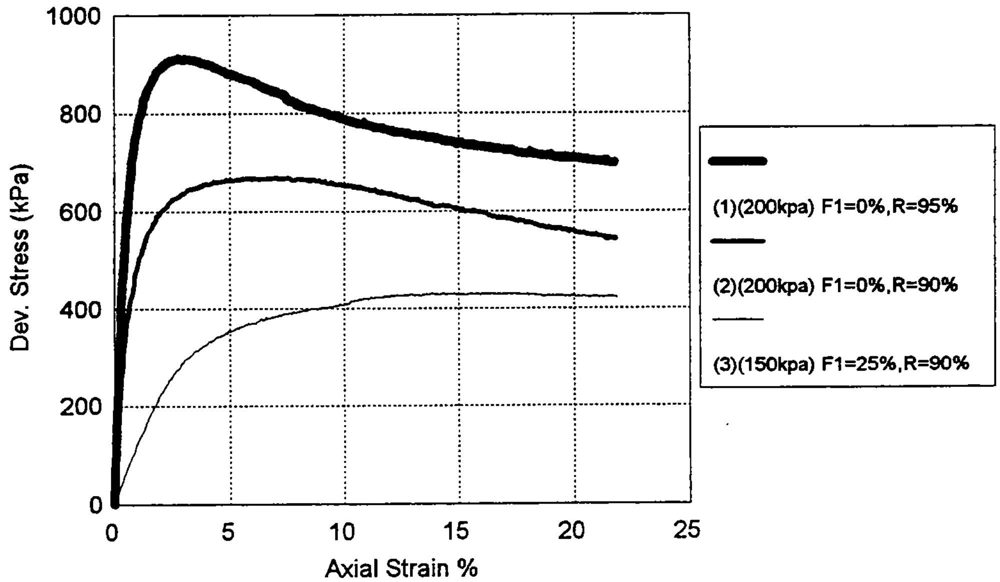


Figure 3. Stress-Strain and Volume Change Strain for Plant A and R = 75%.



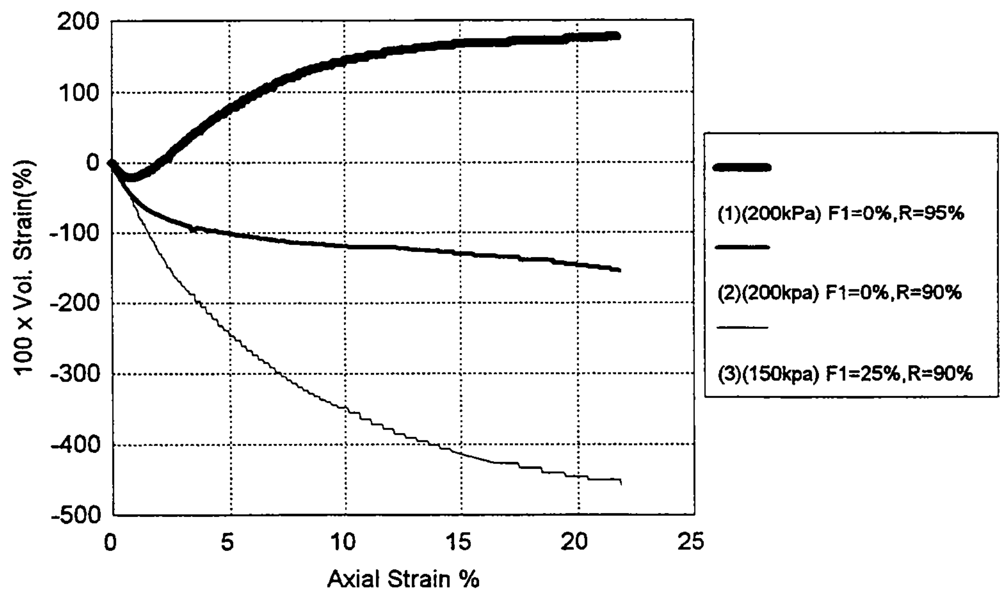


Figure 4. Stress-Strain and Volume Change Strain for Plant A; F_1 = 0% and 25%.

may be expected of these mixes. And it is likely that their performance will be predicted to be adequate for most embankment applications. No particular compaction problems (other than dust control) would be expected.

The use of an optimum mix for co-disposed ash is even less likely, since only limited volumes of this fly/bottom ratio would exist. Again, it is quite reasonable to use the materials as they are commonly encountered in the disposal area. Laboratory testing will be required, in order to predict performance, but it is likely that their performance will be adequate for most applications. The problem of variability and consequent difficulty in setting and implementing compaction controls is an important one. A set of control compaction curves must be pre-established for the borrow area, and one-point compaction tests (Holtz and Kovacs, 1981) must be routinely run in the field to determine the proper compaction control curve. The problem is similar to natural borrow areas that contain a variety of soils. Dust control is again highly important.

To control erosion and to reduce infiltration into the ash embankment, soil encasement is recommended (Huang, 1990). Ashes are usually qualified environmentally as to the concentrations of heavy metals in the leachates. Most ashes can be expected to pass these requirements.

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Abandoned Underground Mine Inventory and Risk Assessment

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ABSTRACT

On March 4, 1995, the eastbound driving lane of Interstate Route (I.R.) 70 in Guernsey County, Ohio collapsed due to subsidence related to abandoned underground mines. The Ohio Department of Transportation (ODOT) since that time has undertaken several site investigations and three notable mine remediation projects on interstate highways in an effort to prevent such subsidence from occurring again in the roadway. The remediation methods have consisted of the grouting of mine voids, construction of land bridges, and the removal and replacement of overburden.

As a result of the March, 1995 collapse of I.R.70, the ODOT Abandoned Underground Mine Inventory and Risk Assessment process was conceived. This process is a proactive response to the need to locate and assess the risk of all mapped or otherwise identified roadway sites beneath which abandoned underground mines exist. The process, as currently documented in a draft manual, is comprised of four basic activities: 1) the establishment of an inventory of all roadway sites beneath which abandoned underground mines may exist; 2) the assessment of the risk to the safety of the traveling public which each site represents; 3) the remediation of sites, if necessary, and; 4) the permanent monitoring of sites.

INTRODUCTION

The eastbound driving lane of Interstate Route (I.R.) 70 in Guernsey County, Ohio collapsed on March 4, 1995 due to subsidence related to abandoned underground mines. The Ohio Department of Transportation (ODOT) recognized that similar sites may exist beneath interstate highways and other roadways. Consequently, an effort was initiated to develop and implement an Abandoned Underground Mine Inventory and Risk Assessment process. The process is currently documented in a draft implementation manual which must still undergo formal internal and external review, and resulting revisions before official ODOT adoption for statewide implementation. This process is a proactive response to the need to locate and assess the risk of all mapped or otherwise identified roadway sites beneath which abandoned underground mines exist.

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The overall purpose of this inventory and risk assessment effort is to enhance the safety of the traveling public. The possibility of sudden abandoned underground mine subsidence in roadways, which could result in fatalities or bodily injuries, will be minimized. The process will: 1) reduce liability by identifying and prioritizing high risk sites, permitting a systematic response; 2) provide a means to budget funds for the remediation of sites to a predetermined risk level, and; 3) create a valuable informational resource available to all staff. This database will be a tool utilized to avoid or anticipate potentially unstable underground conditions during project planning, design, construction, and maintenance.

The first reported production of coal in Ohio was in 1800, three years before the state's entrance into the Union (Crowell, 1995). Since that time, varying amounts and forms of recoverable resource mining have occurred in Ohio. The age of the majority of the abandoned underground mines associated with these sites ranges from 50 to 150 years. The Ohio Department of Natural Resources, Division of Geological Survey has detailed abandonment maps for approximately 4,200 underground mines. In addition to those mines, the Division of Geological Survey estimates there are approximately 2,000 mines in Ohio for which no detailed maps of the mine workings are available.

The development of an Abandoned Underground Mine Inventory and Risk Assessment process for all state roadway sites in Ohio is a formidable task. Thousands of such roadway sites may exist in Ohio. These roadway sites represent an existing, undefined and yet possibly significant risk to the safety of the traveling public. The Ohio counties for which there are available abandoned underground mine maps are included in ten of the twelve ODOT Districts.

No matter how diligently ODOT performs this lengthy inventory effort, some areas of abandoned underground mining will not be identified or located due to the lack of documentation or the poor quality of the available information. However, unanticipated roadway problems related to the presence of abandoned underground mines beneath the roadway will be kept to a minimum through the implementation of the process.

Key aspects of this inventory process are the sharing of information, communication and cooperative problem solving with various State and Federal governmental agencies. These agencies include: the Ohio Department of Natural Resources, Divisions of Mines and Reclamation, Geological Survey and Water; the Ohio Environmental Protection Agency, Division of Drinking and Ground Waters; the Ohio Mine Subsidence Insurance Underwriting Association; the Federal Highway Administration; and the U.S. Department of Interior, Office of Surface Mining. The Ohio Department of Natural Resources, Divisions of Mining and Reclamation, and Geological Survey are providing cooperative technical assistance in support of the ODOT effort to establish the inventory of roadway locations under which abandoned underground mines exist.

ODOT has recognized that, during the development of the process, abandoned underground mine problems might need to be investigated and possibly remediated. Since the 1995 acceptance of the inventory and risk assessment concept, efforts have been underway to create, document and implement the process. ODOT has also been responsive to possible abandoned mine subsidence problems detected during this document development period. During development of the manual, ODOT has undertaken field investigations on several other roadway sites. These investigations have been made in the interest of preventing subsidence similar to the I.R. 70 collapse from occurring again while the inventory and risk assessment process was being developed.

REMEDIAL RESPONSE DURING PROCESS DEVELOPMENT

Including the repair of the I.R. 70 collapse site, three notable mine remediation projects on interstate highways have been undertaken by ODOT. These projects were undertaken in response to roadway conditions suggesting mine subsidence activity. The remediation methods have consisted of the grouting of mine voids, construction of land bridges, and the removal and replacement of overburden.

I.R.70:

The first of these mine remediation projects on an interstate highway was undertaken in the area of I.R. 70 in Guernsey County, in eastern Ohio between March and July, 1995. After investigations by ODOT revealed possible mine-related subsidence activity, a consultant was retained for the purposes of project investigation, design, and construction management assistance. Unfortunately, in spite of monitoring of the site by ODOT every four hours, a fifteen foot diameter by ten foot deep hole suddenly opened in the eastbound travel lane at I.R.70. Three cars and a truck were damaged when they encountered this hole. Fortunately, no critical injuries resulted from this event.

The length of the project area was approximately 2,000 linear feet. The remedial effort involved the air rotary drilling of approximately 1,800 boreholes down to the mined coal seam interval, at a depth of approximately 65 feet. The average depth of the soil-bedrock interface was approximately 47 feet. The drilled boreholes were cased to bedrock to allow for tremie grouting of the lower lying mine-related voids. Approximately 18,000 cubic yards to flyash grout was tremied into subsurface void areas. Two land bridges, having lengths of 700 linear feet and 110 linear feet, were constructed over areas where the drilling and grouting program encountered high concentrations of caved and broken materials in the mined interval. The cost of this project, including land bridge installations and pavement replacement, was approximately $3.6 million.

I.R.70 and I.R.77 Interchange:

The second mine remediation project on an interstate highway was undertaken in the Interchange of I.R.70 and I.R. 77 in Guernsey County, in eastern Ohio between June and November, 1995. The site investigations and design for this project were performed by a team of engineering and construction staff which had been formed in response to the original remediation project on I.R.70.

The office and field investigations of this site revealed the need to perform remediation work on portions of all mainline lanes and all ramp lanes with the exception of one ramp. They also revealed large voids a few feet beneath the pavement of one ramp. This necessitated immediate closure and remediation of the ramp while project investigations and design were completed.

The project entailed approximately 5.7 lane-miles of roadway, including the work on all mainline lanes and ramps. The remedial effort involved the air rotary drilling of approximately 2,600 boreholes down to the mined coal seam interval, at a depth ranging from 10 to 100 feet. Approximately 80,000 cubic yards to flyash grout were tremied into subsurface void areas. The cost of this project was approximately $4.7 million.

I.R.470:

The third mine remediation project on an interstate highway was undertaken on I.R. 470 in Belmont County, in extreme eastern Ohio between September and December, 1996. This project area included 1700 linear feet of interstate highway just west of the structure extending over the Ohio River into Wheeling, West Virginia. This project area had exhibited pothole subsidence in ditch and backslope locations. It was designated by ODOT as a study area for the field testing of investigative techniques and site evaluation techniques being drafted for the manual.

Numerous forms of site investigations including ground penetrating radar, profilometer, ground survey, falling weight deflectometer, drilling, and borehole camera were performed on the site. Through these investigations it was determined that the overburden rock was extremely fractured. A large void was found to be migrating toward the surface. Consequently, it was decided to immediately close the roadway and remediate the site by excavation to the base of the mined coal seam and then backfilling.

The resulting mine remediation project involved approximately 400,000 total cubic yards of roadway excavation, new pavement, signs, lights, guardrail, striping and revegetation. The project was completed in approximately 14 weeks. The cost of this project was approximately $3 million.

Other Sites:

Two other highways have been investigated. One of these sites was remediated through a drilling and grouting program performed by ODOT forces.

LONG TERM PROCESS APPROACH

This process is the most logical and practical approach to responsibly monitor the safety of the roadways relative to abandoned underground mines. Due to the large number of sites, it is not logistically or financially responsible to commit limited forces and funding to random investigation and remediation of sites. A large portion of this effort is the gathering and reviewing of existing information to initially identify and evaluate the risk of sites. A cornerstone of the inventory and risk assessment process is the concept of "being as informed as possible" before ever committing limited resources to individual sites for priority investigations and, if necessary, remediation. The sites which pose the greatest threat to public safety will be assessed as having the highest priority for site investigations, and remediation if required.

The process, as currently documented in a draft manual, is comprised of four basic activities: 1) the establishment of an inventory of all roadway sites beneath which abandoned underground mines may exist; 2) the assessment of the risk to the safety of the traveling public which each site represents;

3) the remediation of sites, if necessary, and; 4) the permanent monitoring of sites. A process flow chart is presented as Figure 1.

ESTABLISHMENT OF AN INVENTORY OF SITES

The first step in this process is establishing an Inventory of Sites. An initial comprehensive site listing is established based on: 1) review of available records; 2) field report forms; 3) follow-up investigation of field report forms, and 4) field visits to all identified potential sites

The inventory portion of this process will involve collection and review of available records for abandoned underground mines within the inventory area. This information will be gathered from a variety of sources. The primary source of available, pertinent information will be the Ohio Department of Natural Resources, Division of Geological Survey. Field Report forms will also be provided to ODOT roadway maintenance and construction personnel for the purpose of gathering field information which might reflect mine-related problems beneath the roadway. Staff who are involved with the inventory process and are familiar with signs of mine-related problems will conduct a follow-up investigation of each Field Report form received.

Next, all sites will be visited and field information will be recorded. The goal will be to obtain enough information from one site visit to allow for the risk assessment of the site through Initial and Detailed Site Evaluations. Following this site visit, all sites which are not eliminated by field findings will be subject to periodic monitoring.

This portion of the process will produce an initial inventory of sites.

RISK ASSESSMENT

A large portion of the overall process is risk assessment of the established inventory of sites. The risk assessment portion of the process involves three levels of site evaluation: 1) Initial Site Evaluation; 2) Detailed Site Evaluation, and; 3) Priority Site Investigations and Recommendations. The risk assessment criteria used for all levels of site evaluation take into account two basic factors: 1) the existing site conditions and; 2) the level of the traveling public's exposure to those conditions.

The Initial Site Evaluation and Detailed Site Evaluation both apply weighted criteria to existing information and information obtained from one site visit. The Initial Site Evaluation will subdivide the initial inventory listing of sites into 5 risk assessment site groups. The Detailed Site Evaluation risk assessment will then be performed on each of the three highest risk site groups in the order of their priority level of risk. All sites within a site group will be evaluated using criteria considered pertinent to the nature of the sites within the group. The result of the Detailed Site Evaluation process will be a prioritized listing of the sites for each of the three highest risk site groups as determined by the Initial Site Evaluations.

ABANDONED UNDERGROUND MINE INVENTORY AND RISK ASSESSMENT

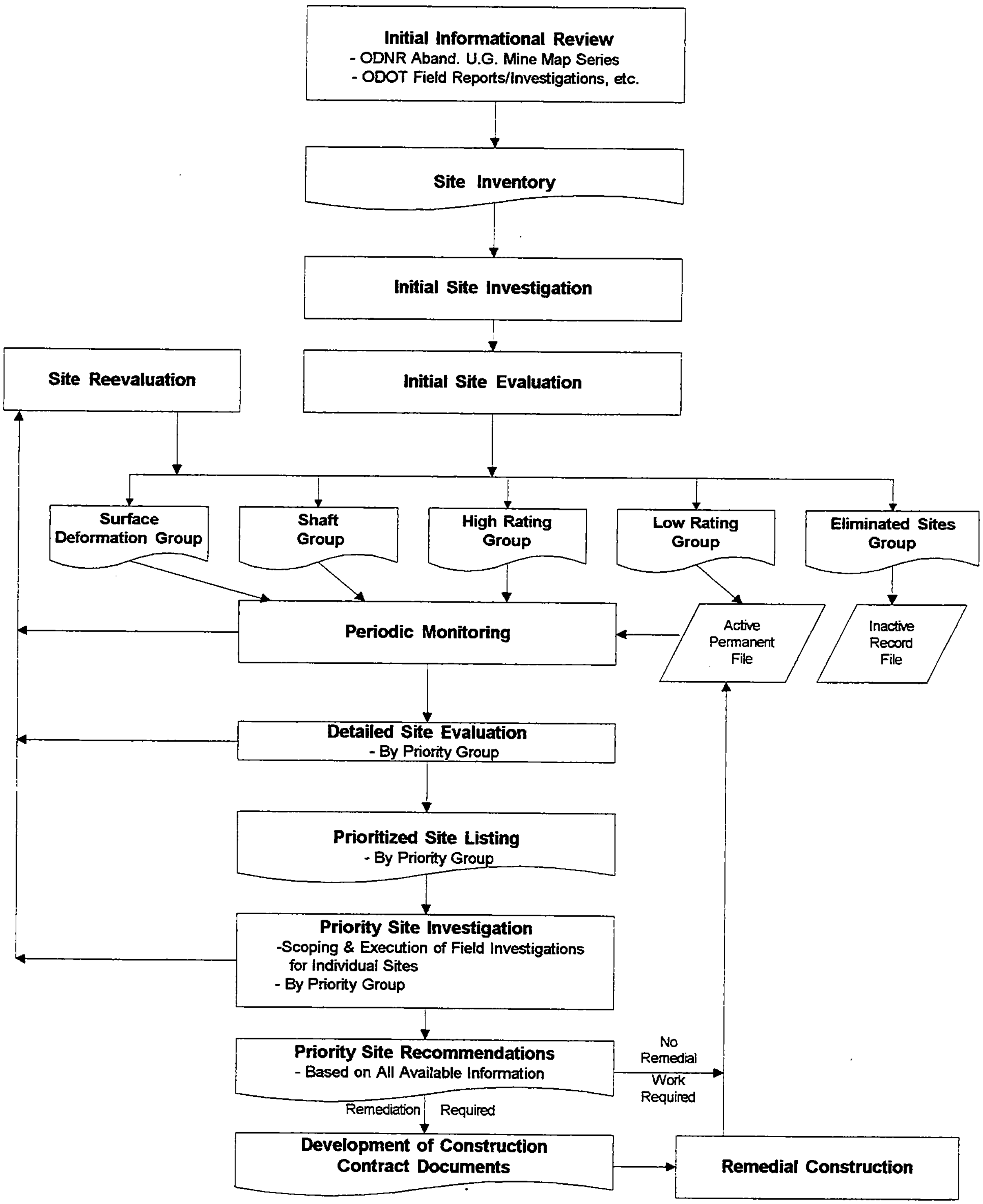

Figure 1

Initial Site Evaluation

This Initial Site Evaluation process will subdivide the entire inventory of sites into five risk-assessment site groups. This subdivision of the established inventory of sites will be accomplished by use of a standard site evaluation form to establish a numerical risk assessment rating for each site. Sites exhibiting surface deformation, containing mine shafts, or documented as posing no threat to roadway safety will be separated for inclusion in separate site groups.

The site groups, listed in the highest to lowest risk level, will include: 1) Surface Deformation; 2) Shaft; 3) High Rating; 4) Low Rating, and; 5) Eliminated Sites Groups. The Surface Deformation, Shaft, and High Rating Site Groups will proceed to the Detailed Site Evaluation portion of the process. The Low Rating Site Group will be placed under a permanent monitoring program and remain as active files in the inventory program. The Eliminated Site Group will become inactive but permanent record files of the inventory program.

Site Monitoring

Interim periodic monitoring will be initiated on all inventory sites following initial site visits. The frequency and extent of required monitoring activities will depend on site conditions as well as they can be identified at this point in the process.

All confirmed inventory sites will be subject to permanent site monitoring. Permanent site monitoring will provide a feedback loop in the process to allow for detection of changed conditions which might warrant site reevaluation. This aspect of the process will make it a dynamic, responsive risk management system. It is necessary because the age of the abandoned underground mines beneath the roadways is continuing to increase. The stability of those mines and associated overburden strata, at least in some cases, will continue to deteriorate with this increasing age.

When new or changed conditions or information is obtained for a given site, site reevaluation will be performed. This reevaluation will be accomplished by first completing a new Initial Site Evaluation form for the site. The site may be placed in a different risk assessment site group as the result of this reevaluation. Whether or not this occurs, the site will next be further reevaluated by completing a new Detailed Site Evaluation form for the appropriate site group. This work will determine the site's adjusted risk assessment priority in the appropriate site group.

Detailed Site Evaluation

Detailed Site Evaluation risk assessment will be performed on the Surface Deformation, Shaft and High Rating Site Groups as determined by the Initial Site Evaluations. This work will be performed by site group in the order of the group's priority level of risk.

Detailed Site Evaluation risk assessment will be performed by applying site evaluation criteria to available information and field observations. All sites within a site group will be evaluated using site evaluation criteria considered pertinent to the nature of that particular site group. A numerical site risk assessment rating will be established for each site by the completion of a standard site evaluation form. This work will provide a risk assessment for each site relative to all other sites within a given site group.

The Detailed Site Evaluation process will produce a prioritized listing of the sites for the Surface Deformation, Shaft and High Rating Site Groups. The inventory of sites at this point in the process will have been subdivided into risk level groups, with prioritization of individual sites within each group.

Priority Site Investigations:

Priority Site Investigations will be performed on each site within each of the three highest risk Detailed Site Evaluation site groups. These site investigations will be performed in the order of the prioritized listing of the sites within each of these groups. All sites within the particular Detailed Site Evaluation risk assessment group will be individually evaluated before evaluation proceeds to sites in the next lower site group.

Priority Site Recommendations:

The Priority Site Investigations will result in Priority Site Recommendations. These recommendations will document the need to either: 1) remediate defined site conditions and periodically monitor the site following construction, or; 2) defer remediation and periodically monitor the site. Some recommendations may involve emergency action or temporary roadway closure.

Whether remediation is recommended or not, all sites remain on the roadway inventory of abandoned underground mines and will continue to be periodically monitored.

REMEDIATION

Development of Construction Documents

Guidance is included in the manual for the development of remedial construction contract documents. Regardless of the extent of investigations performed, the actual site conditions cannot be fully determined prior to construction. Therefore, the manual places emphasis on flexibility of methods, quantities and project limits.

Existing conditions may change, or new conditions may develop on the site in the period required for contract document development. Guidance is included in the manual for continued site monitoring during Development of Construction Contract Documents.

Remedial Construction

Guidance is included in the manual for remedial construction. General information is provided regarding the importance of close inspection of the work, monitoring of time and materials usage, and accurate record keeping. Accurate construction records will be invaluable for post-construction monitoring and reference in the case of future subsidence conditions occurring adjacent to the project area.

Existing conditions may change, or new conditions may develop on the site during remedial construction. Certain forms of remediation may unintentionally induce additional mine-related settlement. Therefore, the manual provides guidance for site monitoring to detect possible changes during remedial construction.

Emergency Action / Road Closure

It is recognized that, at any point in the site investigation process, conditions may be discovered which warrant consideration of the need to close a roadway or portion of roadway to protect the traveling public from exposure to potentially hazardous conditions. The current draft manual provides guidance for such an eventuality through the discussion of parameters to consider in making these decisions.

SUMMARY

A comprehensive, state-wide abandoned underground mine inventory, utilizing standard risk assessment site evaluation criteria applied to existing information, will generate an initial prioritized listing of the locations where abandoned underground mines may exist below roadways under ODOT's jurisdiction.

The proposed risk-assessment techniques will direct the prioritized investigations and prioritized site remediation where necessary. The higher risk sites will be the first sites where field work and associated expenses will be incurred. This type of site will more likely require immediate emergency construction and/or top priority, non-emergency construction than other sites.

Some basic principles of the Abandoned Underground Mine Inventory and Risk Assessment are as follows: 1) work on the highest risk identified site at all times; 2) be as informed as possible before committing resources to a site, and; 3) be prepared to encounter "worst case " conditions for the nature of the site being investigated or remediated.

The benefits of this process will be:

1) Public Safety - The possibility of sudden abandoned underground mine subsidence in roadways, which could result in fatalities or bodily injuries, will be minimized.

2) Reduced Liability - The process will identify and prioritize high risk sites permitting a systematic response.

3) Budgetary Mechanism - The process, when combined with historic construction cost records, can be used to develop projected costs and budgets to reduce risks to a predetermined level.

4) Informational Resource - ODOT will benefit from this inventory process by creating a new database of information available to all staff. This database will be a tool utilized to avoid or anticipate potentially unstable underground conditions during project planning, design, construction and maintenance.

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PHYSICAL DISTRESS EVALUATION OF THE WEST ABUTMENT OF THE SR 22 BRIDGE OVER THE CONEMAUGH RIVER

by

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Abstract

The 600-foot long Conemaugh River Bridge, located on State Route 22 one-tenth mile west of Blairsville in southwestern Pennsylvania, is a product of the Commonwealth's post-World War II highway improvement program. An in-depth inspection of the forty-one year old bridge in 1994 led to the conclusion that the entire bridge deck needed to be replaced. The ensuing consideration of design alternatives brought attention to the fact that the West Abutment pile cap of the bridge had translated and rotated so far that the abutment backwall jammed against the bridge superstructure, a condition that raised questions about reusing the existing foundations for the rehabilitated bridge. The Consultant, under contract to the Pennsylvania Department of Transportation, thereupon undertook a study to establish what may have caused the abutment movements and to determine whether or not the existing piles retained sufficient capacity to support the rehabilitated bridge. The analyses indicated that the abutment movements were attributable largely to consolidation and compression of the approach embankment and underlying alluvial soils and the consequent deflections of the support piles. The results of the axial and lateral pile load tests supported reuse of the piles for the rehabilitated structure.

Introduction

The Conemaugh River has its source on the southeast boundary of the Appalachian Plateau and flows northwestward to the Allegheny River. In the vicinity of Blairsville, the Conemaugh River Valley is a conspicuous feature of the landscape and is one to two miles wide. The alluvial deposits of the floodplain in some places merge with and are not easily distinguished from the older Quaternary lower terrace deposits of the Parker Strath that underlie much of the area (Campbell, 1904).

State Route (SR) 22 is a main east-west regional thoroughfare that is carried over the Conemaugh River just west of Blairsville by a four span 600-foot long deck truss bridge supported on two abutments and four intervening piers, with a simple 50-foot approach span at the east end (Figure 1). Here, the Conemaugh River channel lies along the east side of the valley, bounded by a pronounced, west-facing rock slope upon which highway designers of the 1950's positioned the east abutment of the bridge. The absence of a rock slope of similar height on the west side of the river necessitated the construction of a 78-foot high approach embankment on which to place the west abutment. As is customary in Pennsylvania, a bridge abutment on an approach embankment is founded on piles driven through the embankment and underlying native soils to rock.

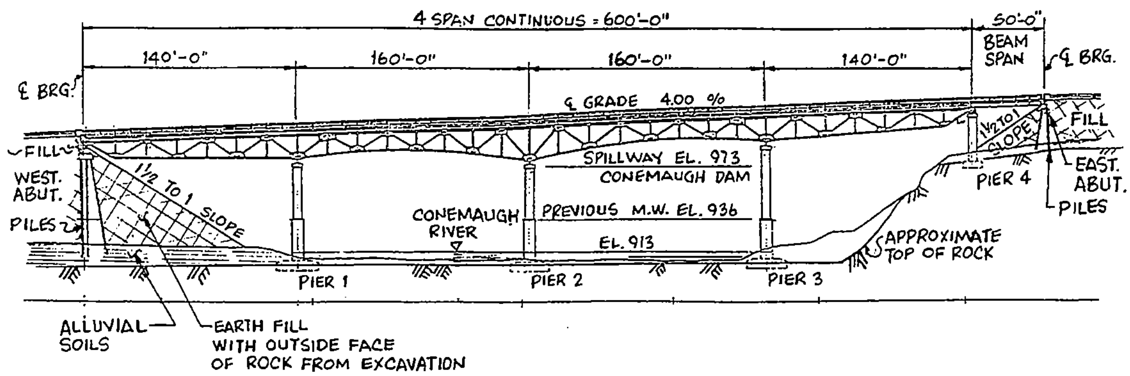

Figure 1

The West Abutment approach embankment is a compacted mixture of stiff to very stiff sandy, silty clay (cl) with rock fragments, along with intervals of dense random rock fill (chiefly tabular shale fragments with little or no soil binder) that was built directly upon the underlying 12.5-foot thick deposit of alluvial soils -- an upper 6.5 foot thick layer of medium stiff to stiff, low plasticity silty clay/clayey silt with some fine sand (cl/ml) and a lower 6 foot thick layer of very dense silty sand with gravel (sm). Underlying the alluvial soils are the nearly flat-lying sedimentary rock units of the Pennsylvanian age Casselman Formation (Conemaugh Group). No underground coal mines underlie the site. The ground water level generally approximates the level of the Conemaugh River; immediately landward of the West Abutment, perched ground water was found in the embankment as high as 43 feet above the top of the alluvial soils. During periods of flooding, the river level sometimes reaches the base of the West Abutment pile cap.

Condition of the West Abutment

A 1994 bridge inspection concluded that, due to general deterioration, the bridge needed to be rehabilitated with bridge deck replacement and upgrading of steel stringers and floor beams. Existing foundations were to be reused if at all possible.

Surveys, measurements, and examinations of expansion rocker bearings during the evaluation of bridge foundations revealed that the West Abutment, without settling measurably, had moved laterally in a riverward direction by approximately 8.5 inches before jamming against the south bridge truss. The pile cap, pedestal and wall stem had rotated backward about the bridge seat away from the river by 0.075 to 0.75 degree, and the backwall had rotated about the bridge seat towards the river by 0.3 to 0.4 degree (Figure 2). Heavy cracks were present throughout the abutment. The bridge truss member had penetrated the backwall. The tooth dam above the abutment was completely closed and the sidewalk sliding plates had jumped over the sidewalk fixed plates. The rocker bearings of the abutment were significantly tilted and approaching their limit of movement, a condition observed during a time of day and at a temperature when the bearings should have been nearly vertical.

Over what period of time the movement took place was not well documented. Little more was known than that the tooth expansion dam at the West Abutment, which had been open 0.25 inch in 1974 (21 years after the bridge was constructed), was "tight" in 1979; the West Abutment and bridge trusses were reportedly in contact in 1988.

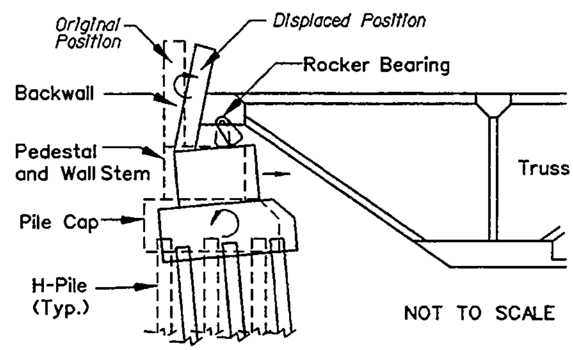

Figure 2

Certain aspects of the abutment movement, it seemed, could be plausibly explained: the slight backward rotation of the pile cap, for example, being the result of differential settlement of the approach embankment (the greatest settlement being behind the wall); the forward rotation of the backwall being the result of earth pressure acting against its rear face, possibly with pavement migration loads acting at the top of the backwall; and the absence of measurable abutment settlement being the result of the vertical support offered by the pile foundation. This notwithstanding, the reason for the significant lateral displacement of the West Abutment towards the bridge structure was unclear and, being suggestive of embankment failure and a possible loss of pile capacity, was a cause for concern.

Identifying the Probable Cause of the Abutment Movement

In light of the these circumstances, an investigation was undertaken to identify the cause of the West Abutment movement by systematically evaluating: 1) abutment movements expected under ordinary service conditions; 2) the approach embankment stability; and 3) abutment movements that might result from other sources of soil deformation.

As this evaluation was underway, it became apparent that on a nationwide basis, movements of bridge abutments are not uncommon. A study of nearly 600 bridge abutments across the United States (Moulton et al., 1985) revealed that over three quarters of them had experienced some type of movement. About sixty percent of the abutments founded on piles experienced horizontal movements and about thirty percent, both horizontal and vertical movements. In the cases reviewed, horizontal movements averaged about 2.5 inches, and vertical movements about 5.5 inches, sometimes reaching 50 inches. Movements of pile-founded abutments involved displacements towards the bridge structure in about two-thirds of the cases. Often the abutments moved inward until they became jammed against the beams or girders, which acted as struts, thereby preventing further horizontal movements. In the other third of the cases, movements of the abutments were in the opposite direction towards the approach fills. Although horizontal movements occurred most often for pile foundations in fine grained deposits overlying granular soils, data were insufficient in most cases to pinpoint the cause.

The West Abutment Foundation

The West Abutment of the Conemaugh River Bridge is supported on a deep foundation consisting of 44 steel piles driven to rock. The piles are arranged in three parallel rows spaced, along the base of the pile cap at 3.5 feet on center (front to back). The back row, near the heel of the cap, includes 18 vertical piles; the middle row, 8 vertical piles that are centered about the pedestals upon which the two trusses bear; and the front row, near the toe of the cap, 18 piles that are battered outward towards the river at 2H:12V. Within rows, the piles are spaced 3 feet on center in the more heavily loaded pedestals and 6 feet on center in the more lightly loaded areas between and beyond the pedestals. The piles are 12 WF 65 steel members. (At the time of construction, steel piles were substituted for concrete piles, which had a rated capacity of 30 tons.) The 3235 lineal feet of steel piling used in constructing the West Abutment foundation comprised piles of 66 to 80 foot length that extended a foot into the pile cap, with tips at or slightly below the top of rock, a medium soft, broken siltstone.

Expected Movements Under Ordinary Service Loads

The dead and live loads acting on a bridge abutment produce a resultant that is generally inclined downward in the direction of the bridge span. The vertical and battered support piles are intended to resist the force resultant with only limited vertical and lateral displacements.

The vertical(V) and horizontal(H) components of load acting on the West Abutment consist of: Dead Load --superstructure, including trusses, bearings, pile cap, backwall, pedestals, and misc., 1326.2 k(V); Earth Pressure Thrust, 360 k(H); and Live Load -- vehicular loading, 480 k(V). These loads are transferred to the piles through the pile cap. The total reaction P on each pile of a moment-resistant pile group is dictated by the number and arrangement of piles, represented by Peck, Hanson, and Thornburn (1978) as:

$$P = \frac{\sum V}{n} \pm \frac{\sum Md}{\sum d^2} \qquad (1)$$

where $\sum V$ is the sum of vertical loads acting on the pile group; n is the number of piles in the group; d is the distance from the center of gravity of the pile group to a specific pile; and $\sum M$ is the sum of moments about the center of gravity of the pile group.

An analysis of the West Abutment showed that:

1) The loads on the piles are unequal; the resultant of forces acts approximately two feet forward of the center of gravity of the pile foundation. The vertical component pile load is theoretically highest in the batter piles (69.5k), about half that amount in the center row piles (37k), and less than one-tenth that amount in the back row piles (4.5k). The loads

are far below the 172 kip pile capacity allowed by the customary 9 ksi allowable stress criterion. Settlements due to axial pile shortening would have been insignificant, and, indeed, settlements of the abutment were not observed.

2) The horizontal loads carried by individual piles were calculated to range from 11.6k to 18k (This assumes none of the lateral load is resisted by shear forces developed along the soil/pile cap interface). The lateral deflection from pile flexure would have been no more than 0.15 inch based on a COM 624G analysis (Reese, 1984). Conceivably, pavement migration could have imposed additional loads on the abutment, but would have had to exceed 400 kips per pile to develop a lateral deflection of 8 inches. Such loads are unrealistically high.

The small lateral and axial pile deflections and the substantial pile capacities computed for ordinary service conditions led to the conclusion that the West Abutment movements must have resulted from loads other than those accounted for in a conventional static abutment analysis. One assumption made in such an analysis is that the only forces acting on the piles are those delivered from the abutment through the pile cap. This may be an oversimplification where the piles are driven through soils that are unstable or compressible.

Stability of the West Abutment Embankment

An instrument survey conducted during the present investigation showed the geometry of the 1.5H:1V riverward-sloping embankment face to be in close agreement with the geometry presented on original design drawings. The embankment face exhibited no evidence of slumping or other instability. Inclinometer casings installed into in place rock at three locations along the embankment face gave no indication of mass movements during the 48-month period of monitoring. A stability analysis of the embankment using the simplified Bishop method indicated a minimum factor of safety against rotational failure of 1.7, with the critical circle passing beneath the abutment wall and daylighting from the embankment face. According to Pennsylvania Department of Transportation guidelines, a factor of safety of 1.3 is considered to represent an acceptable level of safety in cases where the soil properties have been determined by laboratory or field methods, as was the case here.

These findings are consistent with a condition of stability.

Expected Abutment Movements Under Conditions of Compressible Soil

Construction records for the bridge indicate that the approach embankment was brought to its final elevation before the West Abutment foundation was built. With the embankment nearly complete, the piles were driven and the pile cap was constructed. The bridge superstructure was then erected, after which the abutment backwall was constructed and backfilled. Having been built on a deposit of fine grained alluvial soils, the approach embankment is expected to have experienced time-dependent settlements. Settlements continuing during and after the period of pile installation would have subjected the piles to certain unanticipated loadings at levels below the pile cap. Negative skin friction that has resulted in the yielding of steel piles is reported by

Johannesen and Bjerrum (1965) and Bjerrum et al (1969). Lateral earth pressure on piling that has caused their bending and failure has been reported by Franx and Boonstra (1948) and Geuze (1948).

An analysis of the West Abutment showed that:

1) Settlements of as much as two feet may have taken place due to primary consolidation and secondary compression of the soils. Under the embankment weight, the clayey upper layer of alluvial soil is estimated to have consolidated nearly a foot and one half, and compressed secondarily another tenth of a foot. Over the same period of time, the embankment fill is estimated to have compressed about one half foot. The deterioration and crushing of shale particles in the more rocky zones of the fill is considered to have significantly contributed to the settlements.

2) Embankment settlements are estimated to have been progressively less in a riverward direction from the West Abutment, consistent with the lesser embankment height, thereby creating a settlement trough centered at or behind the West Abutment.

3) Nearly half of the primary consolidation and most of the secondary compression is estimated to have taken place after the piles were installed.

On this basis, it is probable that the piles had been subjected to settlement-related loadings at levels below the pile cap that were not accounted for in the original design.

Analyses indicated that the negative drag forces arising from settlement could conceivably have added as much as 6 kips of load per pile. Although, this would not have overloaded the piles, it might have created conditions favorable for bending of the battered piles by decreasing soil pressure on their lower faces. However, because this would have produced a forward rotation of the pile cap rather then the rearward rotation that was observed, negative drag forces were dismissed from consideration as a controlling mechanism for abutment movement.

Rather, the conclusion was reached that the proximate cause of the abutment movement was lateral displacement of the embankment and underlying alluvial soils towards the slope face as a consequence of settlement. A sense of the probable pattern of vertical and lateral deformations is provided by a numerical model (Figure 3), a finite difference formulation as described by Cundall et al., 1988. The model depicts both a settlement trough and lateral deformations of the embankment and alluvial soils in the direction of the slope face (most pronounced near mid-height of the embankment and virtually nil at the top of rock). Being very flexible in bending, the foundation piles of the West Abutment are expected to have deformed laterally towards the slope face in conformance with the lateral displacements of the embankment and native soils. This in combination with lateral earth pressures behind the abutment would account for the lateral translation of the abutment towards the slope face. Such lateral deformations can be likened to a Poisson effect and are generally proportional to the vertical deformations (Suzuki (1988) and Loganathan et al. (1992). Settlement of the embankment fill centered behind the abutment is the probable cause of the slight backward rotation of the pile cap. These mechanisms, in combination, are considered to offer a plausible explanation for the observed abutment displacements.

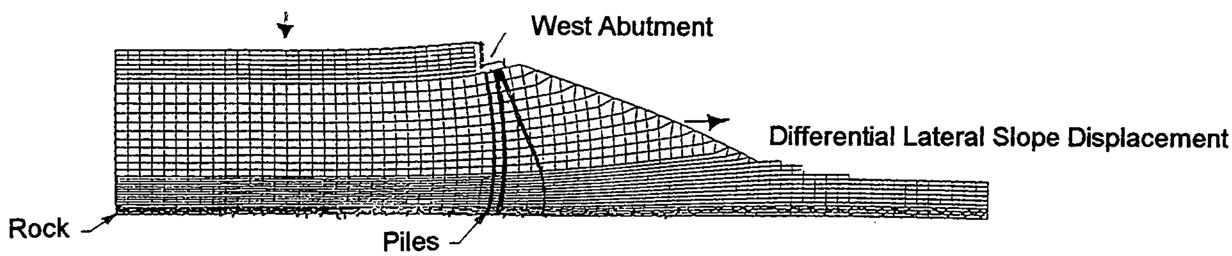

Figure 3

Pile Load Testing

The movement of West Abutment raised questions about the available capacity of the existing piles that could be resolved only by testing. It would have been desirable to evaluate the axial capacity of the existing piles using Dynamic Pile Monitoring (DPM) during redriving. However, because the bridge had to continue carrying traffic during construction, the removal of the pile cap in half widths to conduct the DPM (or to perform a conventional load test) was impractical. Static load testing of a representative pile was therefore carried out in an excavation created beneath the pile cap.

Site Preparation. The load tests were conducted on a 2H:12V battered pile in the front row under the south pedestal area. To expose the test pile, a pit was excavated beneath the pile cap by a two man crew in one day using a hydraulically-operated clay spade. Four piles were partially exposed in the process. None were found not to have corroded significantly. The 0.4 foot gap observed between the bottom of the concrete pile cap and the top of the soil within the excavated pit supported the proposition that the lateral abutment movement was related to settlement of the embankment and the underlying alluvial soils.

A 0.5 foot long section of the pile was cut out to permit installation of the hydraulic ram for load testing. As the pile was cut, the exposed section of the pile below the cut -- the lower section that extends downward into the earth embankment -- moved to the west (toward the approach fill) by approximately 0.25 inch while the exposed section of pile above the cut -- the upper section that is embedded in the concrete pile cap -- remained stationary.

Cyclic Axial Load Test. The test setup with dial gauges, hydraulic ram, and reference beams is shown in Figure 4. Axial testing consisted of loading the pile according to the sequences and time increments specified by ASTM D1143, Paragraphs 5.2 and 6.2. The loads were applied using a 100 ton hydraulic jack, with the pile cap being employed as the reaction block. The sequence involved four successive cycles of loading and unloading, beginning with 50% of the design load and increasing to 100%, 150% and finally 200% of the design load of 90 kips specified by the Consultant's bridge designers. The maximum load of 180 kips was held for slightly over 12 hours prior to incremental unloading that continued until no load remained on the test pile. Movements of the test pile were monitored with five two-inch travel dial gauges having a precision of 0.001 inch.

The Load vs. Deflection plot of the four load/unload cycles (Figure 5) shows an axial shortening of 0.16 inch at the maximum test load of 180 kips. The corresponding uplift of the pile cap was insignificant, as was the ground surface settlement. Because no more than 0.16 inch of axial deflection was measured during the test, it appears that only the upper 40 feet of pile effectively participated in the test; that is, the test loads did not extend to the pile tip. (From the elastic equation [PL/AE], an axial shortening of 0.292 inch is computed for a 75 foot long steel pile with 19.1 square inch cross section under a 180 kip axial load.)

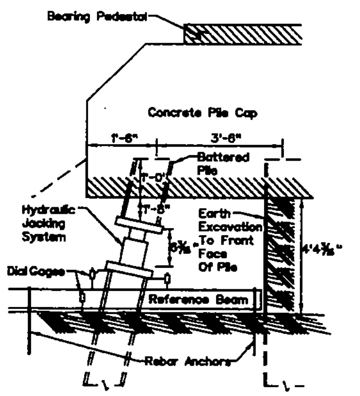

Figure 4

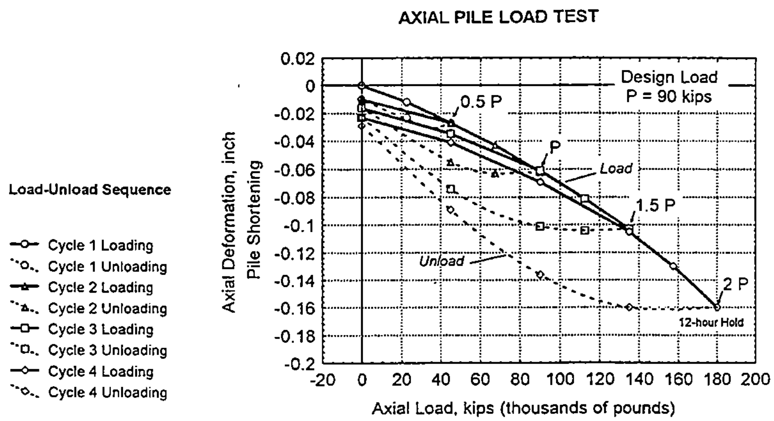

Figure 5

Lateral Load Test. The test setup with dial gauges, hydraulic ram, scale and mirror, tiltmeter plate and reference beams is shown in Figure 6. The test was performed on the same battered pile as the axial test. The loads were applied using the 100 ton hydraulic jack, with a pile in the next row providing the reaction. Pile deflections were monitored with two dial gauges having a precision of 0.001 inch, and with a tiltmeter and a tensioned wire and mirror arrangement.

The test was conducted according to the sequence specified in ASTM D 3966, Paragraphs 6.3 and 7.3, which involved loading the test pile to a maximum of twice the design load in increments of 25% of the design load, followed by incremental unloading to until no load remained on the test pile. A design load of 12 kips was specified by the Consultant's bridge designers. The maximum test load of 24 kips (twice the specified design load) was held for one hour.

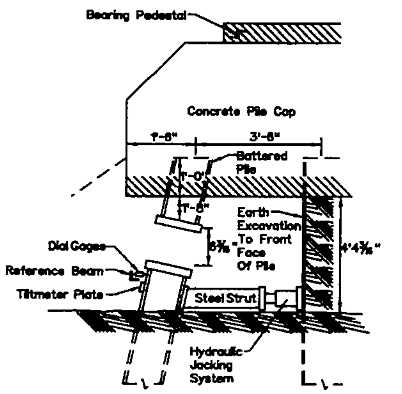

Figure 6

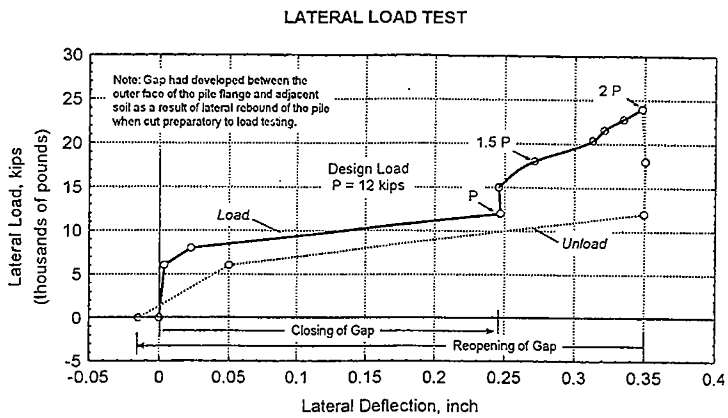

Figure 7

The Load vs. Deflection plot (Figure 7) shows a lateral pile deflection of 0.348 inch at the maximum test load. Much of this deflection (about 0.25 inch) took place between the 8 and 12 kip load levels as the gap, which had developed when the pile rebounded upon being cut preparatory to load testing, closed under the progressively higher lateral loads imposed during the test. At loads greater than 12 kips, the front face of the pile came in full contact with the adjoining soil and the deflections were consequently much smaller. The rotation of the exposed section of pile in a vertical east-west plane under the maximum test load was 0.23 degree.

Conclusions

Visual indications and survey data from a 1994 bridge inspection showed that the pile-supported West Abutment of the forty-one year old SR 22 Bridge over the Conemaugh River had moved in a riverward (eastward) direction by 8.5 inches. The backwall and pile cap had both rotated about the bridge seat, with the bridge truss penetrating the backwall. The abutment had not settled. The time period over which these displacements occurred was uncertain.

A careful consideration of the geometry and stability of the embankment, the abutment, the piling, and the soil properties led to the conclusion that the most probable cause of the movements was the lateral deflection of the foundation piles brought about by the settlement of the soils under the weight of the embankment in combination with lateral earth pressures on the abutment.

An axial and lateral load test on a representative pile supported reuse of the existing piles for the rehabilitated bridge.

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Physical and Chemical Evaluation of Soil-Tire Mixtures for Highway Applications

by

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Abstract

Mixtures of shredded tire material with silt and clay, containing 0% to 100% shreds by weight, were tested for a series of engineering properties including compaction characteristics, permeability, unconfined compressive strength, compression index, friction angle, and cohesion. In addition, the leachate from shredded tire material, soil-tire mixtures, and a test embankment containing 30% shredded tire and 70% clay, was analyzed for chemical composition. The results show that density and unconfined compressive strength decrease, and permeability increases, with increasing shredded tire content for both soil types and all three tire sizes (1/4"-1/2", 1/2"-1", 1"-1.5") used in the study. The addition of 1/4"-1/2" size shredded tire improves the friction angle for both silt and clay but also increases their compression index values. The results of leachate analyses show that concentrations of trace elements from soil-tire mixtures are less than the maximum allowed contaminant levels specified in U.S. Environmental Protection Agency's regulations. Based on these results, soil-tire mixtures can be used as a light weight fill material as well as in situations where improvement in drainage characteristics is required.

Introduction

Of the 279 million scrap tires generated each year in the United States, nearly 85% are landfilled, stockpiled, or illegally dumped (House Bill S.2462, 1990). Since whole tires do not compact well, they tend to rise to the surface of the landfill where they disrupt the landfill cap and allow water to infiltrate the landfill. When stockpiled, scrap tires provide an ideal breeding space for rats, mosquitoes, and other disease vectors, causing a public health hazard. Scrap-tire fires are extremely difficult to extinguish and pose a serious threat to health and environment due to liquid and gaseous emissions.

The best way to reduce the environmental and health hazards associated with scrap tires is to minimize, and ultimately eliminate, the landfilling and stockpiling of scrap tires. This can be accomplished by finding alternative uses for scrap tires. A large-scale potential use of scrap tires can be in soil stabilization. In highway engineering, tire-stabilized soils may be used as a lightweight, or semi-lightweight, fill material for embankments and for reconstruction of potentially unstable or failed slopes. Previous studies by Ahmad (1992), Bosscher et al. (1992), Lamb (1992), Upton and Machan (1993), and Black and Shakoor (1994) have indicated the potential for such applications but more research is needed to determine the optimum size and amount of tire shreds for stabilization of different soils, to develop field construction procedures, and to evaluate the environmental impact of the use of tire-stabilized soils.

Objectives

The specific objectives of this study were to:

1. Investigate the engineering properties of soil-tire mixtures containing 0% to 100% shredded tire by weight, at 10% intervals, so that the optimum amount of shredded tire needed for achieving the desired soil properties could be determined.

2. Investigate the effect of size of the shredded tire material on the engineering properties of interest.

3. Determine the chemical characteristics of the leachate generated by the interaction of water and various soil-tire mixtures.

4. Determine the potential applications of tire-stabilized soils in highway engineering.

Research Methods

Two different soil types (a silt and a clay) and three different size ranges of shredded tire (1/4"-1/2", 1/2"-1", and 1"-1 1/2") were used in the study. The soils were tested for Atterberg limits, moisture-density relationships, permeability, unconfined compressive strength, consolidation behavior, and shear strength parameters. All three tire sizes were tested for their dry densities and permeabilities. The soil-tire mixtures, containing 0% to 100% shredded tire by weight, at 10% interval, were tested for moisture-density relationships, permeability, unconfined compressive strength, consolidation behavior, and shear strength parameters. The soil-tire mixtures were prepared using both soil types and each of the three sizes of shredded tire. The permeability, unconfined compressive strength, consolidation behavior, and shear strength parameters were determined for samples compacted to at least 95% of the maximum dry density and within $\pm 2\%$ of the optimum water content. The consolidation characteristics of soil-tire mixtures containing 1/4"-1/2" size shredded tire material were evaluated from the volume change data obtained during the consolidation stage of the triaxial testing. All lab tests were performed in accordance with the standard procedures specified by the American Society for Testing and Materials (ASTM), (ASTM, 1993).

Based on the results of lab tests, a test embankment 50 feet long, 22 feet wide, and 5 feet high was constructed on university property. The embankment material consisted of a mixture of 70% clay and 30% shredded tire of 1/2"-1" size. The embankment was monitored for settlement and slope stability problems for a period of one year. Leachate samples were collected on a bi-weekly basis from the lysimeters installed in the embankment. Embankment construction also helped evaluate construction procedures (mixing, compacting, etc.) when dealing with soil-tire mixtures. Figure 1 shows an overview of the test embankment.

The leachate generated in the field by the interaction of rain water and all three sizes of shredded tire material, soil-tire mixtures, and the test embankment was analyzed for aluminum, barium, calcium, cadmium, cobalt, chromium, copper, iron, manganese, lead, zinc.

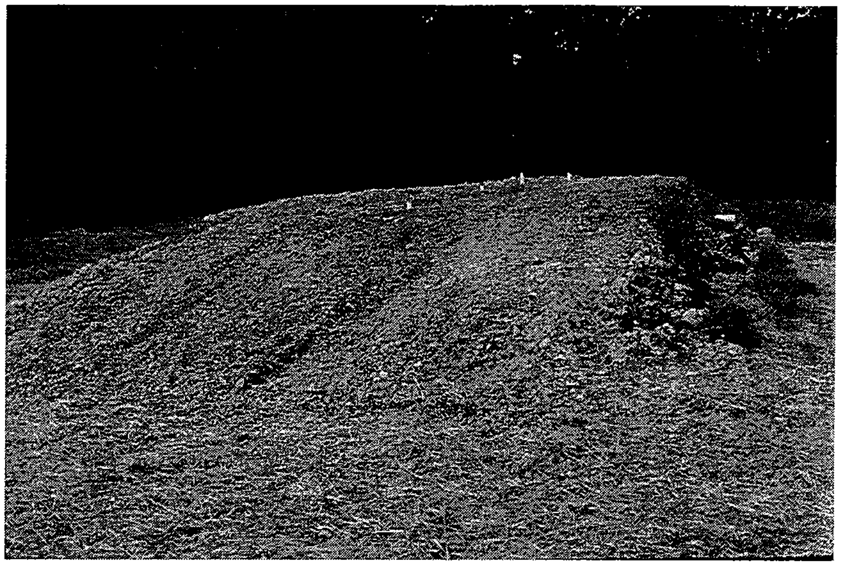

Figure 1. Overview of test embankment. Notice the four white lysimeters along the centerline and the PVC drainpipe on the side for collecting leachate.

Engineering Properties of Soil, Shredded Tire, and Soil-Tire Mixtures

The engineering properties of the two soil types used in this study are presented in Tables 1 and 2, respectively. The dry density and permeability values for the three tire sizes are quite similar to each other. The variation of engineering properties with increasing shredded tire content for soil-tire mixtures is shown in Figures 2 through 9. The strength characteristics of soil-tire mixtures, as determined by triaxial test, are shown in Table 3. Table 4 presents the results of consolidation tests for the mixtures. Figure 2 shows that the density of a soil-tire mixture is reduced to 2/3 of the density for soil alone (a requirement of lightweight fill) with the addition of 60% tire content. This was found to be the case for all three size ranges of shredded tire used.

Leachate Analysis

The leachate was collected periodically from all three size ranges of shredded tire as well as from soil-tire mixtures (silt mixed with 30% shredded tire by weight, and clay mixed with 30% and 60% shredded tire by weight, respectively) for a period of one year. The leachate samples were also collected from various depths of the test embankment on a bi-weekly basis.

All leachate samples were analyzed for selected trace metals (aluminum, barium, cadmium, cobalt, chromium, copper, iron, manganese, lead, and zinc) using ICP techniques. The results for pure tire are presented in Table 5, for soil-tire mixtures in Table 6, and for the field embankment in Table 7. The concentrations of Cd, Cr, Cu, and Pb are less than the Maximum Contaminant Levels (MCLs) established by the U.S. Environmental Protection Agency (EPA, 1992). The concentration of Ba is close to the MCL except silt and clay mixtures with 30% shredded tire of 1/4"-1/2" size. Table 6 also shows that mixing shredded tire with soil greatly reduces the concentration of contaminants.

Table 1: Engineering properties of the soils used.

Property	Soil Used	
	Silt	Clay
Liquid Limit(LL)	26.9	31.5
Plastic Limit(PL)	24.2	20.3
Plasticity Index(PI)	2.7	11.2
Optimum Water Content(%)	15.7	16.8
Maximum Dry Density (pcf)*	106.8	102.5
Permeability (cm/sec)	4.48E-07	4.32E-08
Compressive Strength (psf)**	4077	6600
Soil Classification (USCS)	ML	CL

Table 2: Dry density and permeability values for the shredded tire.

Property	Shredded Tire Size		
	1/4"-1/2"	1/2"-1"	1"-1 1/2"
Dry Density (pcf)*	43.2	43.5	43.6
Permeability (cm/sec)	0.16	0.18	0.18

Engineering Applications of Tire-stabilized Soils as Lightweight Fill

The requirements of a lightweight fill material are that it should have low density, high shear strength, and good drainage characteristics. Since tire-stabilized soils meet these requirements, they can be used to construct highway embankments on soft ground, such as peat and clay, to reconstruct already failed or potentially unstable slopes, to develop play grounds, and to fill low-lying areas.

Conclusions

1. The maximum dry density, optimum water content, and unconfined compressive strength of soil-tire mixtures decrease, and the permeability increases, with increasing shredded tire content for both soil types and all three tire sizes used in this study.

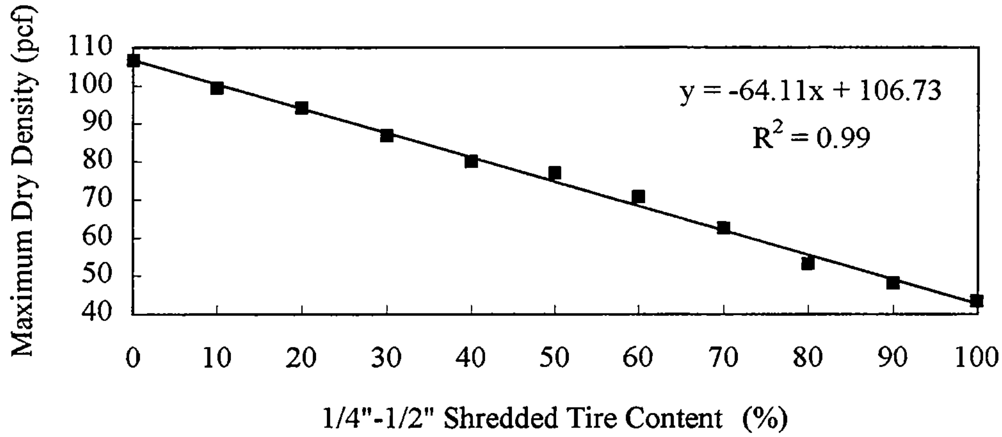

Figure 2: Maximum dry density vs shredded tire content for silt-tire mixtures.

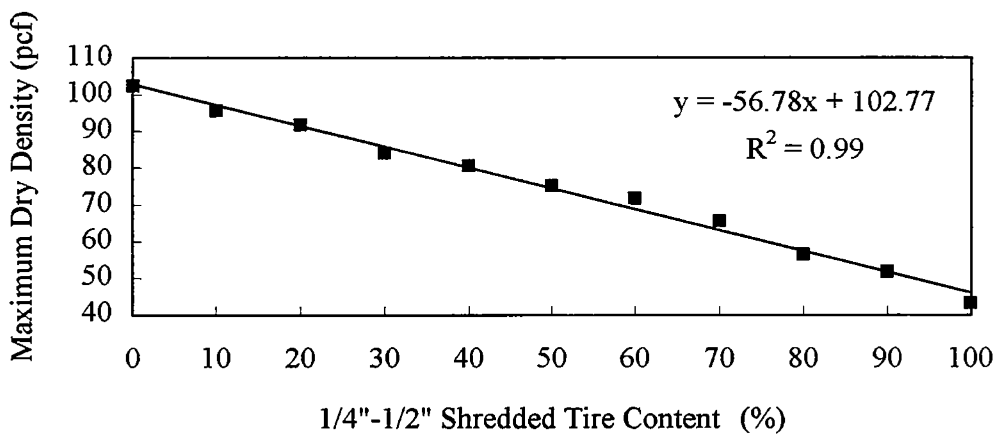

Figure 3: Maximum dry density vs shredded tire content for clay-tire mixtures.

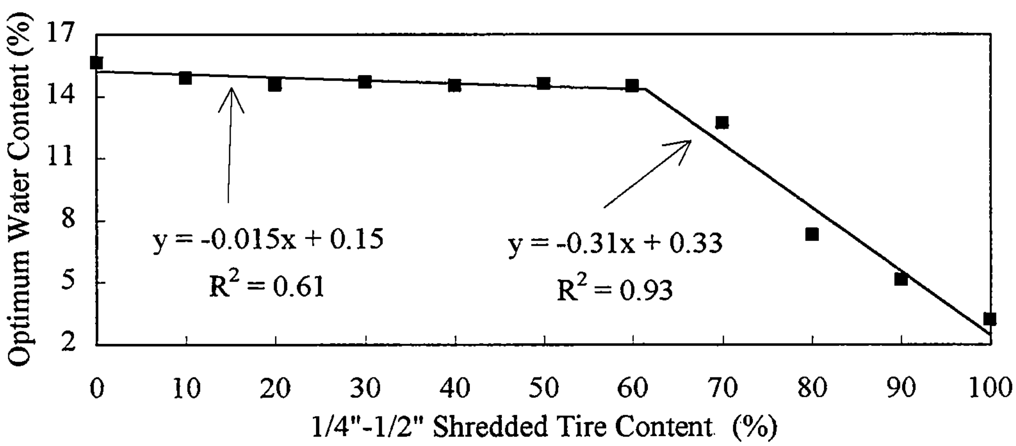

Figure 4: Optimum water content vs shredded tire content for silt-tire mixtures.

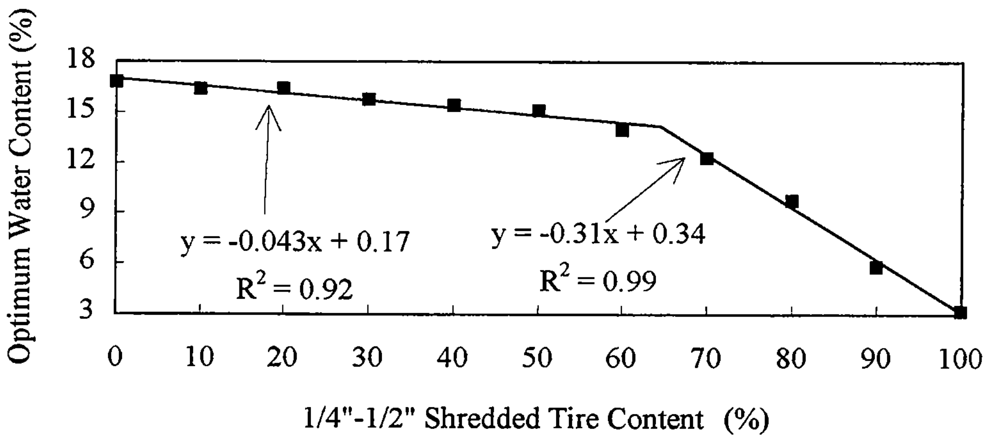

Figure 5: Optimum water content vs shredded tire content for clay-tire mixtures.

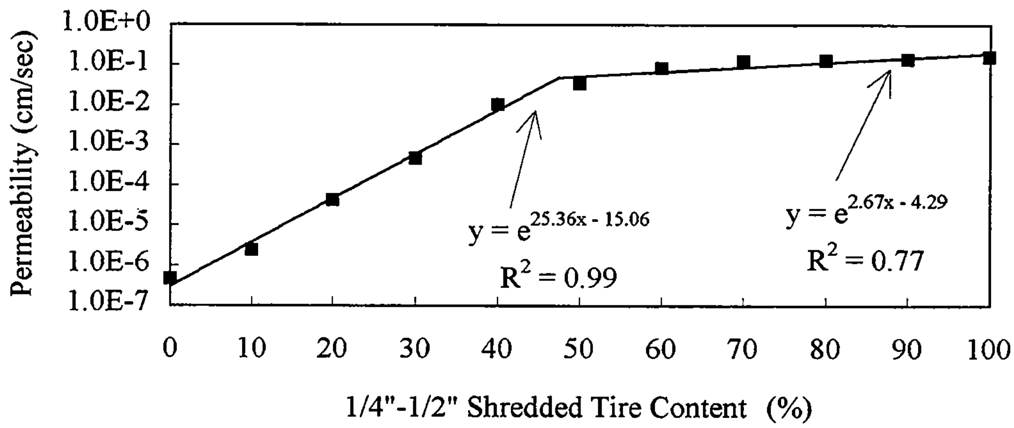

Figure 6: Permeability vs shredded tire content for silt-tire mixtures.

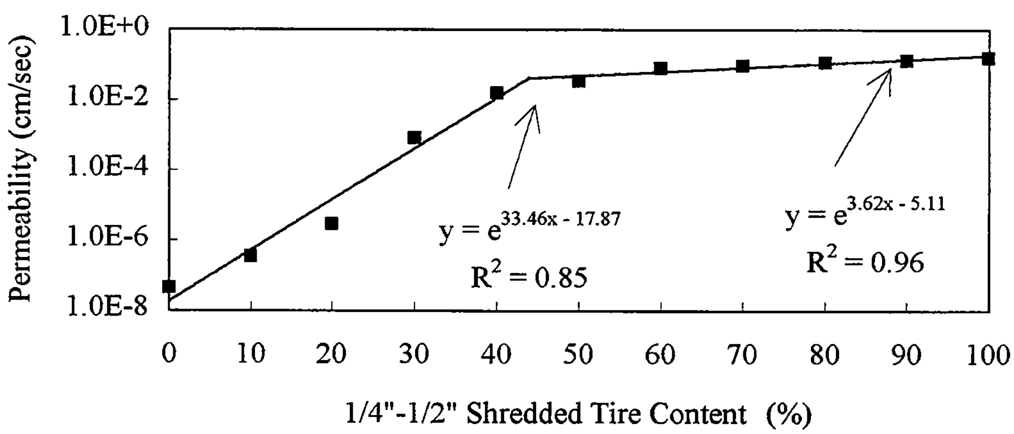

Figure 7: Permeability vs shredded tire content for clay-tire mixtures.

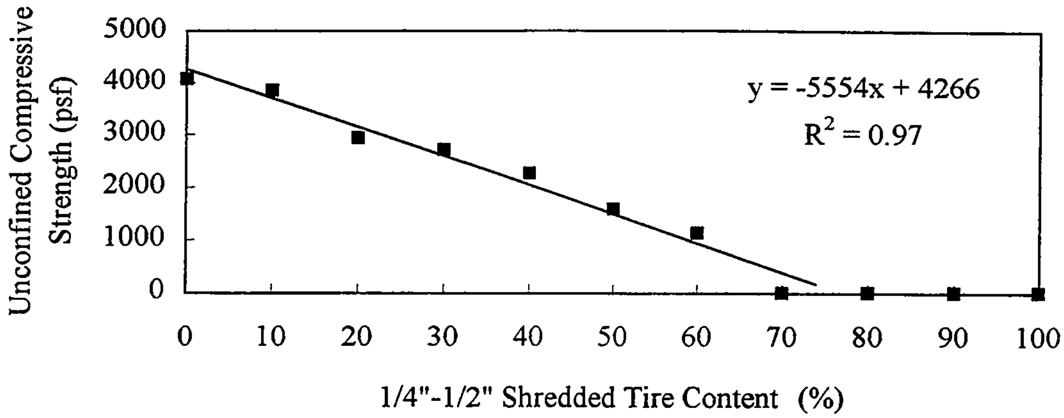

Figure 8: Unconfined compressive strength versus shredded tire content for silt-tire mixtures.

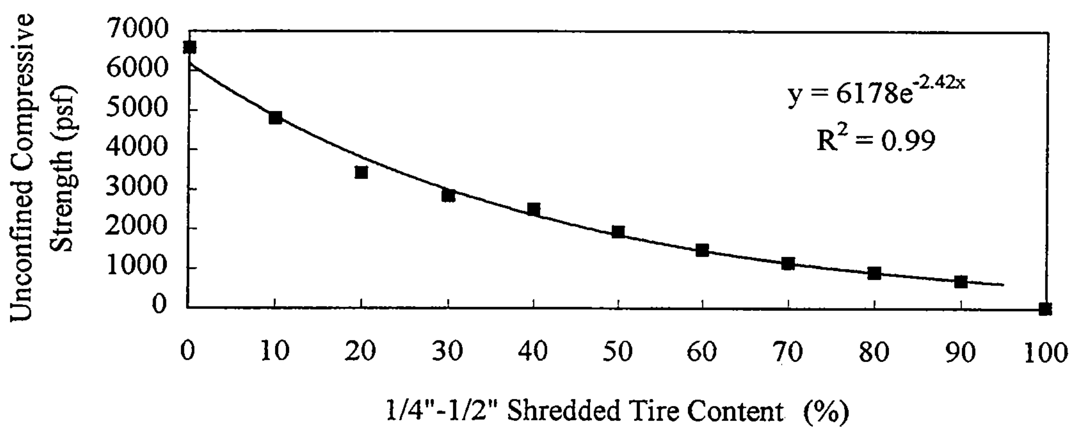

Figure 9: Unconfined compressive strength versus shredded tire content for clay-tire mixtures.

2. The concentrations of leachate for all three tire sizes and soil-tire mixtures are significantly below the maximum allowable contaminant levels as specified by the U.S. Environmental Protection Agency for Cd, Cr, and Cu and are close to the maximum allowable limit for Ba.

3. The optimum amount of shredded tire material needed to improve soil properties depends on the intended engineering application. For usage as a lightweight fill, the dry density of soil-tire mixtures is reduced to 2/3 of the density for soil alone with the addition of 60% shredded tire of any of the three size ranges used.

4. The silt and clay soils stabilized with metal-free shredded tire material, ranging in size from 1/4"-1/2", can be used as a lightweight fill material without any detrimental effects on the environment.

Table 3: Shear strength parameters for silt-tire and clay-tire mixtures containing 1/4"-1/2" size shredded tire material.

Shredded Tire Content (%)	Friction Angle (°)		Cohesion (psf)	
	Silt-Tire	Clay-Tire	Silt-Tire	Clay-Tire
0	30	35	1656	2635
10	32	36	1498	1829
20	34	38	1122	1289
30	36	32	985	1200

Table 4: Compression index values for silt, clay, silt-tire and clay-tire mixtures containing 1/4"-1/2" size shredded tire material.

Shredded Tire Content (%)	Compression Index (Cc)	
	Silt-Tire	Clay-Tire
0	0.051	0.101
10	0.070	0.108
20	0.089	0.138
30	0.099	0.219

Acknowledgments

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Table 5: Results of leachate analyses for pure tire.

Ion Concentrations (mg/l)	Field Samples: Pure Tire		
	(1/4"-1/2")	(1/2"-1")	(1"-1.5")
Al	0.44	0.31	0.44
Ba	2.28	2.09	2.69
Ca	45.45	26.17	23.97
Cd	0.002	0.003	0.003
Co	0.05	0.06	0.08
Cr	0.02	0.03	0.02
Cu	0.03	0.01	0.01
Fe	1.17	4.65	3.11
Mn	0.73	1.52	3.38
Pb	0.01	0.003	0.005
Zn	67.09	42.5	32.73

Table 6: Results of leachate analyses for soil-tire mixtures.

Ion Concentrations (mg/l)	Field Samples: Soil-Tire Mixtures		
	70% Silt, 30% Tire (1/4"-1/2")	70% Clay, 30% Tire (1/4"-1/2")	40% Clay, 60% Tire (1/4"-1/2")
Al	0.11	0.01	0.14
Ba	1.94	0	2.05
Ca	580	502	2127
Cd	0.002	0	0.001
Co	0.04	0	0.04
Cr	0.07	0.04	0.09
Cu	0.02	0	0.01
Fe	10.41	50	2.31
Mn	84.11	15	9
Pb	0.003	0	0.009
Zn	19	6	27.6

Table 7: Results of leachate analysis for field embankment.

Ion Concentrations (mg/l)	Embankment (70% clay, 30% 1/2"-1" Tire)		MCLs (EPA Specified)
	1' depth	2' depth	(mg/l)
Al	0	0	-
Ba	0.07	0.009	2.00
Ca	255.24	563.01	-
Cd	0	0	0.005
Co	0.004	0.001	-
Cr	0	0	0.1
Cu	0	0	1.3
Fe	1.68	0.602	-
Mn	2.34	2.13	-
Pb	0	0.0001	0.015
Zn	0	0	-

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CASE STUDY - DRILLED SHAFT FOUNDATIONS
LAKE OF THE OZARKS COMMUNITY BRIDGE

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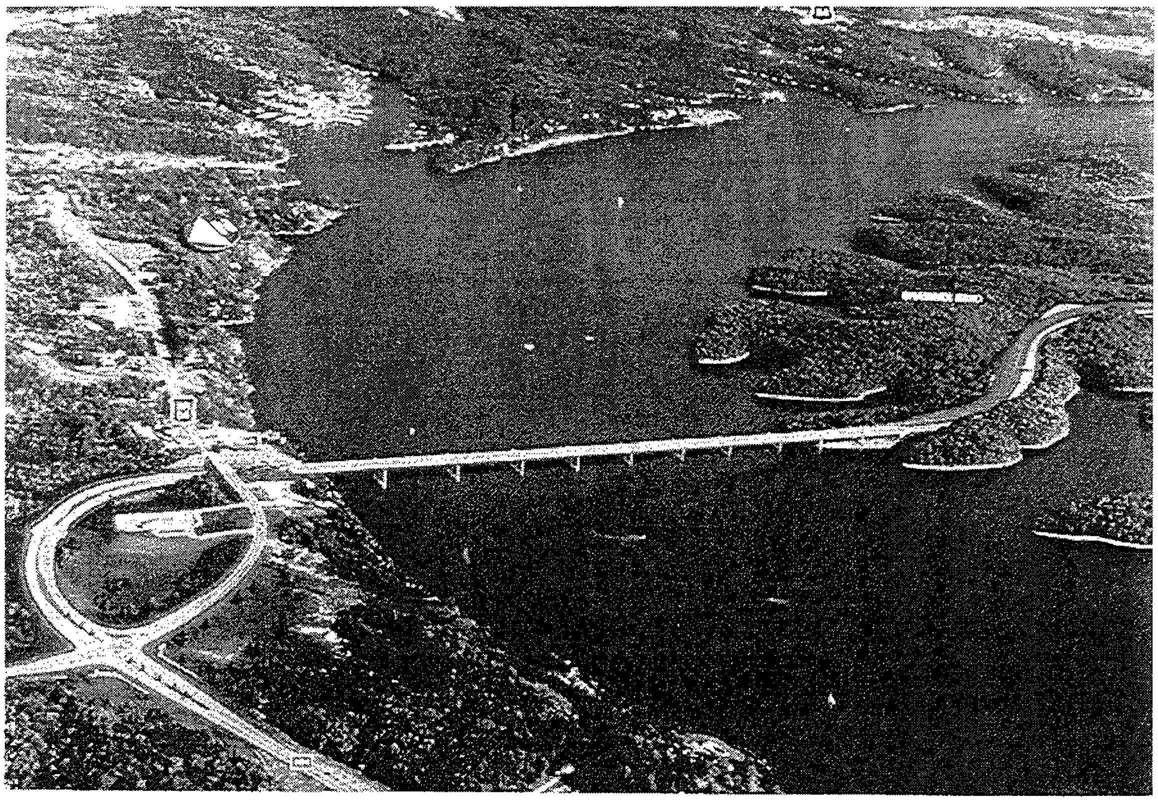

INTRODUCTION

The Lake of the Ozarks was formed by impounding the Osage River with Bagnell Dam by Union Electric in 1930. Geographically, the Lake is located near the center of Missouri in the beautiful Ozarks. The dam itself was a local feat, completed in only twenty six months. The hydroelectric project created a reservoir 94 miles in length with a shoreline of over 1300 miles. In order to help defray the cost of the project, Union Electric subsequently sold the shoreline property required for construction of the reservoir. Agreements between the landowners and Union Electric assure a nearly constant water level is maintained throughout the year. During the 1940s and 1950s, small resorts and fishing cabins began dotting the shoreline, but before long developers began realizing the potential of the area due to its central location between St. Louis and Kansas City. The area is now home to luxury resorts, condominiums, primary and secondary residences as well as an abundance of retail, restaurant and entertainment entities. The Lake of the Ozarks now lures 3 million tourists annually. The Lake is now home to boats ranging from small fishing to large multi-engine offshore power boats. "Party Cove" is known to have hundreds of boats joined together on a weekend.

BRIDGE FEASIBILITY STUDY

With 1300 miles of shoreline and only four bridge crossings, transportation routes around the lake average 30 to 50 miles on rural, two lane roads creating problems for law enforcement and public safety as well as impeding growth. Lengthy traffic delays are also common. Because of this, a need exists for a more efficient way from traveling from the "St. Louis" (commercial) side of the Lake to the "Kansas City" (residential) side, however, the bridge has never been a priority for the Missouri Highway and Transportation Department (MHTD) (now Missouri Department of Transportation or MoDOT). The proposed bridge crossing would link the Shawnee Bend Peninsula with U.S. Route Business 54 near Route HH in Osage Beach. The bridge would cut over an hour travel time on some routes and provide relief to overburdened local roads as well as provide access to the largely undeveloped thousands of acres on Shawnee Bend. A small private ferry currently serves as the only direct link between the sides.

For more than 30 years, residents and business owners in the region have proposed a bridge crossing over the main channel of the lake in this area. Legislation passed in 1990 enabled the financing, construction and operation of public transportation facilities by private, not-for-profit corporation in the State of Missouri. Following the enactment of this legislation, the Lake of the Ozarks Community Bridge Corporation was formed by local businessmen to design, construct, and operate the bridge. Tolls will be charged to use the bridge and repay the debt.

MoDOT participated in the project by providing the approach roadways and necessary local intersection improvements. MoDOT also administered the design of the bridge with the Consultant, construction bidding, and inspection services. The Bridge Corporation will reimburse MoDOT for tasks performed within designated Bridge Corporation project limits. Much of the right of way for the 6 miles of approach roadway was donated to the Department who, by agreement will assume ownership of the toll bridge when the financing debt is paid by the Corporation.

PRELIMINARY STUDIES

As part of the bridge type study, a preliminary geotechnical report was undertaken to determine the general geologic site conditions, overburden type and thickness, bedrock type and quality. These assessments would aid in the selection of foundations for the bridge type and cost analysis. The investigation included a review of available literature, site reconnaissance, geologic mapping, and lake bottom profiling using a sonar charting unit. Some understanding of the lake bottom sediments was also derived using the sonar. The preliminary studies also included a discussion of bridge foundation types, bearing values and suggested final exploration program.

Several bridge types were considered during this phase. Initially "signature" type bridges such as cable stayed and suspension were considered for aesthetics and uniqueness by adding amenities such as observation platforms or restaurants either atop the pylons or suspended under the bridge deck. Eventually, economics won out and a high level multi-span plate girder was selected. In part this selection was due to the ability to use single columns supported on single drilled shafts. This method has proven itself as a viable, economic foundation for bridges over deep water.

Traffic studies indicated a long term need for a four-lane facility, however, short term projections could be accommodated with a two lane facility. Therefore an economical, expandable bridge was selected over building an additional bridge when traffic increased.

The bridge selected, is a high level, multi-span plate girder. Span lengths are approximately 250 feet with a total bridge length of 2920 feet. A commitment to provide 70 feet of vertical clearance for navigation was to be provided. While this bridge is not unique, it differs from others of its type due to its excessive pier height. The tallest pier is 220 feet and necessitated post tensioning the bridge structure to the piers, increasing stiffness by having the substructure and superstructure act as a frame. This design greatly reduced the amount of reinforcement required in each column.

Further economic and traffic studies dictated a two lane superstructure should be build on a four lane substructure. The substructure at each pier consists of 3 columns supported on single drilled shafts. This selection greatly reduced the future costs of foundation construction when the need for additional lanes approach. The alternative was to construct 2 lanes on 2 columns and add a twin bridge at a later date.

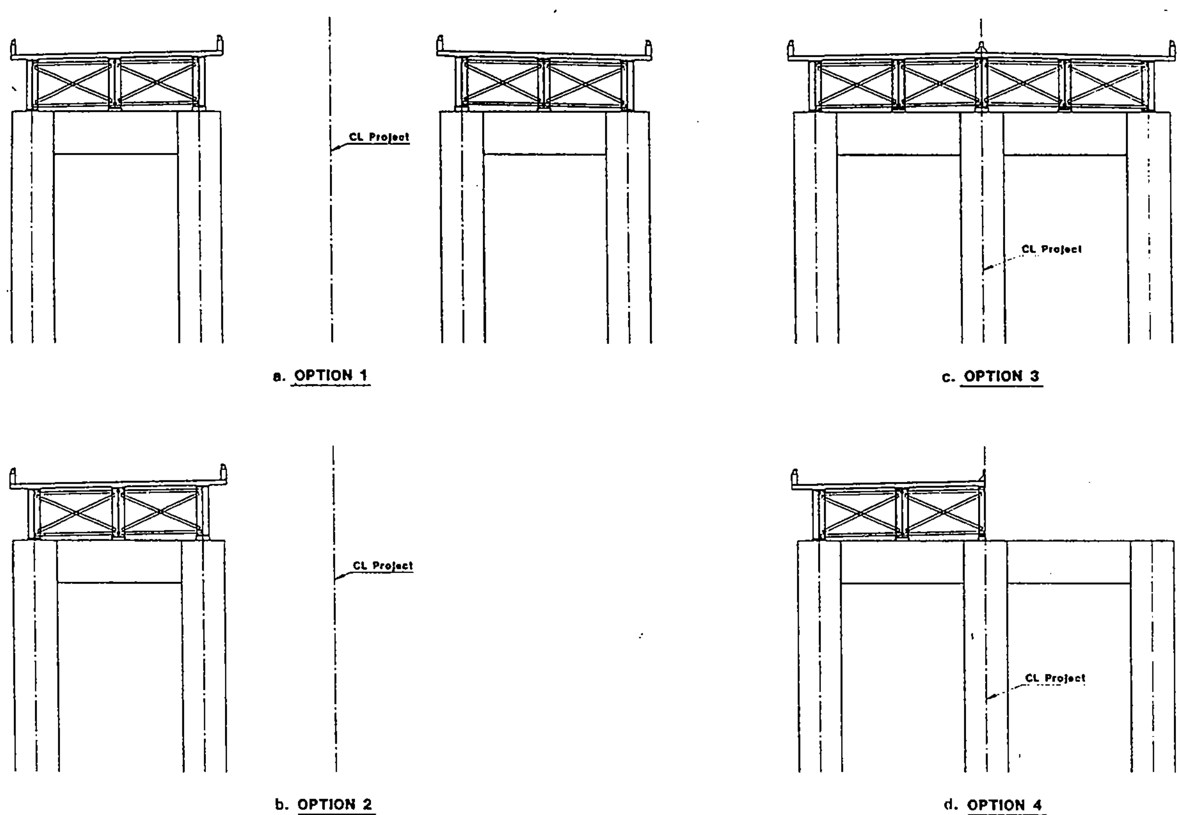

Figure 1 Bridge Type Options

GENERAL SITE GEOLOGY

The bridge is located in the Salem Plateau Section of the Ozark Physiographic Province. The area is characterized by long tapering ridges above narrow valleys. As it is said, "you go down into the mountains of the Ozarks". The Ozark region was gently uplifted from generally Silurian through Cretaceous time. The waterways during this time downcut through the bedrock creating the present landscape. This downcutting has produced bluffs adjacent to the Osage River in excess of 300 feet.

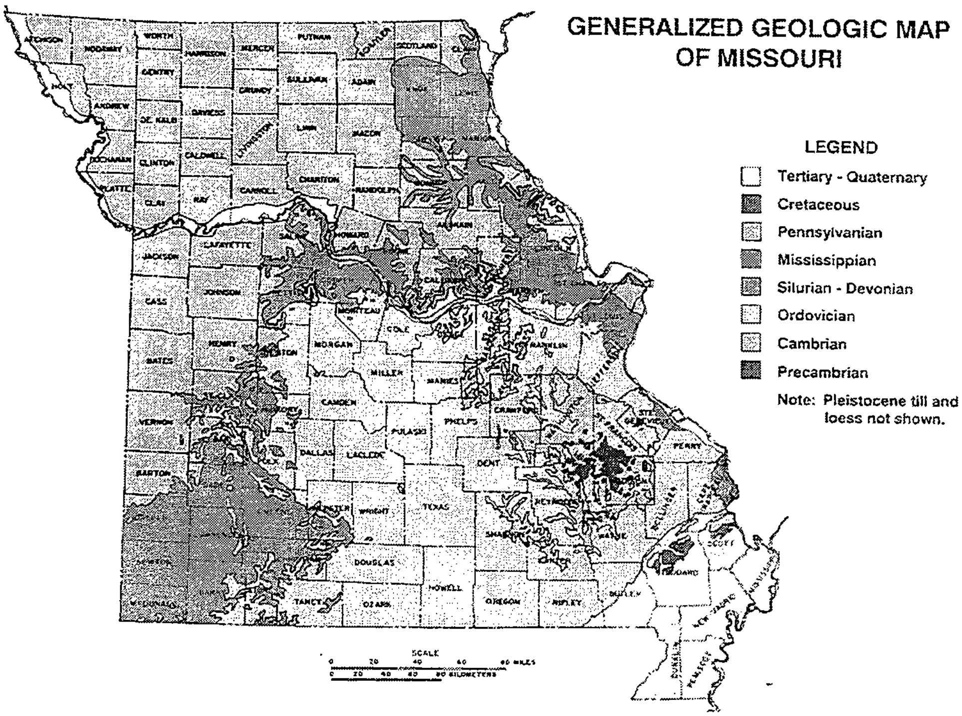

Figure 2 - General Geology of Missouri

The bedrock is characterized by essentially flat-lying horizontally bedded dolomite with lesser amounts of sandstone. The bridge foundations are located in the Gasconade Dolomite Member of the Canadian Series, Ordovician System.

The Gasconade is approximately 250 feet thick in the project area. The Gasconade consists of light to brownish gray to light brown, medium to coarsely crystalline dolomite, finely crystalline dolomite, and cherty dolomite. The upper Gasconade is usually thick to massive bedded, with only occasional chert zones and thin sandstone stringers. The Lower Gasconade consists of medium to finely crystalline thin to medium bedded dolomite and may contain zones where chert makes up over half the mass.

These carbonate rocks are subject to solution, forming features such as widened and filled joints as well as caves. Several commercial caves are located in the area and solutional features are well displayed in the bluffs overlooking the lake. Springs and seeps are also numerous.

Former stresses have also produced fractures in the bedrock, especially the brittle chert. This fracturing is random at the bridge site and as expected, a greater amount of weathering was also noted in the broken areas. Generally, the dolomite beds are horizontal with dips of less than 3 degrees, however local faulting and complex flexure may produce dips of up to 70 degrees.

The overburden section in the uplands and slopes is typical of much of the Ozarks. Residual soils can vary from rock being exposed on the surface to a soil thickness of up to 50 feet. This phenomenon is linked to the variability of the solutioning and weathering of the parent bedrock materials. The thick residual soils typically consist of fat, red-brown clay with various amounts of chert gravel in the clay matrix or as layers. Some thick zones of chert gravel may be encountered as layers and in the composite, chert gravel may make up to 50 percent of the overburden.

Although the physical setting is a man-made reservoir, the site geology is characterized by its location in the Osage River Floodplain. The overburden section in the alluvial plain consists of thin to little overburden on the rock at the former river channel. Otherwise, above the bedrock 10 to 40 feet of alluvial sand with various zones of gravel is encountered. A fairly constant 20-foot thick layer of alluvial clay caps the sand.

Water depths from full reservoir were generally on the order of 40 to 60 feet, with the extreme of 100 feet encountered in the former river channel. The bedrock and land surface slopes gently to the lake from the west but rises on a precipitous, near vertical bluff on the east shore.

SUBSURFACE INVESTIGATIONS

Once the project was approved for design and construction, financing restrictions prohibited payment to design consultants until after the bridge was bid and construction contract awarded. The nature of this payment scheme necessitated a fast track design. Since the most variable and unknown item was the shaft lengths and final founding elevations, the subsurface investigation and geotechnical report became the most time critical item.

Since the bridge would be supported on three columns 60 feet apart on the outside, and the bedrock conditions were thought to be rather constant, it was decided that a boring would be drilled at the two outside shaft locations at each pier leaving the option to drill at the middle pier if differing conditions were found between the two.

As it was desired to complete the borings in a short amount of time, the HNTB Geotechnical Engineering Department required the contractor to use two rigs simultaneously. A drilling contract was awarded to Terracon Consultants Inc., of Lenexa, Kansas. Terracon provided two truck mounted CME 75 drill rigs on a floating plant consisting of sectional barges. The barges were configured in a platform measuring 60 by 80 feet, the 60 feet being the same distance as the outside piers. The barge was secured on location using 4 - 1000 pound anchors attached by wire rope to deck mounted winches. A tug boat supported operations on the barge and moved the anchors. Boring locations were established using Total Station surveying instruments from known points on the shoreline. Boring locations were achieved to within tenths of feet of the desired locations.

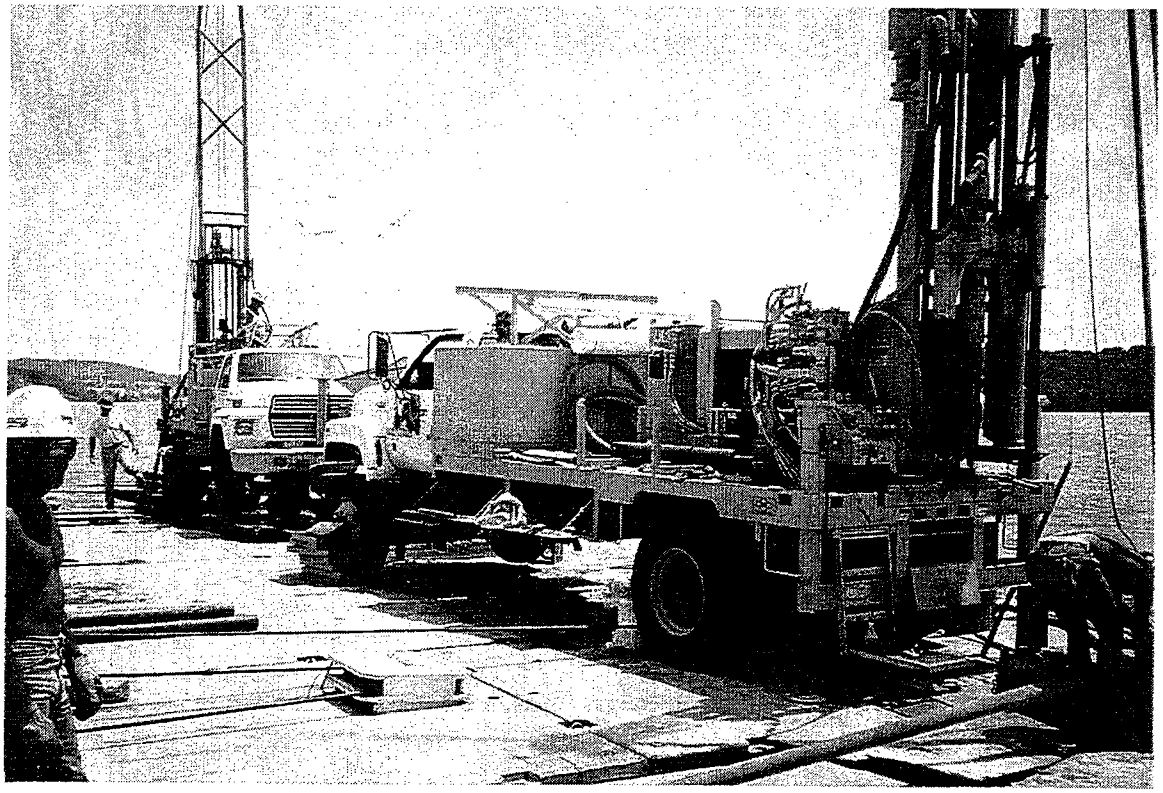

Figure 3 - Subsurface Explorations

The borings were advanced through the water and overburden using flush joint casing. Sampling of the overburden was accomplished by Standard Penetration Tests performed at 5 foot intervals. The casing was then seated into bedrock and cleaned. Continuous coring of the bedrock was then accomplished using NX double tube core barrels. In general, 40 feet of core was taken at each location.

All the drilling was observed and logged by geologists from HNTB. Due to the critical time involved, copies of the logs were faxed from the field immediately after completion allowing for simultaneous design of the drilled shafts. Drilling of the nine water piers was accomplished in 20 days. Work was only slowed by boat traffic on weekends which produced overly rough water and excessive barge movement.

BEDROCK CONDITIONS

Boring data and observations made by the HNTB geologists in the field were received along with core photographs. Selected specimens of the rock core were sent to the laboratory for unconfined compressive strength testing. Boring profiles were plotted at each pier and along the bridge length to evaluate the subsurface conditions for the foundation design. Recovery and RQD data indicated that overall the bedrock is generally of fair to good quality. Unconfined compressive strengths on the tested rock core specimens ranged from 206 to 770 tons per square foot (tsf) with an average of about 508 tsf.

DESIGN OF DRILLED SHAFT FOUNDATIONS

Foundations supporting the 8-foot columns were studied for various sizes and lengths of rock sockets to determine the most economical socket that would meet the loading criteria. Vertical loads of 2000 to 4000 kips per column were required to be supported. Design of the rock sockets is based on side friction between the rock and socket concrete. Side friction in the overburden was not considered and end bearing was ignored due to anticipated settlements of the base of the socket being less than 0.5 inches. The upper several feet of broken and weathered rock was also not considered when calculating necessary rock socket lengths for shaft vertical capacity.

Rock sockets varied in length form 20 to 28 feet into rock depending on loading and characteristics of the rock. Allowable design side friction values ranged form 5 to 7.5 kips per square foot (ksf). A value of two thirds of these side friction values was allowed for uplift. Lateral capacity in rock was more than sufficient for the rock socket lengths required for vertical capacity. Shafts and rock sockets were reinforced full length with 15 number 18 bars.

CONSTRUCTION

A contract to construct the bridge was awarded to Ed Kraemer and Sons, Inc., Plain Wisconsin during February 1996. Construction of the drilled shafts for the bridge began during June of 1996. The drilled shaft foundation subcontractor is Case Foundation Co., Inc., Chicago, Illinois. The first drilled shafts constructed were for the two piers on the land approach on the west end of the bridge. These were constructed using traditional methods of excavating the overburden with a soil auger, placing temporary surface casing, and excavating the rock sockets with core barrels, drop bars, and rock augers. An air core barrel was later used to increase production of the rock sockets. After excavation, permanent corrugated casing was installed, reinforcing placed, and concrete poured.

The shafts for the nine water piers were constructed in a much different manner. The contractor proposed a method slightly different than indicated on the plans. The plans called for 8 foot shafts constructed on 8 and ½ foot rock sockets. The contractor proposed and was allowed to construct 9 foot shafts on the same 8 and ½ foot rock sockets. The shafts then tapered from 9 to 8 feet at the junction of the permanent and temporary casing, approximately 20 feet below the full reservoir elevation.

The drill rig consisted of a crane-mounted attachment on floating barges. A floating template to aid in the location and construction was also assembled into which the casing would be located and held.

Construction consisted of attaching permanent casing to split temporary casing. The attachment was accomplished with a factory fabricated flange on top of the permanent casing, with a similar flange on the bottom of the temporary split casing. The two were joined with bolts through the flange. A pilot hole was then augured into the overburden into which the casing was placed and drilled a minimum length into rock. The casing had carbide teeth attached to the bottom cutting edge to facilitate the advancement into rock. The seating into rock was necessary to seal the socket from sloughing of the overburden and provide for stability of the long, unsupported casing lengths.

After the casing was advanced sufficiently for stability, augers were used to clean as much of the remaining overburden as possible. A full face rotary cutting bit weighing 60 tons was then placed in the shaft to excavate the rock socket. The bit advances the socket with grinding of the rock. The cuttings are removed by reverse air action as air is pumped down the drilling stem and the cuttings are evacuated up and through a discharge hose.

Advancement of the 8 and 1/2 foot diameter rock socket usually proceeded at the rate of 1 foot per hour. After the sockets were advanced to the proposed depths, they were cleaned using air-lifting methods.

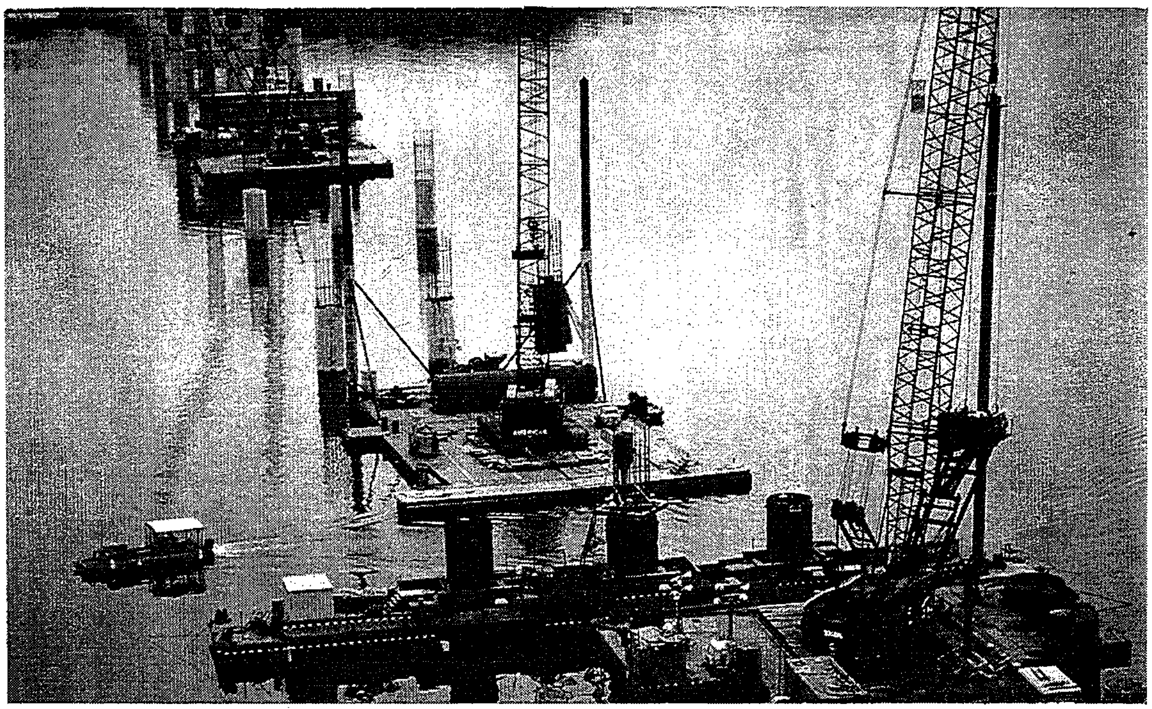

Figure 4 - Drilled Shaft Construction

INSPECTION

After cleaning, the shafts are allowed to settle for about 24 hours to facilitate clearing of the water. Once desirable water clarity was achieved, the casing and rocket sockets were inspected using a remotely controlled, color, underwater camera unit as prescribed in the project special provisions. This video inspection enabled the confirmation of adequate bearing values as desired by the design. The video inspection also allowed for the detection of cavities or voids (none were found). This method also produced timely inspections enabling the contractor to continue without delay, or the necessity of having a diver inspect the shaft. This procedure also allows others such as the geotechnical engineer and geologist to view the founding socket instead of only the diver.

The camera used was a color Simrad Osprey with full pan and tilt. The camera unit was mounted on a frame which was rotated about the shaft and raised and lowered utilizing a small electric winch. A high resolution VHS recorded tape was also produced and archived.

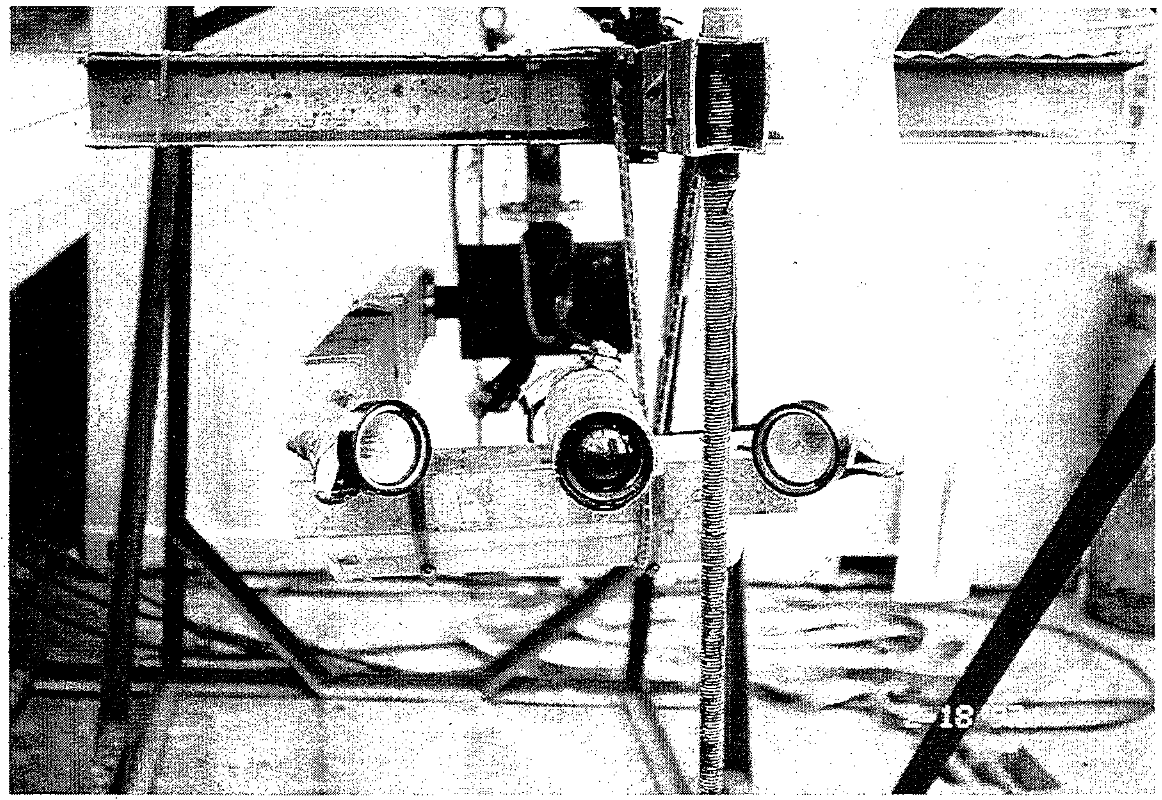

Figure 5 - Camera Used For Underwater Inspection

Since all concrete was to be placed by underwater tremie, a means of inspecting the integrity of the concrete shaft and socket was desired. This was accomplished using a crosshole sonic logging method. Four, inch and one half diameter iron pipes were installed the continuous length of the reinforcing cages, spaced equidistant around the circumference. The pipes were sealed shut at both the top and bottom.

Concrete was then placed in the shafts under water with the tremie placed at the bottom of the socket. A positive head of concrete was always maintained. After the concrete cured to desired strengths, usually 7 days, the shafts were sonic tested.

Since the inspection was the responsibility of the contractor, Case Inc. enlisted STS Consultants of Chicago, Illinois to perform this portion of the work. The crosshole sonic log method is a down-hole Ultrasonic Pulse Velocity (UPV) test. The UPV through concrete is a function of the density and modulus of the material, and can therefore be used to assess material quality. If a series of measurements are made at uniform spacing and over a uniform path length, they can be plotted graphically to give a rapid visual assessment of material uniformity.

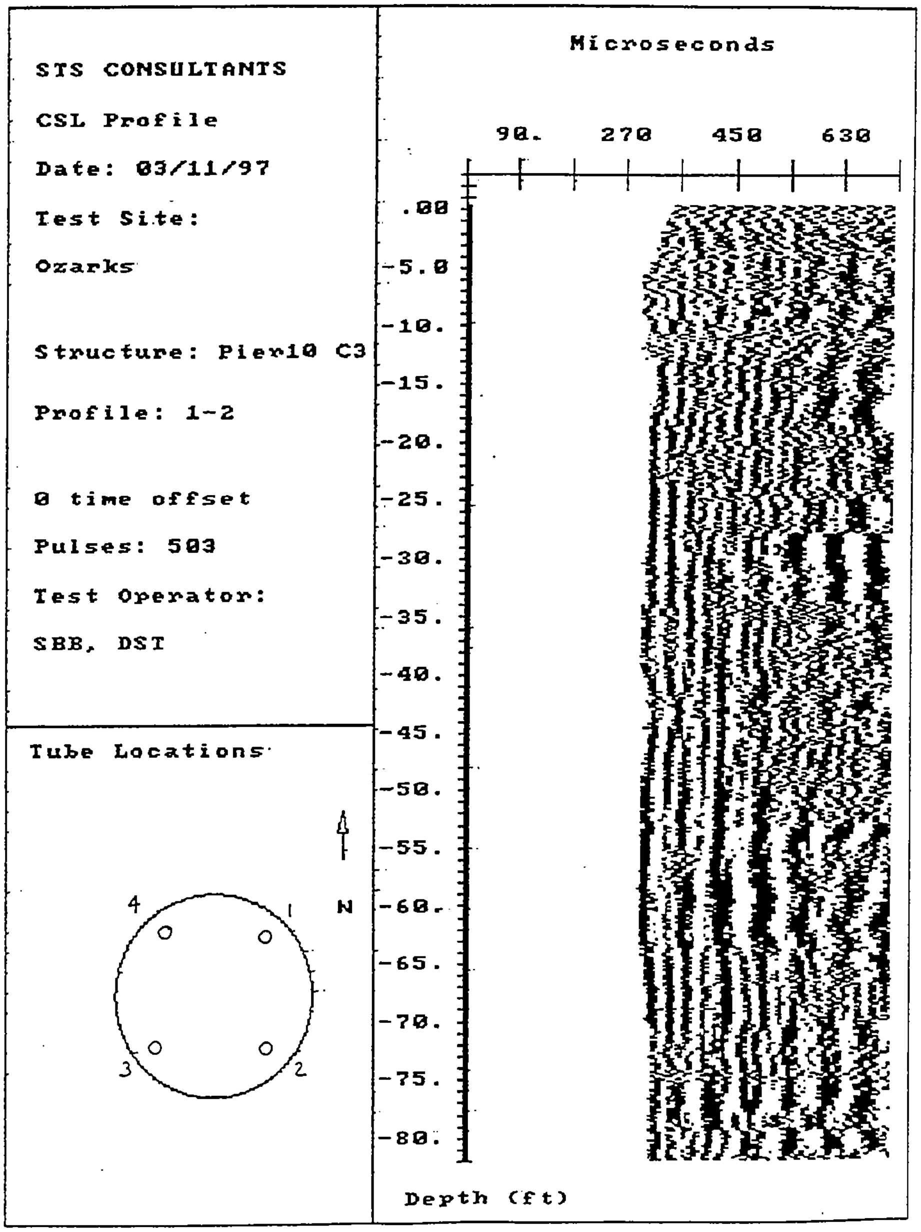

Figure 6 - Cross-hole Sonic Log

CONCLUSION

The project is proceeding ahead of construction schedule and on budget. The construction of drilled shaft foundations socketed into rock enhanced the economics and constructibility of a plate girder bridge in moderate to deep water. The contractor has successfully completed the shafts according to design and plan parameters without any claims of changed conditions or delays. The single column supported by single drilled shaft was the most economic, constructable foundation.

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Resistivity Field Testing of Mechanically Stabilized Earth Structures

by

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Abstract

In the fall of 1996, the New York State Department of Transportation completed an initial field resistivity survey of twelve Mechanically Stabilized Earth (MSE) walls at ten separate locations within New York State. This testing was part of a pilot corrosion study to assess the legitimacy of field measuring the resistivity of the contained "select fill" within these metal reinforced structures.

Resistivity measurements were collected using the Vertical Electrical Sounding (VES) technique and a Wenner electrode arrangement. When possible, three separate and unique resistivity sounding lines were completed at each MSE structure location to characterize the reinforced structure fill, the non-reinforced site fill, and the native soil. The thirty five resistivity survey lines completed during this study were then modeled with an automatic processing and interpretation program by Bisdorf and Zohdy (USGS Open-File Report 90-211 A&B). The modeling results from the three unique sounding line locations were compared so that gross differences could be observed.

In the interpreted data a wide range of values was observed within the reinforced select fill. The native material tested showed the least variability in resistivity values. Resistivity at all locations usually decreased in both range and amount as the measured depth increased. This was especially evident in the reinforced select fill, possibly an indication of the conductive effects from the reinforcing material, soil moisture, or both. It is not evident from this survey what effects the metallic reinforcements had on the results.

Introduction

The Engineering Geology Section of the Geotechnical Engineering Bureau, of the New York State Department of Transportation, has completed an initial field resistivity survey of twelve MSE walls (Table 1). This field resistivity survey was initiated to determine if representative values of earth resistivity within the contained reinforced fill could be assessed.

Location of Site	Number of Walls
Rte. 146A Clifton Park	2
Port of Rensselaer Access Rd. - Rensselaer	1
Rte. 291 over Conrail (Marcy & Whitestone)	1
Rte. 34 & 96 over Conrail - Ithaca	1
Corning By-Pass (Structure #11)	1
Rte. 17C over Conrail - Waverly	1
East Water St. over Conrail - Elmira	1
Bronx River Parkway over Metro North, North Castle	1
Derby Rd. over Conrail - Wallkill	2
Oriskany - River St. over Conrail	1

Table 1. TESTED MSE WALLS

The MSE walls tested consist of granular select fill, metallic strip or grid reinforcements, and concrete facing panels (Figure 1). The friction between the select fill material and the reinforcements allows the fill mass to effectively support both itself and a vertical concrete wall section.

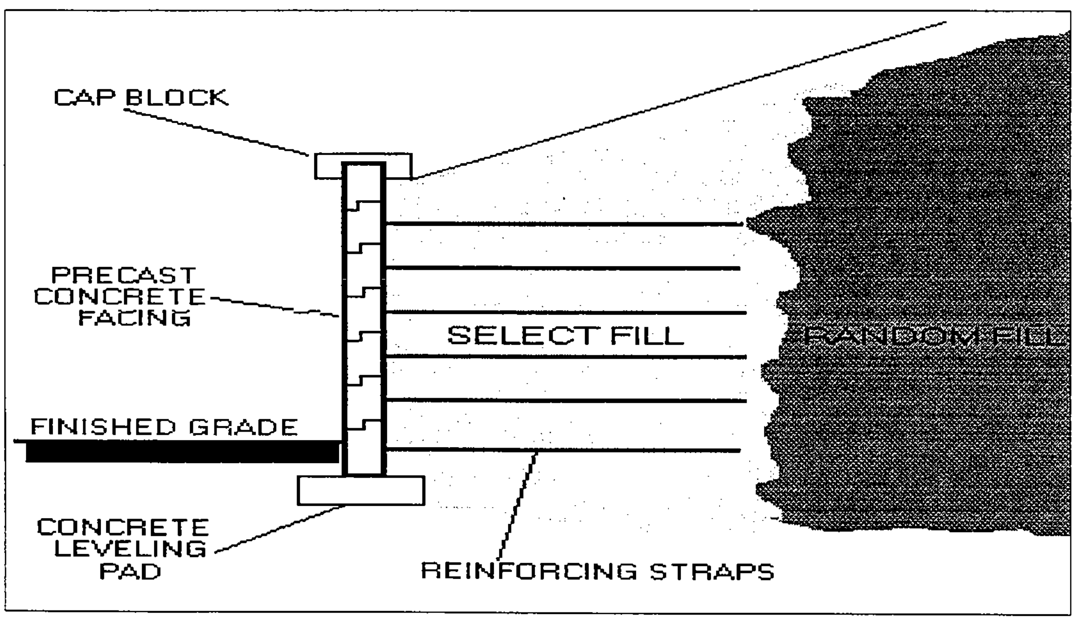

Figure 1. Mechanically Stabilized Earth Wall

Background

The New York State Department of Transportation (NYSDOT) began utilizing MSE structures in 1979. Since that time, over 200 MSE structures have been constructed within New York State. These structures represent three different MSE systems and an evolution of nine different construction specifications, with each new specification reflecting the latest thinking nationally for material and construction control.

In 1987, The Geotechnical Engineering Bureau (GEB) became aware of an MSE structure that had been constructed using backfill material within the reinforced volume that was outside of the specification limits currently used for chemical control of the backfill. Severe corrosion of the reinforcing material occurred and resulted in the removal and replacement of the structure in 1988. This situation and its resolution made the GEB aware that, despite efforts at applying specification controls, corrosion did occur and that the NYSDOT was potentially vulnerable to safety and performance issues concerning MSE structures. It was soon thereafter that the GEB undertook a concerted effort to institute much tighter controls on the backfill materials being used for MSE structures and to begin investigating the effects of corrosion and its potential impact on the structures that NYSDOT owns.

In 1996, the GEB initiated a pilot project to ascertain the effects of corrosion on the reinforcements used in MSE structures. Background information, in the form of laboratory and field resistivity data, was to be obtained with the intent of comparing and evaluating this information so that several candidate structures could be chosen for further evaluation through nondestructive corrosion testing of the reinforcements with equipment currently being developed.

Resistivity

The Earth Electrical Resistivity method of subsurface exploration distinguishes the resistive qualities of earth materials. The flow of current within the earth is a consequence of its conductive, or conversely, resistive qualities. The unit of resistance used in this study is the ohm-centimeter, which describes the impedance of 1 ampere of current across a block of material (1 cm^3) when a potential of 1 volt is applied across opposite faces of the block.

When electrical energy is conducted between two current probes, current flows through the interior soil mass. The distance between the current probes affects the depth of penetration of current flow. A large probe spacing will usually deepen current penetration and the gross resistance becomes increasingly influenced by the deeper subsurface materials. When field measurements are adjusted for probe spacing, an "apparent resistivity value" is achieved. The apparent resistivities must be interpreted in some way to allow "true" resistivity values to be assigned to subsurface materials.

Earth materials conduct electrical energy almost entirely as a result of their water content. Resistance within a soil mass is also a consequence of a soil's porosity, composition, and the ionic qualities of the contained fluid. Overall, saturated soils will therefore be less resistive while less saturated soils will be more resistive.

Several conditions within a MSE structure may influence the evaluation of the select fill resistivity. The effects of conductive, metal reinforcing materials within a MSE structure may obstruct an attempt to distinguish fill material characteristics. The degree of compaction of the select fill may also influence the resistivity characteristics of the structure fill and the measurements obtained.

Field Resistivity Test Method

All measurements of apparent resistivity made during this survey were in accordance with the ASTM G57-95a "Standard Test Method for Field Measurement of Soil Resistivity". The Wenner four probe arrangement consists of four equally spaced metallic probes arranged in a straight line (Figure 2.). The metallic probes were inserted into the soil to a depth of about 15 centimeters. Electrical current is introduced between the two outer current probes while measuring the resulting resistance between the two inner probes. The measuring process is repeated as the probe spacing is increased incrementally.

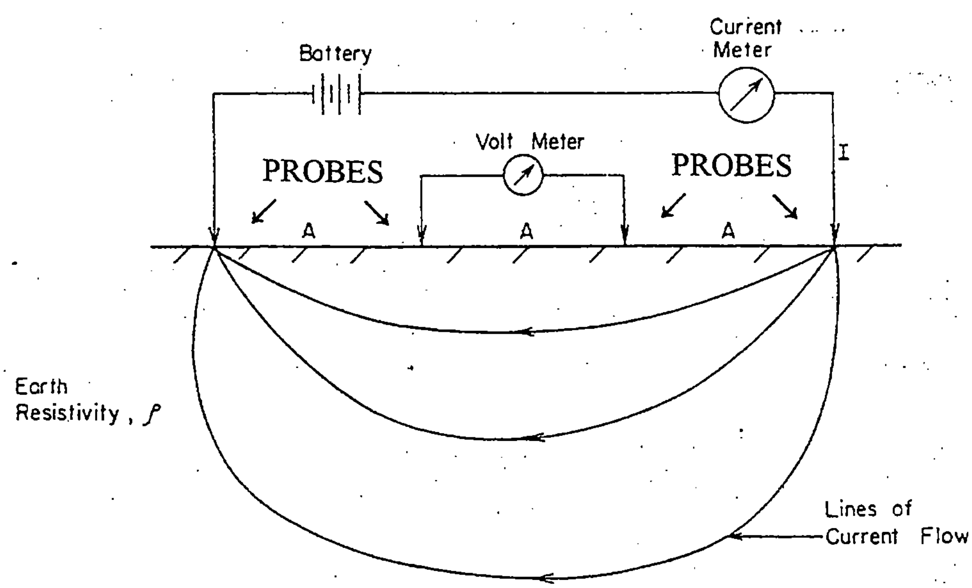

Figure 2. Earth Resistivity

The use of separate probe spacing intervals from a common center point is the Vertical Electrical Sounding (VES) technique. All field measurements of earth resistivity during this study were made using a Bison Earth Resistivity Meter (Model 2350A). This instrument introduces an alternating current into the earth of about 20 milliamperes and 540 volts. A minimum probe spacing of 0.5 meters was used and increased in increments to a maximum spacing (usually 10 meters) based upon the available wall length at each site.

After evaluating several resistivity sounding lines from the first site tested (Clifton Park - Rte. 146A), a test procedure was established. The procedure consisted of three representative survey lines, where possible, to distinguish the resistivity characteristics at each site. A representative test site is displayed below showing the separate locations of the three resistivity lines (Figure 3).

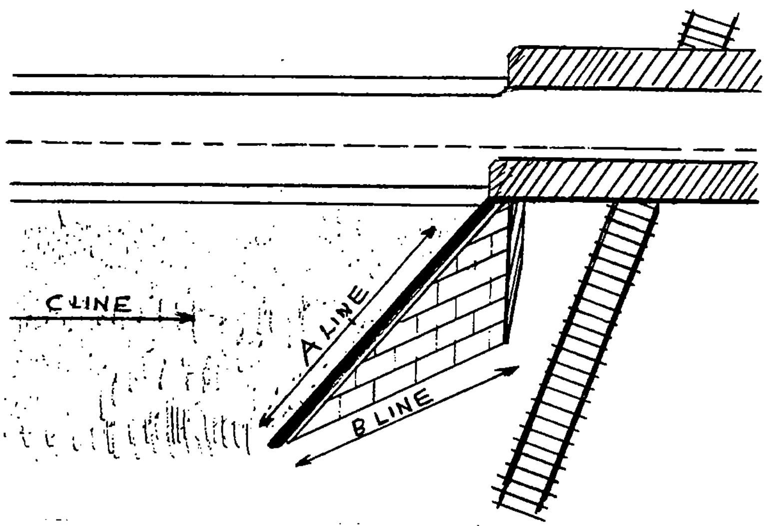

Figure 3. Survey Line Locations (A, B, and C) at a Representative MSES site

1. The A Line, the primary line of importance, is located to attempt to measure the resistivity of the select fill contained within the structure. The line was typically located 2.5 meters up slope and parallel to the MSES wall. It was oriented perpendicular to the reinforcing material to minimize the conductive effects of the metallic reinforcement contained within the select fill. The center point of the A Line is located near the center of the wall length.

2. The B Line is used to measure the resistivity beneath the MSE structure and is located over an area considered native material. The line is usually oriented parallel to the base of the wall, offset one to three meters from the wall face. Occasionally the B Line could not be located at the wall base and therefore was placed in another region of assumed native material.

3. The C Line is located adjacent to the reinforced area in a region of non-select and unreinforced fill. This line is positioned on fill material and is intended to differentiate the select, reinforced fill material from the non-select, unreinforced material.

Determining True Earth Resistivities

An automatic interpretation computer program by R. Bisdorf and A. Zohdy (USGS Open-File Report 90-211 A&B) was used in this study. The program interprets apparent resistivity curves, creating a multilayered subsurface model of resistivity values with their corresponding depths.

Offset Effects

At the first MSES location tested, the Clifton Park Site, several individual A Lines were completed on the two separate structure walls. The three lines on the north wall exhibited similar line form as resistivity values decreased with an increase in offset from the wall (Figure 4). This may be a result of the "boundary condition" that exists due to the close proximity of the structure wall, presumably diminishing as the offset to the wall increases. To standardize the test procedure, and available space, an offset of 2.5 meters was established and used whenever possible.

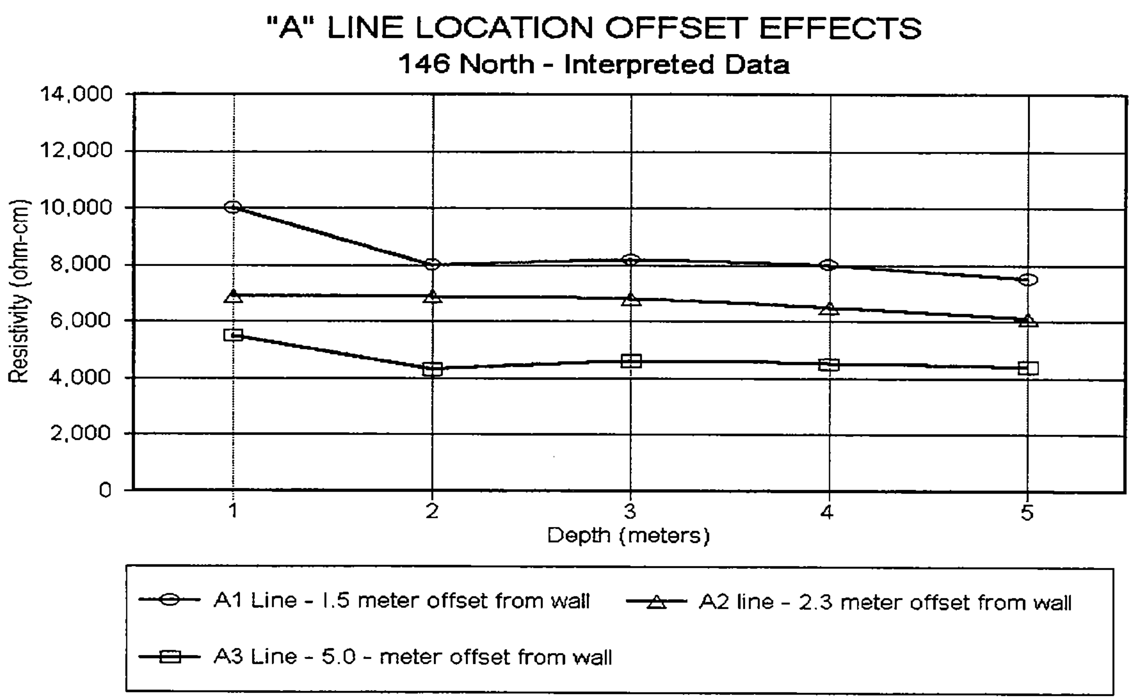

Figure 4. A Line Offset Effects

Rainfall Effects

The potential effect of rainfall and the accompanying increase in fill moisture was explored early in the MSES field survey. This was accomplished by resurveying A Lines at two previously surveyed locations within about 24 hours after a rainfall of approximately 1.5 inches fell within the site area. The resistivity survey lines were accurately relocated at each site, duplicating original survey alignment. The rainfall event did not appear to significantly affect resistivity results at either location (Figure 5 and Figure 6).

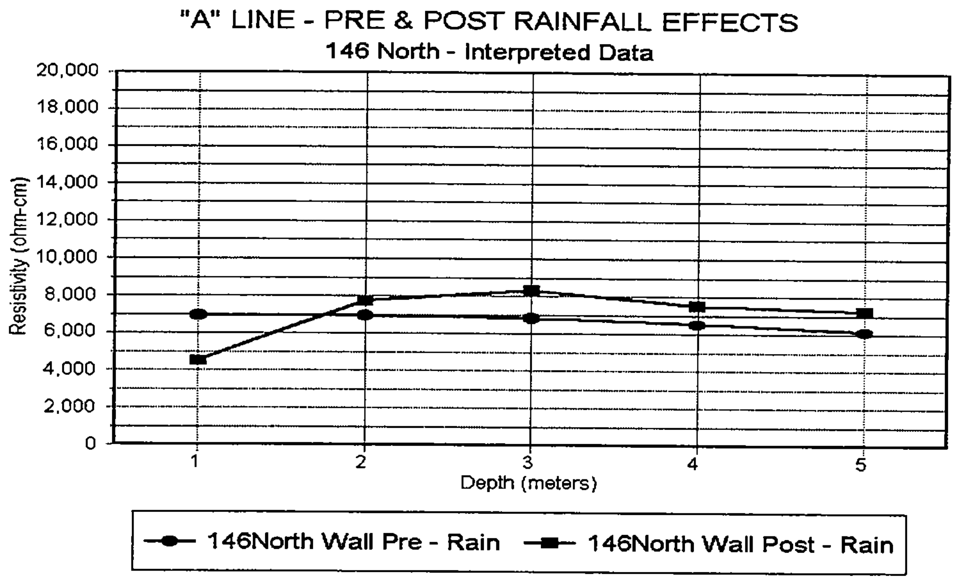

Figure 5. Pre & Post Rainfall Effects (146 North -Clifton Park)

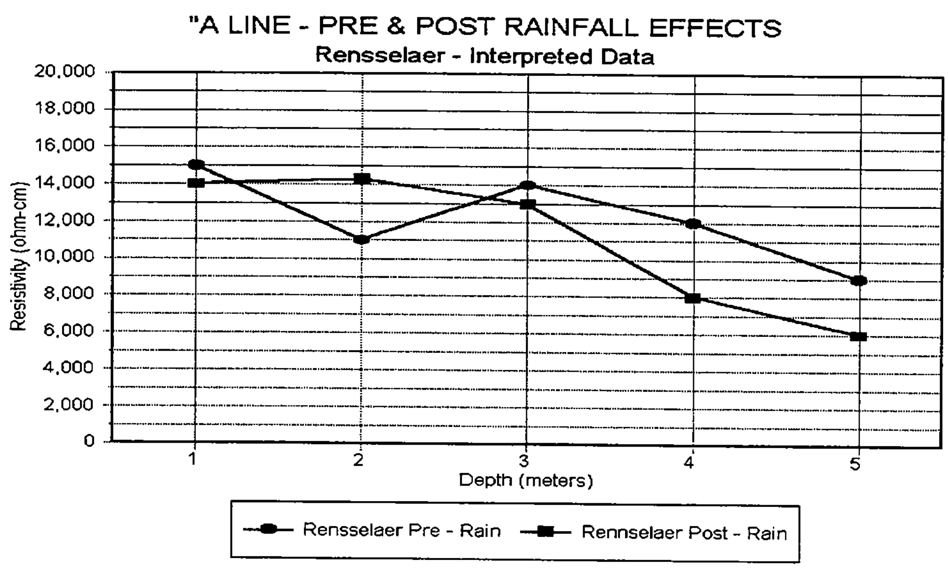

Figure 6. Pre & Post Rainfall Effects (Rensselaer Site)

Results

Resistivity values derived from the automatic interpretation program are presented in Figures 7, 8, and 9 for each of the three separate Line locations (A, B, & C).

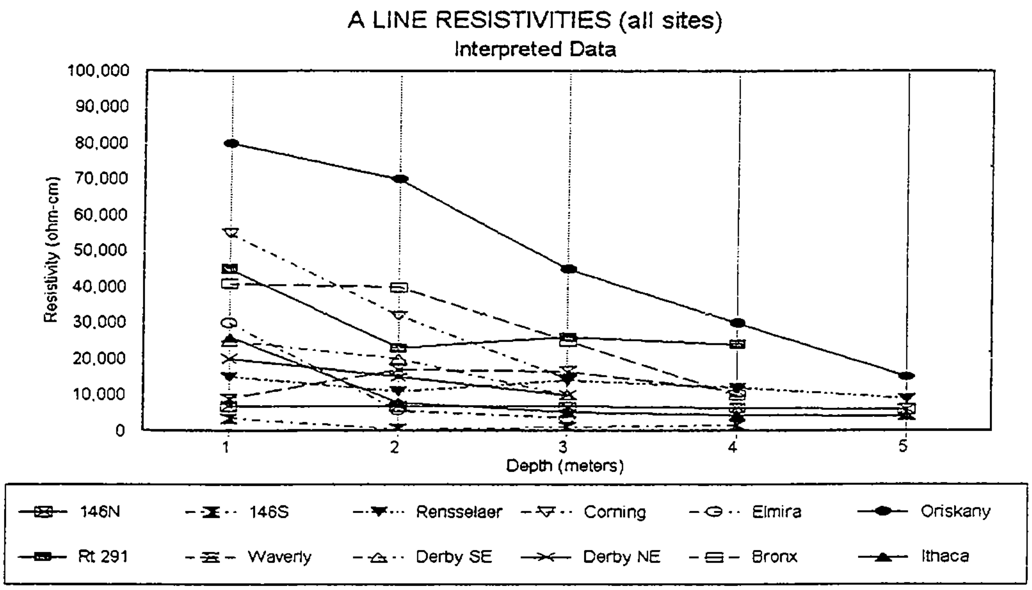

Figure 7. Interpreted Resistivities for A Line Locations

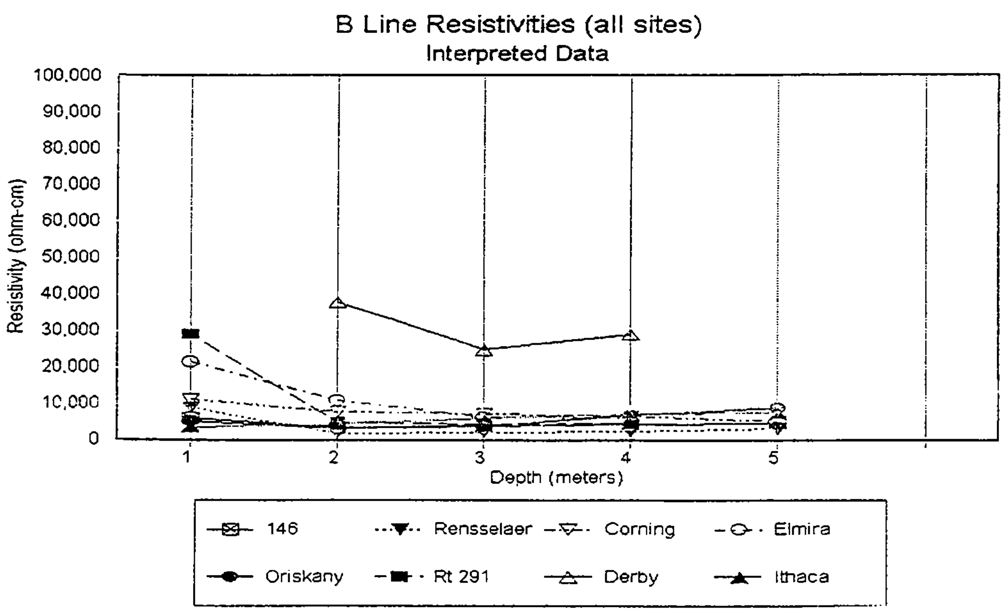

Figure 8. Interpreted Resisitivities for B Line Locations

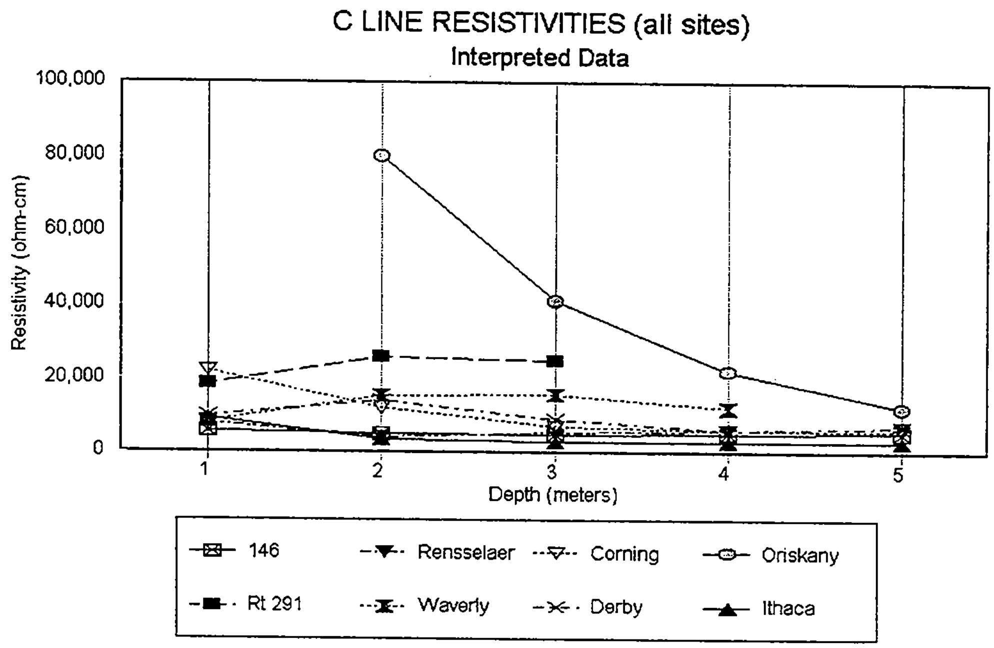

Figure 9. Interpreted Resistivities for C Line Locations

Conclusions

The widest range of resistivity values were shown within the A Line survey results, especially at shallower depths (figure 7.). Typically all of the three separate survey line locations (A, B, & C) show resistivity values decreasing as depth increased. This is probably a consequence of increasing moisture levels within the deeper subsurface material - but this trend may also be a consequence of the reinforcing material which would also affect the A Line results.

A much narrower range of values was shown at the B Line locations at most site locations. This is probably due to similar moisture conditions at the B Line survey locations. The effects of a contacted zone of saturation beneath the B Line locations probably did much to produce similar results at the B Line survey locations.

The C Line survey location results shows a range of values somewhere between the A Line results and the B Line results. Unfortunately, the differences in results from the A Lines and the C Lines can't be directly compared due to their differences in both soil characteristics (random vs. select) and compaction rates.

It would probably be necessary to physically sample the select fill (preferably at several depths) to adequately evaluate select fill resistivity values. The effects of the contained metallic and conductive material, as well as varying moisture conditions within the MSE structures create a difficult situation for the application of field resistivity testing.

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Potential Effects of Superpave Implementation on the Arkansas Aggregate Industry

by

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ABSTRACT

Since the 1950's Arkansas and many other states have designed hot-mix asphalt concrete (HMAC) mixes using the Marshall mix design procedure. Aggregate suppliers in these states have taken an active role in shaping specifications for using quality aggregates in asphalt concrete. The long experience with Marshall mix design and corresponding aggregate specifications has afforded producers and suppliers the relative luxury of working with "known" quantities and quality of aggregate materials.

In response to an increasing number of premature flexible pavement failures, the Federal Highway Administration and other agencies sponsored the development of a new mixture design procedure for hot-mix asphalt concrete. The new mix design procedure is termed "Superpave", for **Sup**erior **Per**forming Asphalt **Pave**ments. Superpave retained many standard quality tests for aggregates to be used in HMAC, but also includes some additional tests. Superpave also changes the methods used by most Marshall mix design states in specifying HMAC aggregate gradation. The Arkansas State Highway and Transportation Department (AHTD) is working toward implementing the Superpave mixture design system. Research is ongoing at the University of Arkansas to determine some potential effects of Superpave implementation on the paving contracting and materials industries in Arkansas.

Aggregates were sampled from existing stockpiles at four HMAC plants in Arkansas to determine compliance with Superpave aggregate and mixture specifications. Results to date suggest that current aggregates should meet Superpave quality requirements. However Superpave gradation specifications, which are significantly different from current Arkansas (Marshall-based) specifications, may change the relative amounts and types of materials used in Arkansas hot-mix asphalt concrete. Specifically, results suggest that the total sand-sized particle content of mixes will be reduced and less natural sand will be used relative to manufactured or angular sand. Also, viable Superpave mixes appear easier to achieve using combinations of crushed gravel and crushed stone rather than using either of the materials exclusively. Changes such as these could have significant impacts on mining, crushing, and sizing operations in the state of Arkansas.

Potential Effects of Superpave Implementation on the Arkansas Aggregate Industry

by
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INTRODUCTION

Arkansas and other states have enjoyed a long history of designing hot mix asphalt concrete (HMAC) mixes using the Marshall method of mix design. Over the years HMAC suppliers, contractors, and the Arkansas Highway and Transportation Department (AHTD) have gained a wealth of experience and knowledge of the behavior of various sources of fine and coarse aggregates used for HMAC in the state. Aggregate suppliers have also played an active role in developing quality specifications for aggregates used in asphalt concrete.

In the late 1980's concern was expressed over an alarming number of premature flexible pavement failures experienced across the United States. In response, the Federal Highway Administration (FHWA) sponsored an aggressive research program, part of which was aimed at developing new specifications for asphalt cement and asphalt concrete. The Strategic Highway Research Program (SHRP) spent approximately $55 million from 1988 to 1993 on asphalt-related research. One of the primary products of SHRP was a new asphalt concrete mixture design procedure known as Superpave, or **Su**perior **Per**forming Asphalt **Pave**ments. Superpave represents an effort to move away from the empirical relationships between asphalt mixture design and pavement performance underlying the Marshall mix design method, to a more "performance-based" mixture design procedure, in which material specifications and laboratory mixture design tests would more directly address the potential performance of HMAC in the field.

Superpave places great emphasis on selecting quality materials for asphalt mixtures by including many standard aggregate quality tests and some newer tests directly into the mixture design system. In addition, aggregate gradation for Superpave mixes is markedly different from the gradation requirements currently used by many states, including Arkansas. The University of Arkansas and AHTD are conducting ongoing research efforts aimed at determining the potential effects of implementing the Superpave mixture design system on the aggregate industry in Arkansas. This paper reports on some of the early conclusions drawn from these efforts.

SUPERPAVE - AN OVERVIEW

The Superpave design system strives to be a "performance-based" procedure, in which laboratory tests and material specifications work together to ensure that Superpave mixes will perform adequately in the field with respect to the three primary failure modes of flexible pavements, namely, rutting, fatigue cracking, and low-temperature (thermal) cracking. Mix design requirements and tests under Superpave can range from very basic to quite sophisticated; the more sophisticated the mixture testing, the greater reliability given to the designer that the mix will perform adequately. Basic HMAC design in Superpave, termed *Superpave Volumetric Mix Design*, uses volumetric relationships between aggregates, asphalt cement binder, and air voids in a compacted specimen to determine the "best" aggregate structure using available aggregates and the optimum asphalt binder content for the specific aggregate combination. This first "level" of mix design can (and will be) used as a stand-alone design procedure. However, it also serves as the basis for increased levels of design sophistication and reliability. The second "level" of mix design under Superpave is termed *Superpave Intermediate Mixture Analysis*, and uses new tests developed under SHRP to screen mixes with respect to their performance potential in the field. The third Superpave "level" of design, *Superpave Advanced Mixture Analysis*, extensively tests candidate HMAC mixtures in the lab using sophisticated tests and equipment developed under SHRP. The second and third levels of Superpave mix design remain primarily a research topic for many highway agencies as testing procedures, equipment (which is quite expensive), and data interpretation continue to undergo refinement. Superpave volumetric mix design (the first level) appears to be the standard to which many agencies are turning to design their HMAC mixtures.

Aggregate Quality Specifications in Superpave

The Superpave system includes requirements for selecting quality aggregates for use in HMAC (Cominsky, et al, 1994; Asphalt Institute, 1995). Aggregate property requirements included in Superpave were selected by a survey of top pavement professionals, who were charged with both identifying those aggregate properties most critical to the performance of HMAC in the field, and selecting a specification limit value on those properties. The outcome of this process are two categories of aggregate property specifications. *Aggregate consensus properties* are those properties identified by the pavement professionals as critical to HMAC performance, and for which the professionals could agree (hence "consensus") on specified values for the property in question. *Aggregate source properties* are those properties on which the professionals could agree were critical to HMAC performance, but are very source specific in terms of the actual specified value of the property. The specification limits placed on the source aggregate properties are left to the specifying agency, rather than being specified by Superpave.

Consensus aggregate properties in Superpave include coarse aggregate angularity, fine aggregate angularity, flat & elongated particles, and clay content. Specified values for each property are based on the traffic level expected on the pavement and the location of the aggregate within the pavement. Criteria are intended to be applied to the blend of aggregates making up the skeleton of the HMAC; however, many states apply the criteria to individual aggregate sources to identify undesirable components (Asphalt Institute, 1995). Table 1 shows the consensus aggregate properties and the Superpave recommended test procedures for determining the properties.

Source aggregate properties recommended by Superpave include toughness, soundness, and deleterious materials. Specific values for the properties are determined by the specifying agency. Table 1 lists the source properties and the recommended associated test procedures.

In addition to aggregate material properties, Superpave also specifies aggregate gradation for HMAC. Superpave gradation specifications include the maximum density line, control points, and

Aggregate Consensus Properties		
Coarse Aggregate Angularity	Penn DOT Test Method 621 (Arkansas Test Method 304)	*Determining the amount of crushed particles in gravel*
Fine Aggregate Angularity	AASHTO TP33	*Uncompacted void content in fine aggregate*
Flat and Elongated Particles	ASTM D 4791	
Clay Content	ASTM D 2419	*Sand equivalent test*
Aggregate Source Properties		
Toughness	AASHTO T96	*Los Angeles Abrasion*
Soundness	AASHTO T104	*Sodium sulfate soundness*
Deleterious Materials	AASHTO T112 (Arkansas Test Method 302)	

Table 1. Superpave Aggregate Properties

a "restricted zone" through which the aggregate gradation should not pass. The maximum density line is defined as a straight line from the origin to 100 percent passing the maximum size aggregate. Control points are specified "percent passing" values for critical sieve sizes. The restricted zone is designed to limit the amount of fine sand in relation to the total amount of sand-sized particles in the mix. Research over the years has suggested that HMAC gradations that pass through this region, effectively forming a "hump" in the gradation curve in the sand-sized particle size range, have a tendency to become "tender", that is, those mixes lack the internal shear strength needed to resist permanent (plastic) deformation under the action of construction equipment and to resist rutting while in service.

This approach to specifying gradation may be a departure from traditional gradation specifications typically used in states. Arkansas, for example, has always used gradation bands through which aggregate gradations must pass. In addition, Superpave gradations appear to encourage mixes that show a more coarse gradation compared to traditional mixes in many states. Figure 1 shows the traditional Arkansas Type 1 surface mix gradation compared to Superpave gradation criteria.

ARKANSAS AGGREGATE INVESTIGATION

In 1995 the University of Arkansas initiated an investigation, sponsored by the Arkansas Highway and Transportation Department, to determine some of the potential effects to the highway materials and construction industry of implementing the Superpave mixture design system. One of the focus areas of the study is aggregates. Coarse and fine aggregates were sampled from existing stockpiles at hot mix asphalt plants in four locations around the state of Arkansas. The

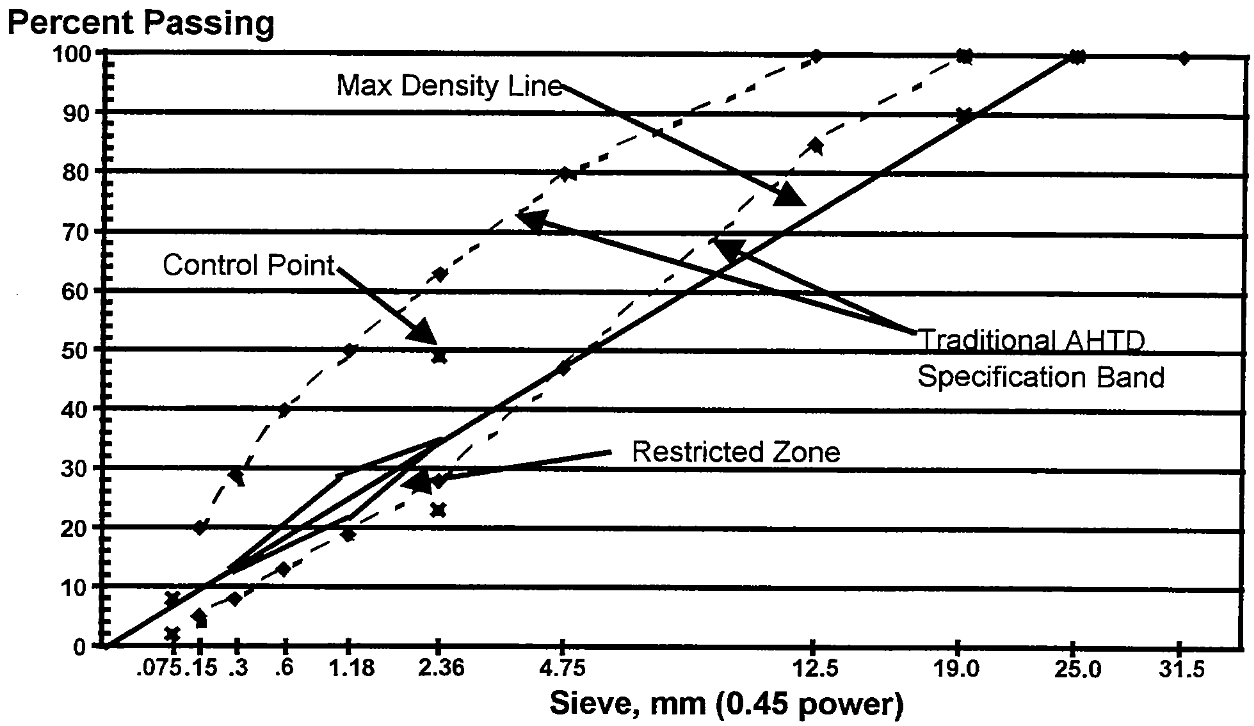

Figure 1. Superpave and Arkansas Aggregate Gradation Criteria

aggregates sampled were all currently being used in approved HMAC mixes designed using traditional Arkansas criteria (e.g. the Marshall method of design). The aggregates represented the major types of aggregate used in Arkansas – crushed limestone, crushed gravel, manufactured sand (screenings), natural field sand, and Arkansas river sand. A small amount of crushed sandstone is used in Arkansas for hot mix asphalt concrete; however, sandstone was not available from the sources sampled.

The primary goal of the aggregate portion of the investigation was to determine the suitability of using existing stockpiled aggregates in Superpave mixes. Aggregates were evaluated in terms of source and consensus properties, and in terms of suitability of use for blending to meet Superpave gradation requirements for HMAC.

Consensus and Source Properties

Although Superpave contains distinct specifications for aggregate properties, many of the properties specified are not new to the aggregate industry. Arkansas currently specifies aggregate quality in terms of coarse aggregate angularity, clay content, and flat & elongated particles --all of which are Superpave consensus properties, as well as toughness, soundness, and deleterious materials -- all specified Superpave source properties. Current Arkansas specification values for these properties are very similar to the values contained in Superpave (AHTD, 1996). Aggregates currently produced in Arkansas should not have a problem meeting these specified

Superpave criteria. The lone aggregate quality test not currently used in Arkansas is the fine aggregate angularity test (a Superpave consensus property).

The fine aggregate angularity test (AASHTO TP-33) is an adaptation of the National Stone Association "flow test" for measuring the uncompacted void content of a fine aggregate. Some studies have indicated this test relates to the angularity and surface texture of fine aggregates, which in turn relates to the performance of a hot mix asphalt concrete (Cross, et al, 1994). However, concern has been expressed that the current test and Superpave criteria may eliminate some sources of fine aggregate with a proven record of performance (Abdo, 1997). Of all the source and consensus properties in Superpave, the fine aggregate angularity test and specification is the most likely candidate for revision.

Fine aggregates sampled for this investigation were tested for angularity using AASHTO TP-33. Table 2 is a partial listing of the results. It is noteworthy that while three individual aggregates shown in Table 2 do not meet Superpave criteria (minimum uncompacted void content of 45%), all aggregate blends in which the aggregates are present could be considered acceptable. This points out the importance of applying the Superpave criteria to HMAC aggregate blends, as intended, rather than to individual aggregate sources in a "screening" process.

Based on the results of the fine aggregate angularity tests and given current AHTD criteria relative to other consensus and source aggregate properties, it is reasonable to conclude that current aggregates in Arkansas should meet aggregate quality requirements in Superpave.

Source	Aggregate I.D.	Uncompacted Void Content (%)
APAC	Limestone Screenings	44.3
(West Memphis)	Field Sand	37.3
	River Sand	43.1
	Superpave 12.5mm blend	46.1
	Superpave 19.0mm blend	44.7
	Superpave 25.0mm blend	47.1
L.J. Ernest	Screenings	46.5
(Texarkana)	Sand	44.4
	Donna Fill	49.9
	Superpave 12.5mm blend	47.7
	Superpave 19.0mm blend	45.6
	Superpave 25.0mm blend	46.9

Table 2. Fine Aggregate Angularity Results

Aggregate Blending / Gradation

The gradation of each stockpiled aggregate sampled was determined and used in blending the aggregates to produce a gradation meeting Superpave criteria. For each combination of aggregates, a minimum of three trial blends were determined: (1) a "fine" blend, in which the gradation curves passes above the restricted zone; (2) a "medium" blend, in which the gradation curve passes below, but very close to the restricted zone and maximum density line; (3) a "coarse" gradation, in which the gradation curve passes well below the maximum density line and the restricted zone. Figure 2 illustrates the aggregate blend gradations. For each blend HMAC specimens were compacted using the Superpave gyratory compactor and tested to determine the volumetric properties air voids (VTM), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA); in addition, the compaction of each specimen was compared to Superpave compaction criteria.

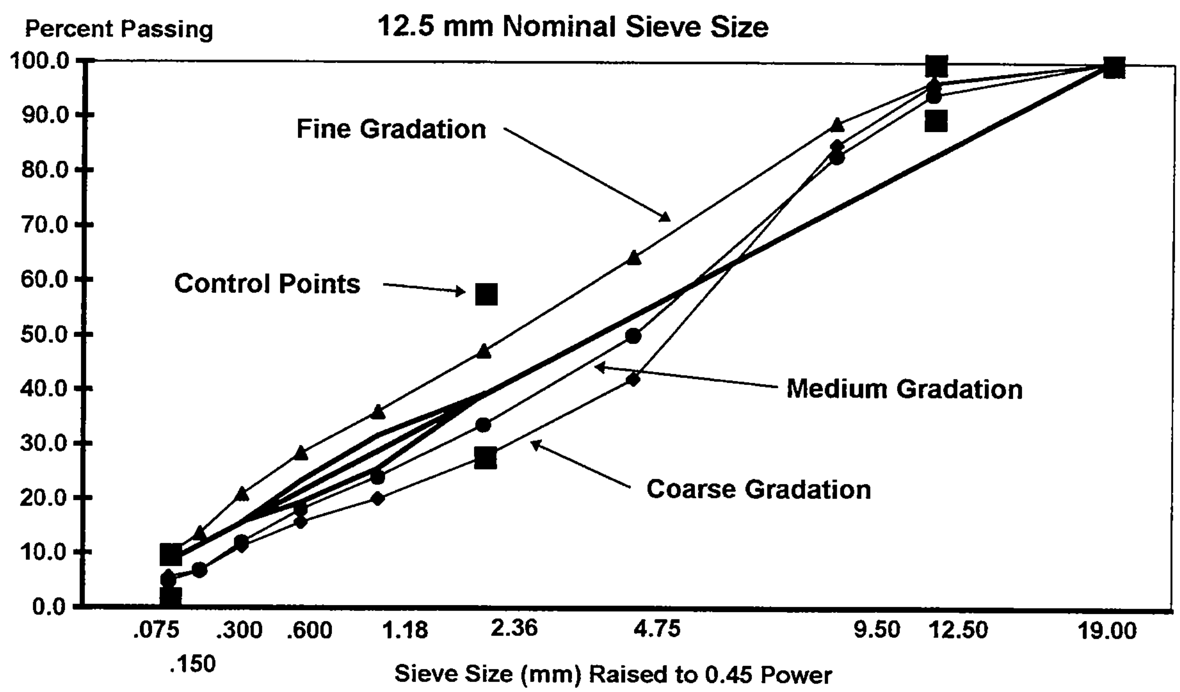

Figure 2. Aggregate Blend Gradations

To date, approximately twenty-seven (27) asphalt/aggregate blends have been evaluated for volumetric and compaction properties using Superpave criteria. Of these, ten (10) blends have proven acceptable. According to anecdotal evidence gathered by talking with others involved with Superpave mix design this success rate (about 37%) is very common. For the Arkansas blends tried, the property most commonly violating Superpave criteria is VMA. This suggests that the mixes must be designed to be more "open", that is, mixes should contain increased percentages of larger stone filled with sufficient fines to bind the mixture into a stable mass. To achieve this type of mixture, the total amount of sand-sized particles, particularly coarse sand, will probably be reduced compared to current practice.

Another interesting observation concerning acceptable blend gradations is that all the blends except one judged acceptable by Superpave had a gradation that passed below the restricted zone. In other words, only one "fine" gradation of the nine attempted was acceptable by Superpave. Should this observation be borne out by continued mix design efforts, mixes in Arkansas will become much more coarse than is the case currently.

CONCLUSIONS

Superpave represents a fundamental change in the design of hot mix asphalt concrete mixes. Studies of the effect of Superpave implementation on current materials and mix design practice in Arkansas suggest the following points with respect to aggregates used in HMAC:

- Current aggregates used in Arkansas should be acceptable for Superpave mix design with respect to quality tests of consensus and source properties.
- Superpave consensus and source properties, particularly fine aggregate angularity, should be measured on the HMAC aggregate blend, as intended, to prevent exclusion of aggregates that might be judged unacceptable when evaluated individually.
- Aggregate gradations used in HMAC designed by Superpave will be more coarse compared to traditional Arkansas mixes; based on studies to date, Superpave gradations will pass below the restricted zone.
- The amount of sand-sized particles in HMAC aggregate blends will be reduced for Superpave mixes compared to traditional Arkansas mixes, especially relatively coarse sands.
- Based on results to date, of the sand-sized particles used in Superpave aggregate blends, most will be manufactured (angular) sand rather than natural (rounded) sand.
- Relative to aggregate type, results to date suggest that acceptable Superpave mixes may be easier to achieve using a combination of crushed stone and crushed gravel, rather that by using either material by itself.

Overall, producers of larger (coarse) aggregates in Arkansas should see little difference with the implementation of Superpave. Producers of sand-sized aggregates, however, may be significantly affected by Superpave implementation, as HMAC mixes become increasingly coarse and the demand for more angular sands increases compared to the demand for natural sand. It is apparent that continued efforts regarding Superpave mix design and the effect of implementation are needed. It is also apparent that the aggregate industry in Arkansas play a role in shaping the focus of the ongoing research efforts.

ACKNOWLEDGMENT

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The Landslide At Waverly Street And Maryland Route 36 A Case History

by

A. David Martin
Maryland State Highway Administration

Abstract

During the early spring of 1991, a landslide occurred in a cut slope on the west side of MD Rt. 36 that closed the shoulder of the roadway. A house located above the top of cut began to break up due to differential movement caused by the slide.

The Maryland State Highway Administration (SHA) conducted a study of the landslide that consisted of borings, instrumentation, a land survey, and literature search.

The finding of the study was that the slide was the re-activation of an ancient slide. The area had been weakened by the roadway cut slope at the toe, and the drainage from collapsed, abandoned coal mines located behind the soil mass. An unusually warm wet winter helped trigger the movement when the area was saturated by surface water that was channeled into it from the haul roads associated with a logging operation.

After relocating the residents and living with the slide for two years, SHA developed a contract for remediation. Some of the coal mines were drained prior to the construction of the repair. This was the first slide repair by SHA where the total slide mass was not removed. Only the base of the slide mass was removed and replaced with a rock buttress. The upper portion of the slide was arrested by the buttress.

Introduction

Maryland Route 36 (MD 36) is a two lane state road that connects the towns of Frostburg and Westernport in Allegany County, Maryland. The valley occupied by the road was created by Georges Creek for which it was named. This valley was once the premier coal mining area of Maryland, and deep mines for the Pittsburgh and other coal seams were made on both sides of it. When deep mining became unprofitable, strip miners moved in and some of these are still in operation.

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In the late 1970's the old road was replaced in the Westernport area with a wider roadway requiring higher cuts and some creek relocation.

The Harry Taylor property is located near the north boundary of the town of Westernport. The house faces east and the property is bounded on the east by the MD 36 right of way which runs northeast-southwest. To the south of the Taylor property are developed properties. To the north and west are undeveloped properties. The property location is identified as to the left of stations 40 to 45 on MD 36.

During the late winter and early spring of 1991, Mr. Taylor noticed that his house was apparently being stressed by ground movement. The foundation was cracking, the structure was emitting sounds, doors became stuck, and the chimney pulled away from the wall.

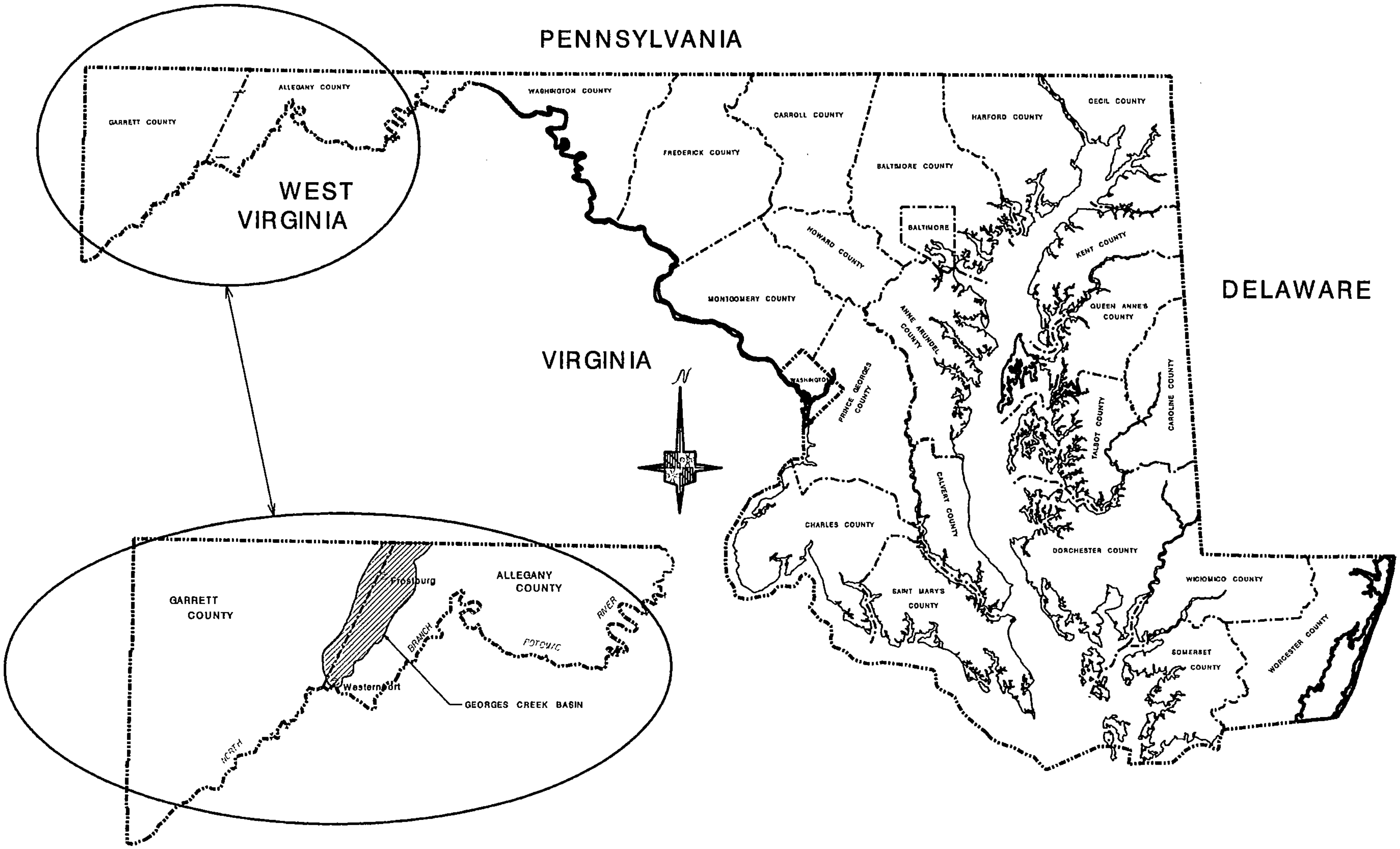
Feeling that past coal mining activities may be the cause of the ground movement, Mr. Taylor brought his problem to the attention of the Maryland State Bureau of Mines. Representatives of that organization met with him on February 21, 1991 and made a field inspection of the problem. Bureau of Mines developed a February 22, 1991 file report that stated that the abandoned coal mines in the area were not a part of the problem. Their report was based on their field observations and research of their mine records that placed the mines above and behind the house. They noted fresh activity in the road cut for MD 36 and assumed that to be the cause. They communicated their conclusions to Mr. Taylor and to the State Highway Administration (SHA). Subsequent investigations by the SHA's Engineering Geology Division and District Survey Team confirmed that the area around the Taylor property was an active slide and the house was or would be damaged beyond repair. The house was determined unfit for habitation, and the Taylor family was relocated at SHA expense.

On February 23, 1991, the District Engineer was verbally advised by the Engineering Geology Division that although the road cut might not be the primary cause of the recent acceleration of the slide, it would be difficult if not impossible to avoid having the cut being considered a contributing factor to the slide. An investigation of the slide and its causes was initiated.

Geology

The town of Westernport is located at the southern most end of the Georges Creek Basin, a northeast trending valley that roughly parallels the Allegany/Garrett County line. The valley follows the axis of a syncline that plunges gently to the northeast. It was cut by Georges Creek which flows in a southwesterly direction to the Potomac where it enters that river at Westernport. The creek meanders as it follows the axis of the syncline so that in the area of the Taylor property the axis of the syncline is about 2000' west of Georges Creek. Dip of the rock formations are from 1 to 1½ degrees to the northwest. The Taylor property is near mine entrances that are about 1500' east of the syncline axis. MD 36 is immediately to the east of the Taylor property, between it and Georges Creek. See Figure 1.

The rock formations in the slide area are mapped as the Monongahela Formation, the Conemaugh Formation, and the Allegany-Pottsville Formation. The highest peaks in the area are capped by the Monongahela Formation. The Conemaugh Formation forms the slopes and the Allegany-Pottsville Formation lies in the valleys. The bottom of the Conemaugh Formation is marked by the presence of a coal seam that is named "Upper Freeport" in current geologic publications. Earlier



MARYLAND STATE HIGHWAY ADMINISTRATION

FIGURE 1

publications, especially those developed in other states, refer to this coal as the Davis coal, Split Six coal, or Lower Kittanning coal.

Geologic Sections developed from borings made in other parts of the Georges Creek Basin show the Upper Freeport to be bounded on the top by the Lower Mahoning sandstone and on the bottom by the Bolivar fire clay. The Bolivar is absent in many areas and is not known to be present in the Taylor slide area, in which case the Upper Freeport coal would rest directly on a sandstone which also bears the name "Upper Freeport".

Ground water flow in the slide area is largely controlled by the presence of the mines and the under clays normally associated with the coal seams. In the slide area, the outcrops of the formations on the east flank of the syncline do not rise to the elevations enjoyed by these same formations on the west flank so the flow of water through the mines and coal seams is towards the east flank. Due to the higher recharge area of the west flank, water flows up the slope of the bedding of the east flank and exits at the mine openings that are located on the outcrop face.

Current MD 36 was built in 1979. The openings of the abandoned mines on the north perimeter of the Taylor slide were noted on the 1977 design plans for this project. Drainage from these openings is visible on aerial photographs dated 1970.

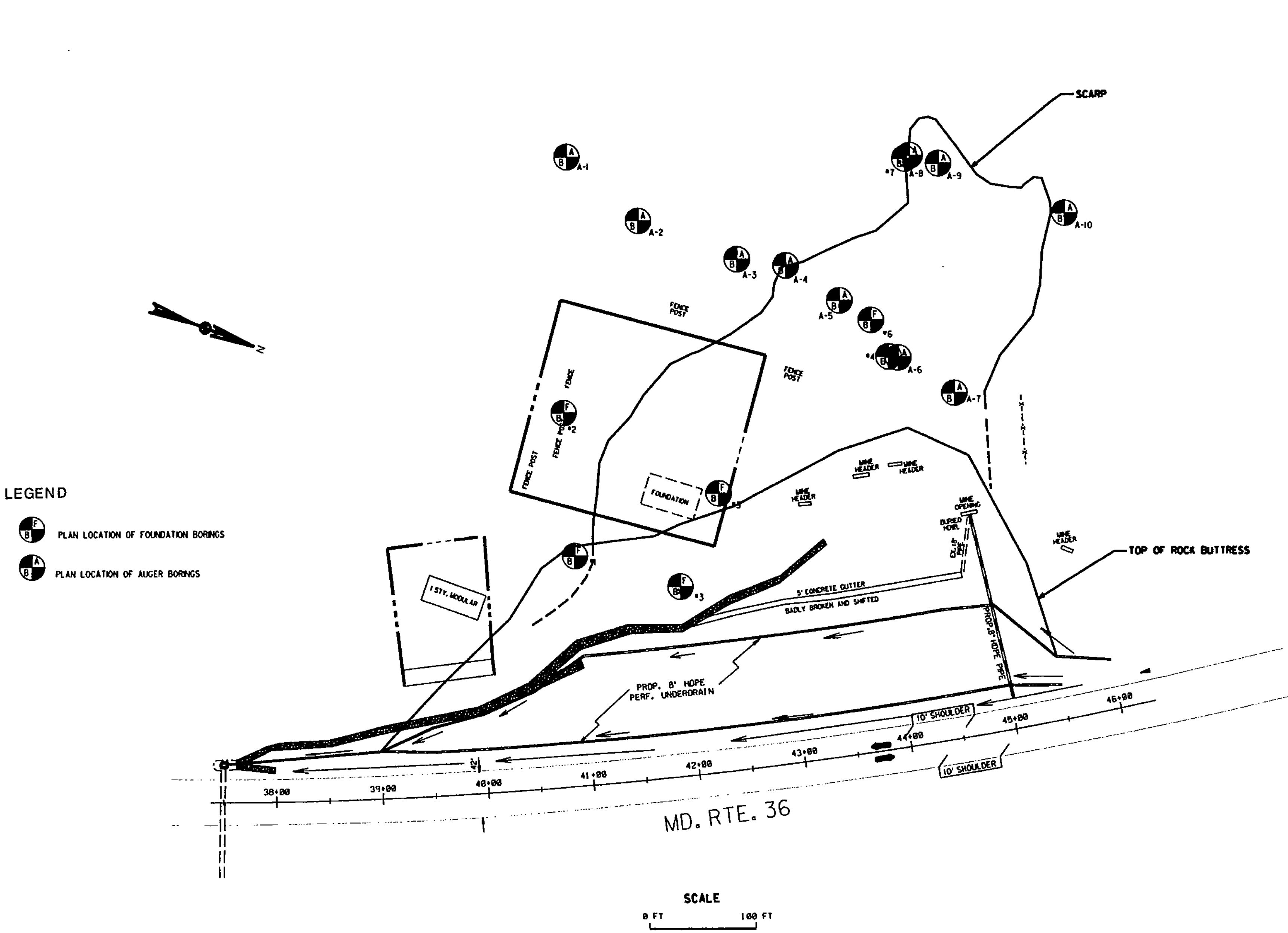
Stability problems with the slope face between stations 40 and 45 were evident during construction. Corrective action in the form of slope flattening and mine drainage control was made at that time, and side ditch clean up and slope regrading has been needed every few years since the road opened.

Phase I Investigation

The objective of the initial phase of the slide investigation was to determine if adjacent properties or MD 36 were endangered by the slide. In April, 1991, two slope inclinometers were placed at what was thought to be the southern perimeter of the slide between the Taylor residence and the neighboring houses to the southwest. A third inclinometer was placed in front of the house, between it and MD 36. Also, a series of auger borings was made in the distressed area to the west, behind the house. See Figure 2

Data readings were taken from the 3 inclinometer wells at biweekly intervals from April to July. The readings of the 2 southern wells showed that the southern limit of the slide was somewhere between the Taylor house and the southern boundary of his property. Continued readings at monthly intervals through the summer of 1991 showed that the 2 adjacent houses were not in danger of being affected by the slide. The inclinometer placed between the Taylor house and MD 36 sheared within several weeks of installation. The few readings that were obtained showed movement in a southeasterly direction at an elevation a few feet above the elevation of the nearest point of MD 36. Although this movement was rapid and dramatic, it did not appear to the investigators that MD 36 was in imminent danger at that time. It was decided to continue to make visual and survey observations of the slide through the rest of the year.

FIGURE 2



SCARP
TOP OF ROCK BUTTRESS
A-1
A-2
A-3
A-4
A-5
A-6
A-7
A-8
A-9
A-10
#2
#3
#4
#5
#6
#7
FENCE POST
FENCE
FOUNDATION
1 STY. MODULAR
MINE HEADER
MINE OPENING
5' CONCRETE GUTTER
BADLY BROKEN AND SHIFTED
PROP. 8" HDPE PIPE
PROP. 8" HDPE PERF. UNDERDRAIN
10' SHOULDER
38+00
39+00
40+00
41+00
42+00
43+00
44+00
45+00
46+00
MD. RTE. 36
N
SCALE
0 FT
100 FT
LEGEND
PLAN LOCATION OF FOUNDATION BORINGS
PLAN LOCATION OF AUGER BORINGS

The rest of the boring survey was to determine materials types and to locate the mine elevations. These borings showed that some of the rock in this area is covered with a soil mantel that is unusually thick for this part of the state. Water tables as shown by these borings were higher in the slide area than in areas immediately adjacent. The auger boring survey was not successful in determining mine elevations. In some cases the mines were below rock layers that stopped the augers, in other cases the borings were located in front of the coal seam outcrop. The fact that in some of the borings the auger refusal was as high as elevation 1180 indicated that the mine might be present in or under some of the slide area.

Concurrent with the boring investigation, a detailed land survey of the slide area was carried out. This survey located several apparent mine openings within 100' of each other at elevations that ranged from 1062 to 1085. Published mine data does not show this much elevation variance in the coal seams. Also some of the published data shows the mines nearest the Taylor house to be at elevation 1020. The Taylor house is at elevation 1055. It is possible that for operational convenience, the openings may not have been located exactly at the outcrop of the coal seam and are therefore not reliable indicators of the actual mine elevation. It was felt by SHA that the mines should be located by rock core boring. This work was scheduled for the fall of 1991.

The unusually thick soil mantel found in the auger borings of the first phase study are an indication that there may have been past instability in this area. Staff geologists of the Engineering Geology Division theorized that this area might be the site of ancient sliding caused by undercutting of the area by Georges Creek in an old channel that was further to the west than the current location. The absence of rock outcrops in the slide area and the presence of outcrops to the north of the slide area and large boulders seen on the surface to the southwest of the slide area further support this theory. The soil mixed with rock fragments appears to be a colluvium resulting from an ancient slide. Areas behind the house mapped as mine subsidence may in fact be evidence of old slides. Should this be the case, this colluvial material would be sensitive to increases in moisture, or to increased loading due to collapse of the mines behind it.

During the investigation, it was found that a logging operation on the slopes above the slide area had taken place in the summer and fall of 1990 and had left behind a number of haul roads that caused surface water to concentrate and infiltrate in the area.

Phase II Investigation

This part of the investigation consisted of 3 foundation borings (GOW/Core) located along the centerline of the slide as determined by the previous work. All 3 borings were drilled to the lowest elevation where mines are suspected to exist. The first 2 of the 3 borings included slope inclinometers. The first boring was located closely behind the Taylor house. The second boring was located half way up the slide, and the third boring was drilled at the top of the slide.

The borings confirm the presence of the colluvial layer which supports the theory that there was past sliding. Coal mine voids were found between elevations 1067 and 1058 at the top of the slide, and between elevations 1074 and 1067 in the boring made 150' down slope from the top of the slide. The inclinometer data showed that the mine was collapsing in the area of the top of the slide. The area around the house was still moving.

Conclusions

This slide is related to movement and drainage in the mine area. The presence of the roadway cut at the base of the colluvium slope probably has contributed to the severity of the movement, but it is doubtful that all sliding would have been averted had no cut been made.

As the mines collapse, the roof material breaks up and becomes part of the unstable mass. Eventually the mines will stop their current period of collapse and the roadway slope movement should slow, but not stop. Further collapse activity of the mines can be expected, but accurate predictions of when this will happen were not possible. Each time the mines experience collapsing in the future, further major slide activity can be expected.

The analysis of the inclinometer data and the boring logs indicated that this was not a single slide, but a series of small movements that stair stepped up the mountain.

The water draining from the mines and coal seam area will continue to cause deterioration of the stability of the roadway slope, so movement at the base of the slide will continue.

The Taylor property can not be considered safe for any kind of occupation or development for the foreseeable future.

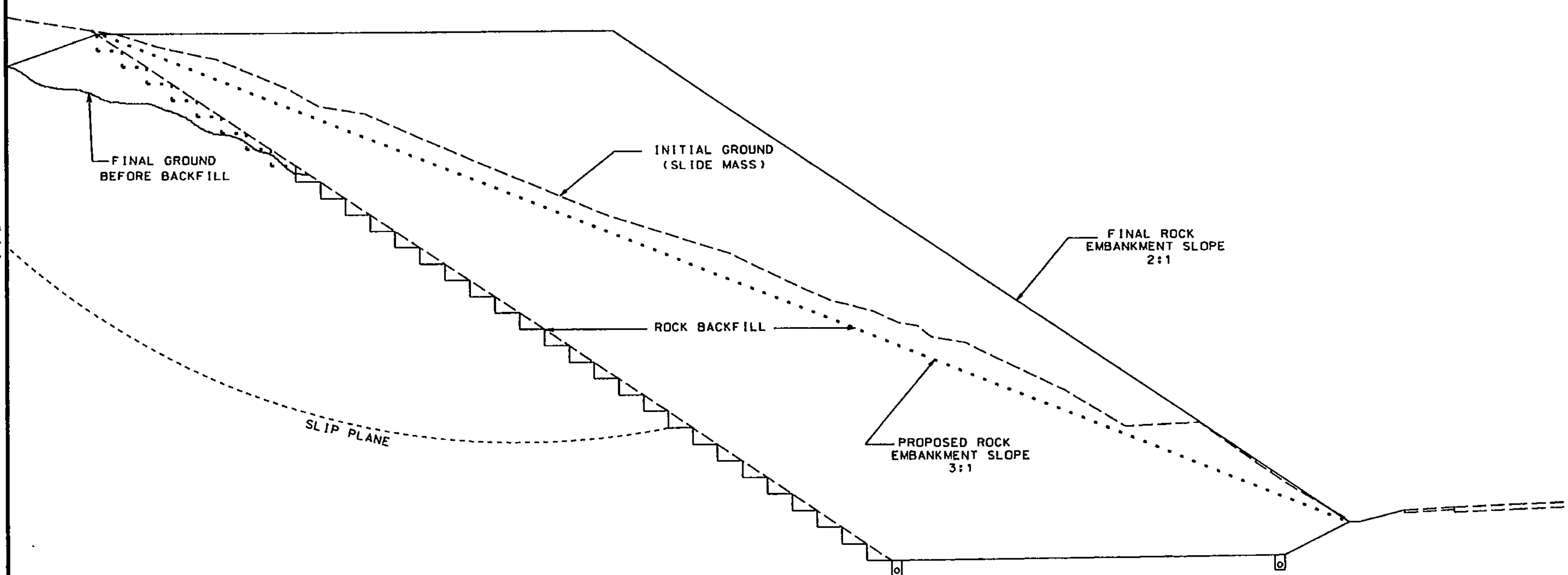
Remedial Actions

The final recommendations for remediation were for a limited excavation and backfill. Since it was believed that this was not a single slide, but a series of small movements that stair stepped up the mountain, theoretically it should be possible to excavate the lowermost slide and place a rock buttress large enough to arrest movement of the upper masses. Figure 2 shows the limits of this plan. Figure 3 shows a typical section. The water flowing from the mine openings and from the outcrop of the seam would be collected and removed from the area so that saturation of the new slopes would not occur.

Conventional retaining structures were not considered practical for this situation. The excavation required to place them on a solid foundation is extensive. Also, since further collapse of the mines is inevitable, the structures must be designed to withstand increased loading of indeterminate amounts.

Although horizontal drains have been used by SHA in the past to temporarily stabilize slide masses, they have never been successfully utilized as a permanent solution. In this instance, they would have to be at least 300' long to be effective. The water in this area has a history of forming precipitates that close drainage.

FIGURE 3



NOT TO SCALE

In order to reduce the risks to the traveling public during the period before the contract could be executed, an attempt was made to drain the water that had accumulated in the mines. Since the axis of the syncline had a more northeasterly strike than the road in this area, it intersected the road a few hundred yards north of the slide area. Old mine drainage placed by the miners and by earlier highway builders was known to exist at this point. These drain pipes had been blocked by precipitates dropped from the acid mine drainage when it came in contact with the air and surface waters. Attempts to open these pipes were only mildly successful, however, a nearby seep turned into a major flow when probed with a shovel. This seep was improved with a corrugated metal pipe and successfully lowered the water level in the mines at the slide so that construction in that area was eased. There was no measurable stabilization of the slide area.

Since the area historically had little or no movement during the July through September dry season, a contract was advertised so as to concentrate the excavation during that period. Unfortunately, 1996 turned out to be the worst year in the last 50 to try to execute this type of plan. Record rain fall insured that continual movement occurred throughout the excavation period and the excavation item grew from an estimated 114,500 cubic yards to 129,042 cubic yards.

The rock backfill item for the buttress increased from 86,470 cubic yards to 93,000 cubic yards due to a change in the finished slope from 3:1 to 2:1. The elevation of the top of the buttress was not changed although there was significant distress in the upper slope.

The completed project was successful in creating a stable slope at the roadway. The upper slopes are still in motion, but they are expected to reach a stable condition without threat to the roadway.

The Bureau of Mines made a further review of the slide and the data developed by SHA. They revised their earlier opinion and determined that drainage from the abandoned mines was at least a contributing factor to the slide. Their revised opinion was based on the fact that the water in the slopes fit the chemical profile of the typical mine drainage of the area, and it occurred at the location of old mine entrances. This change in position resulted in the Bureau of Mines' funding parts of the correction.

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Analysis and Remediation of the Miles Road Landslide Complex, Cuyahoga County, Ohio

by

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Abstract

The Miles Road landslide complex is located just east of the intersection of Miles Road and Bentleyville Road in Cuyahoga County, Ohio. A geotechnical investigation of the complex involving field mapping, drilling of five test borings, installation of four ground water monitoring wells and seven inclinometers, lab testing of selected soil samples, and a series of stability analyses was conducted over a 3-year period (1993-1996). The investigation revealed the presence of five distinct landslides, three on the south side of the road and two on the north side. The majority of the site is comprised of silty clay or clayey silt (CL, ML) with friction angles of 8-30 degrees and cohesion values of 140-898 psf. Stability analyses show that placing the water table at different assumed positions lowers the factor of safety to nearly 1 in all cases, suggesting buildup of pore pressure as the major cause of movement.

Introduction

Northeast Ohio, especially the Cuyahoga County, is a region of high landslide incidence and susceptibility (Radbruch-Hall et al., 1982). Most slope movements in the Cuyahoga County are concentrated along the gorge-like valleys of the Cuyahoga and Chagrin Rivers. The repeated advance and retreat of ice sheets during Pleistocene glaciation led to natural damming of the northward flowing rivers such as the Chagrin River (Ford, 1987). The lake that formed between the ice front and the southern uplands gave rise to deposition of large quantities of silts and clays (Ford, 1987). Following glaciation, the rapid rate of erosion has carved numerous steep-sided valleys in northeast Ohio. It is the presence of steep valleys, fine-grained sediments, and humid climate that makes northeast Ohio so susceptible to landsliding. The human activity, through construction of homes and roads, has further aggravated the situation.

The Miles Road landslide complex is located in the Chagrin River valley (Figure 1) and has been a major hazard to Miles Road since 1984 due to its active nature. The term complex is used here because the site involves a combination of several distinct areas of movement. The flow and slump type movements affecting the slope to the north of the Miles Road deposit wet soil material onto the roadway that blocks traffic, creates hazardous driving conditions, and forces the road to be closed frequently (Hippley, 1993). Miles Road is threatened also by the settlement of the embankment upon which it is built as a result of slope movement to the south. The study presented in this paper was prompted by the hazard the landslide complex poses to the Miles Road and to the adjacent dwellings.

Study Objectives

The main objectives of this study were to:

1. Identify the types and causes of slope movements affecting the Miles Road site.
2. Establish the stratigraphy, hydrogeology, and engineering properties of the soil materials at the site, and relate these to the types of slope movement observed.
3. Perform quantitative stability analyses for selected slide areas within the complex under varying drainage conditions.
4. Monitor the slide areas using inclinometers to identify continued movement and possible failure surfaces.
5. Recommend various cost effective remedial measures to stabilize the failures.

Field and Laboratory Investigations

The landslide complex was topographically mapped, using the transit method, to determine the dimensions of each failure and delineate geomorphic features such as scarp faces, tension cracks, ponded water, and zones of seepage. The mapping phase also lent itself to the establishment of soil stratigraphy, classification of the various slope movements according to the classification scheme developed by Varnes (1978), development of the original slope, identification of the

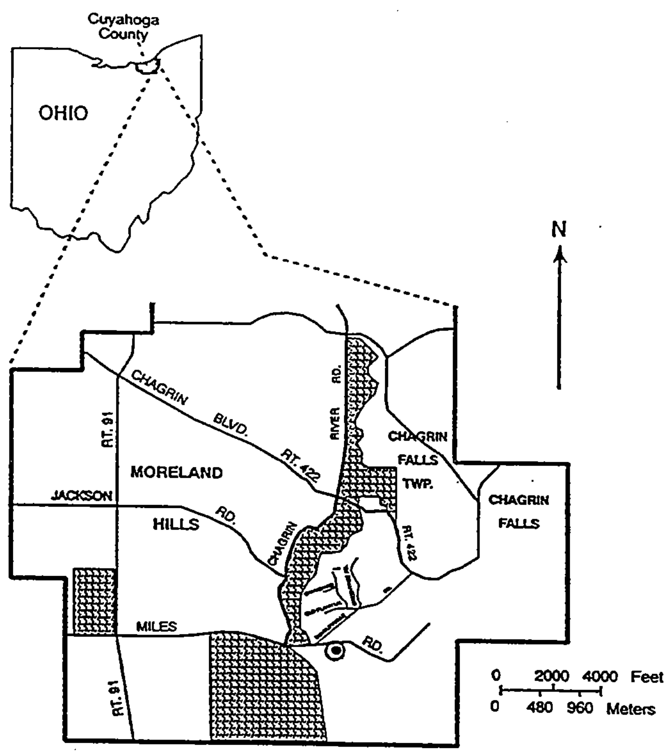

Figure 1: Map showing location of the Miles Road landslide complex.

causes of each failure, and location of the failure plane for those slides where failure occurred along a distinct surface.

During the field mapping stage, soil samples (bulk as well as undisturbed chunk samples) were collected from each layer for laboratory testing. The lab tests performed included determination of natural water content, Atterberg limits (liquid limit, plastic limit, plasticity index), and shear strength parameters by direct shear method. The natural water content and Atterberg limits data were used to compute the liquidity index values for the soils encountered. Atterberg limits were also used to classify the soils according to the Unified Soil Classification System (Holtz and Kovacs, 1981). The shear strength parameters were used for quantitative stability analyses. All lab tests were performed according to the standard procedures as specified by the American Society for Testing and Materials (ASTM) (ASTM, 1993).

Stratigraphy and Engineering Properties

The stratigraphy north of Miles Road consists of 10 feet (3 m) of brownish-gray silty clay (CL), overlying 40 feet (12 m) of brown silt (ML). Table 1 shows the subsurface geology and lists the engineering properties of the soils involved in slope failures to the north of Miles Road. The stratigraphy of the slope to the south, and underlying the Miles Road, consists predominantly of gray silty clay (CL), ranging in consistency from medium to hard and extending to depths of 60 feet (18 m), as indicated by previous drilling reports by R&R International, Inc. of Akron, Ohio (R&R International, Inc., 1991). Table 2 describes the subsurface geology and lists the engineering properties of the soils involved in slope failures to the south of Miles Road.

Table 1: Stratigraphy and engineering properties of the soils involved in failure north of Miles Road.

Depth (ft)	USCS	Soil Description	Engineering Properties						
			w_n	LL	PI	N_{avg}	ϕ	c (psf)	γ_m (pcf)
0-10	CL	Brown-gray silty clay	23	43	15	21	25	165	125
10-50	ML	Brown silt	24	29	8	23	30	140	125

Table 2: Stratigraphy and engineering properties of the soils involved in the failures south of Miles Road.

Depth (ft)	USCS	Soil Description	Engineering Properties						
			w_n	LL	PI	N_{avg}	ϕ	c (psf)	γ_m (pcf)
0-38	CL	Gray silty clay	22	44	18	3	14	898	115
38-53	CL	Gray silty clay	22	26	8	15	8	468	110

Description of Individual Failures

The Miles Road landslide complex consists of five separate slope failures of relatively the same age. Three of the five were chosen for this study due to the threat they pose to Miles Road and the adjacent development. The remaining two were not significant in either size or hazard to warrant detailed investigation. In fact, the 1989 installation of a 420-foot (128m) long sheet pile retaining wall (Figure 2) by North Fork Properties appears to have arrested the slope movement at these two failures, eliminating the need for their investigation.

The two slope failures north of Miles Road, designated as the east and west landslides, are combinations of slump and flow type movements. The east landslide (Figure 3) is more of a classic slump than the west slide. It displays a well developed circular scarp ranging in height from 2 to 5 feet (0.6 to 1.5 m) and extending laterally to 62 feet (18.9 m). The slumped block has undergone distinct backward rotation as evidenced by tilted trees. Minor flows and shallow sloughs originating from this slump block periodically deposit soil onto Miles Road.

The west landslide (Figure 4) is more complex. It exhibits 15 to 20 foot (4.6 to 6.1 m) high scarp faces and has been sculpted by slump and flow movements. The flow movements periodically encroach onto Miles Road and create hazardous driving conditions. The west landslide runs adjacent to Miles Road for a length of 237 feet (72 m) and is less circular in nature than the east landslide.

The slope failure to the south of Miles Road (Figure 5) is not easily categorized using the classification developed by Varnes (1978). However, poorly developed head scarps and the presence of a failure plane at depth, as revealed by inclinometer data, indicate that the failure is a combination of slump and flow movements. The failure stretches for 360 feet (109.7 m) along Miles Road and has a height of 40 feet (12.2 m). There are no well developed scarp faces or slump blocks.

Figure 2. The sheet pile retaining wall installed at the Miles Road site to stabilize south slope.

Figure 3: Overview of the east landslide.

Figure 4: Overview of the west landslide.

Figure 5: Overview of the south slope. Notice the absence of well-developed scarp faces.

Monitoring of the South Slope

The slope movement south of Miles Road was monitored from January, 1993 to June, 1994 through the use of seven inclinometers, designated I-1 through I-7, installed by R&R International, Inc. of Akron, Ohio during the summer of 1992. Five of the seven inclinometers were located above the sheet-pile retaining wall, with two of the five located beneath the stretch of Miles Road which had experienced the most damage due to apparent settlement of the embankment. The remaining two inclinometers were installed below the retaining wall to identify any shear zones which may have developed beyond it. Figure 6 shows the inclinometer locations. The five inclinometers located above the retaining wall were installed to depths of 90 to 95 feet (27.4 to 28.9 m), and the two below the retaining wall to depths of 74 to 78 feet (22.5 to 23.8 m). Baseline measurements were taken after the inclinometers had been allowed to set for one month. New measurements were then taken every month for a period of one and a half years.

Monitoring of the slide has shown that the site is still unstable. In fact, the data indicate the presence of a failure plane at a depth of approximately 30 feet (9.1 m). Plots of the inclinometers I-2, I-5, and I-7, depicting the cumulative displacement in inches versus the depth in feet, are shown in Figure 7. The plots are of two types: the A-axis plot and the B-axis plot. The A-axis plot displays the downslope displacement of the inclinometer whereas the B-axis plot depicts any lateral movement of the inclinometer. There are noticeable differences in the A-axis plots between inclinometers I-2, I-5, and I-7. The differences can all be explained by the locations of the inclinometers. The plot for I-5 reveals a failure plane at a depth of 30 feet (9.1m), which could be due to yielding of the wall in response to the active earth pressure

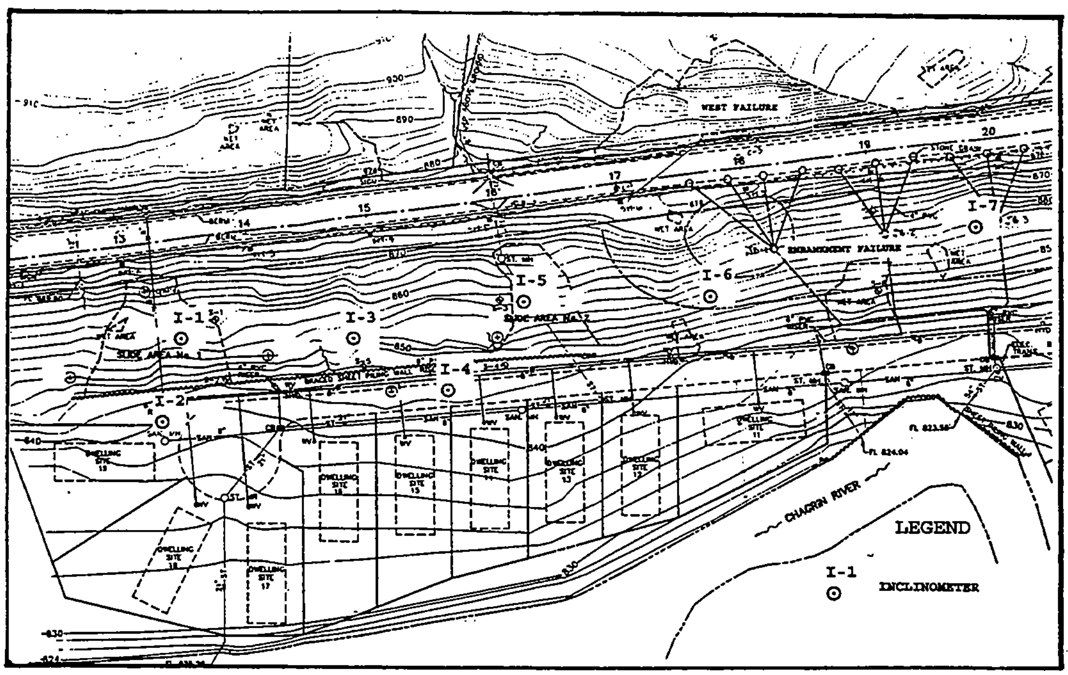

Figure 6: Map showing the locations of seven inclinometers (R&R International, Inc., 1991).

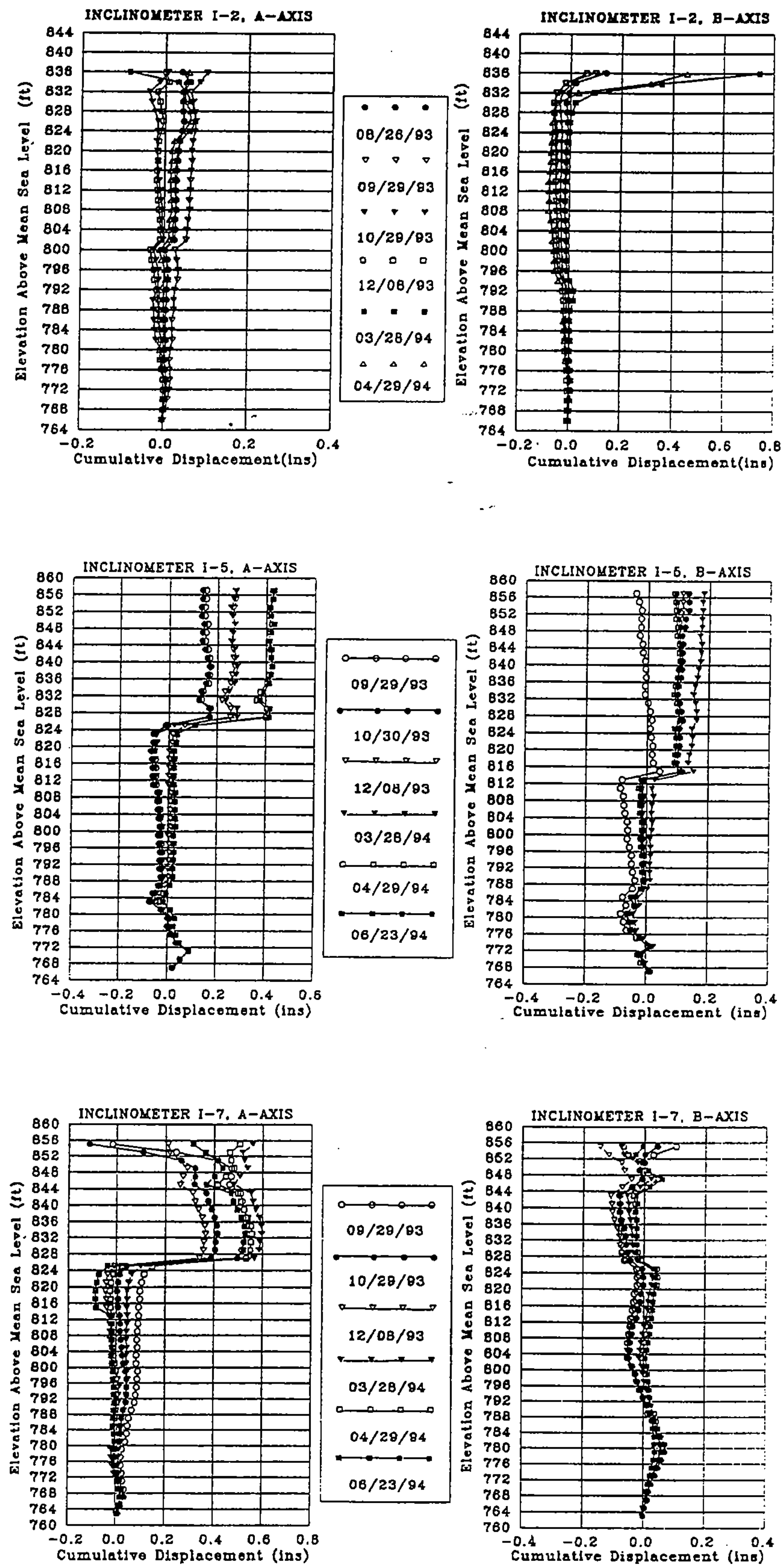

Figure 7: A & B axis plots for inclinometers I-2, I-5, and I-7.

exerted by the embankment slope, since inclinometer five is located above the retaining wall. The I-2 plot displays no significant shear displacement at depth. This is a good indication that the sheet pile retaining wall has been effective at arresting slope movement at the west end of the Miles Road complex. The plot for I-7 again displays shear movement at a depth of 30 feet (9.1 m). The amount of displacement, however, is noticeably larger than the displacement experienced by I-5. This is because I-7 is located outside the arresting influence of the retaining wall and thus has experienced more deformation. It is no coincidence that the stretch of Miles Road which has sustained the greatest amount of damage due to settlement is adjacent to the unretained south slope where inclinometers I-6 and I-7 are located. The plots of inclinometers I-1, I-3, I-4, and I-6 indicated similar results.

Stability Analyses

The slope stability analyses were conducted using the STABL4 program and the results are summarized in Table 3. The analysis for the embankment failure was conducted taking into account the presence of a failure plane at a depth of approximately 30 feet (9.1 m), as indicated by inclinometer six and seven data, the loading caused by the Miles Road surface, including vibratory impacts of heavy truck traffic, and the weak subgrade soils. The original slope for the embankment was not separately analyzed for stability because of lack of information about pre-embankment slope profile. Lastly, the region encompassing the western slide and the embankment slide areas was analyzed for the presence of a large, deep-seated failure. The analysis was conducted considering the influences of Miles Road loading, vibrations of heavy truck traffic, and the failure plane at depth. Figure 8 shows the location of the critical circle for the deep-seated failure beyond the eastern end of the sheet pile retaining wall and in an area where the settlement of Miles Road pavement was most pronounced. The results of stability analyses show the significance of water in initiating and perpetuating the slope failures at this site.

Table 3: Summary of stability analyses for various failures comprising the Miles Road landslide complex.

Failure Location/Slope Cross-section	Factor of Safety	
	Dry Condition	Wet Condition
East failure, original slope	1.08	1.01
East failure, existing slope	1.23	1.03
West failure, original slope	1.12	1.05
West failure, existing slope	1.12	1.01
Embankment failure, existing slope, saturated	-	1.00
Deep-seated, large failure	1.22	1.05

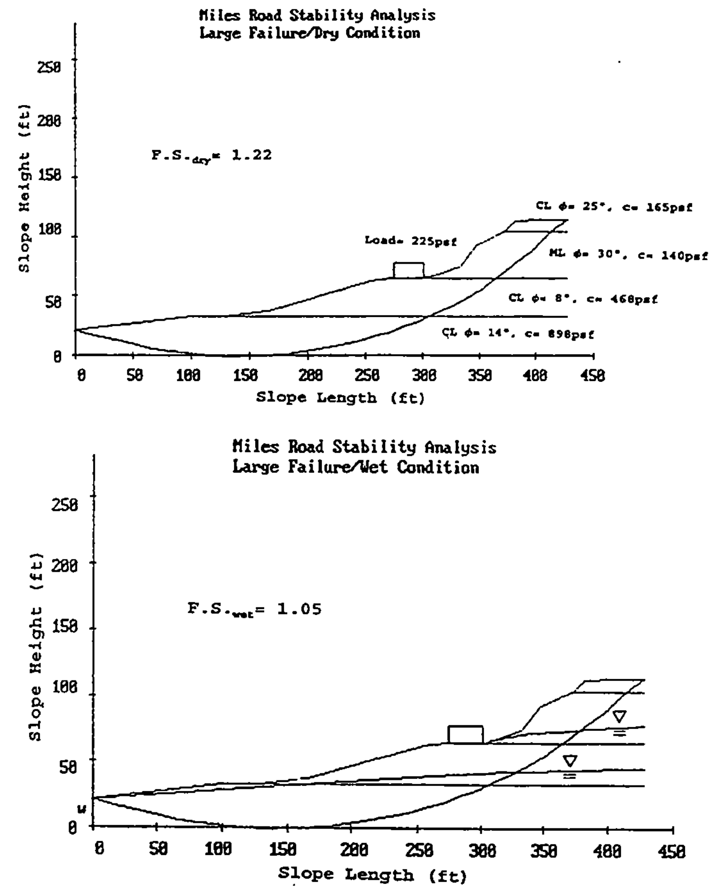

Figure 8. Results of stability analyses for the entire slope, encompassing the west embankment failures.

Probable Causes of Failure

The probable causes of failure at this site are related to both environmental and human elements. Three main factors combine to create the ongoing settlement of the Miles Road surface. The first is the erosion of the toe of the slope by the Chagrin River. The second factor involves placement of the Miles roadbed which affects the slope in two ways: 1) the roadbed enhances the failure through loading of the slope, and 2) cutting of the roadbed oversteepened the slopes to the north, creating the east and west slope failures that exist today. In addition, the movement of traffic, especially heavy trucks, adds powerful vibratory forces to the slope which mimic seismic disturbances. Finally, northeast Ohio's humid climate provides plenty of water through rain and snowmelt, especially during the winter and spring months, to elevate the water table to the critical level at which the safety factor approaches 1.0.

Previous and Suggested Remedial Measures

Previous remedial measures undertaken to stabilize the Miles Road embankment included installation of 13 vertical stone column drains, 30-foot (9.1 m) deep and 30 inches (76 cm) in diameter, along the south edge of the Miles Road surface in the Spring of 1988 (Richland Engineering, 1992). The columns were placed at 30-foot (9m) centers, and were drained by a 4-inch (10cm) PVC pipe. The columns have been largely ineffective as indicated by the continued degradation of the Miles Road surface, the cumulative displacement depicted by the inclinometer data, and the presence of numerous seeps that outcrop on the embankment downgradient of the stone column drains. The toe area of the embankment has been reinforced by the placement of a sheet-pile/gabion wall structure, supported by boulder size rip-rap, to provide erosion protection against the Chagrin River (Figure 9). A geofabric was placed between the gabion baskets and the river bank soils to prevent piping. In addition to these remedial measures, North Fork Properties installed a 420-foot (128 m) long braced sheet pile retaining wall along the western half of the embankment (Figure 2). The sheet pile wall was driven to a depth of 32 feet (9.75 m) in the hopes of arresting slope movement south of Miles Road (Richland Engineering, 1992). The results of the stability analysis of this retaining wall show the factor of safety against sliding for the saturated-undrained condition is equal to 6.2, while the factor of safety against overturning is equal to 5.1. The wall has been quite effective in retaining the slope movement as indicated by the data for inclinometers I-2 and I-4. Although effective on the western half of the Miles Road embankment, the braced sheet pile retaining wall does not support or influence the stability of the eastern half of the embankment, where the most recent, ongoing damage to Miles Road is visible. Also, the east and west failures continue to be active. Therefore, additional remedial measures are needed to further stabilize the site.

The east and west slope failures, north of Miles Road, are suitable for regradation and geofabric reinforcement. The embankment failure south of Miles Road can be remediated by extending the existing sheet pile retaining wall eastward and to a depth of 50 feet (15.2 m). This would ensure the wall is anchored well beneath the depth at which the failure plane is expected to occur.

Conclusions

Based upon the findings of this research, the following conclusions can be drawn:

1. The most common types of slope failure affecting the Miles Road site are slumps and flows.
2. The Chagrin River greatly enhances the active nature of the Miles Road landslide complex by undercutting the toe. The stability analysis reveals that the failure surface for the entire slope daylights in the Chagrin River.
3. Aside from the influence of the Chagrin River, water is a primary catalyst in initiating and perpetuating the slope failures at the Miles Road site as indicated by the stability analyses.
4. Oversteepening and loading of the slopes through the construction of Miles Road, in conjunction with the vibrations caused by heavy truck traffic, has aided the ongoing degradation of the Miles Road surface.
5. The surficial geology of the Chagrin River valley is largely the result of Pleistocene glaciation. The steep valley slopes incised by the Chagrin River are composed primarily of weak glacio-lacustrine silts and silty clays. This combination of weak glacial soils, steep valley slopes, northeast Ohio's humid climate, and human activity has resulted in the formation of numerous landslide complexes in this region.

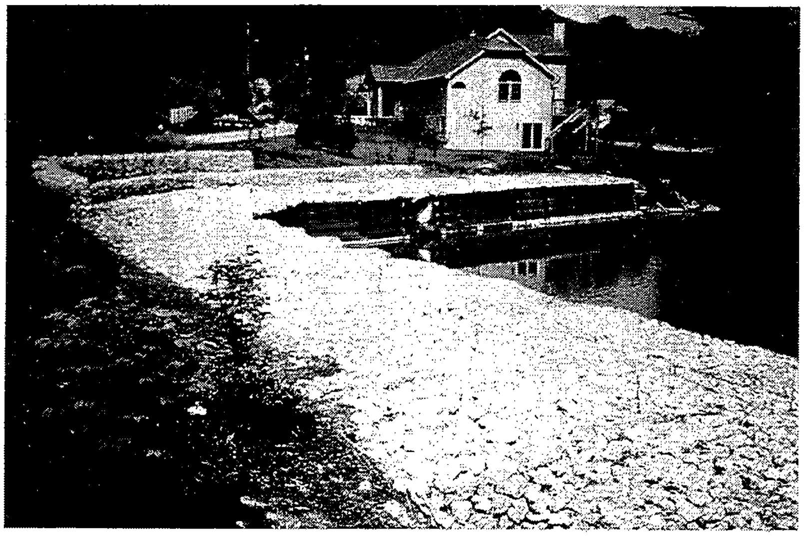

Figure 9. Combination of retaining wall, gabion baskets, and rip-rap to protect the Chagrin River bank from erosion.

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A Case History on the Use of Bioengineering in Reinforced Soil Slope Design

by

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Abstract

The Norton branch tributary, located in Sevierville Tennessee, is a meandering plane-bed type stream carved from alluvial soils. Its proximity to a proposed main entrance of a land development project along Highway 441 required that 180 m (600 feet) of the stream be relocated in this area. Engineers proposed relocation of the stream by damming and redirecting it in three stages. To maximize the amount of land available for the new shopping center, a steepend reinforced soil slope (RSS) was constructed to contain the future flow of the stream. The main challenge for this project was to redirect the stream, construct the steepend RSS, redirect the stream again, and meet the environmental constraints established by the local and state agencies. This paper explores the use of this unique system to meet the challenge associated with this environmentally sensitive stream diversion project.

Introduction

The Norton branch tributary, located in Sevierville Tennessee, is a meandering plane-bed type stream carved from alluvial soils. Its proximity to a proposed main entrance of a land development project along Highway 441 required relocation of 200 m (650 feet) of the Norton Branch Tributary to the Pigeon River. This tributary runs west of Highway 44, and is located along the southern perimeter of the project (Figure 1). To maximize the amount of land available for the new shopping center, a steepend slope would be constructed to contain the future flow of the stream. The main challenge for this project was to redirect the stream, construct the steepend slope, redirect the stream and meet the environmental constraints established by the local and state agencies.

From the project's inception state agencies required that a natural environment be reestablished within the reach of the redirected stream. According to the Tennessee Department of Natural

Resources and environmentalist's stipulations, the proposed steepend RSS could not be designed with a concrete or gabion basket facing system. Timber was an acceptable alternate to satisfy the aesthetic requirements. However, since the chemical treatment required to prevent degradation of the wood was deemed as inappropriate, due to the impact such a treatment would have had on the aquatic and riparian (i.e., frogs, turtles, etc.) life, this facing system was eliminated as a potential design alternate. Instead, reinforced soil slopes in combination with a soil bioengineered facing was selected for this project.

Design of the Reinforced Soil Slopes

This project was designed using the manufacturers' computer program. This slope stability program uses the simplified Bishop method of slices for the RSS analysis. The geometry, soil boundaries and strengths, phreatic surface, seismic coefficients, geogrid layout and geogrid strengths are input into the program. The program then uses a predetermined pattern of failure circle centers as specified by the user to calculate driving and resisting moments for each failure plane. The increased resistance against failure due to inclusion of the geogrids intersected by a circular failure plane is modeled as a resisting moment based on the depth of overburden, geogrid strength and length of the geogrid beyond the potential failure plane. The program determines the critical radius for each circle center analyzed. After all circle centers have been analyzed the program then determines the lowest factor of safety and summarizes the information for this circle center.

The design for the bioengineered reinforced slopes on this project consisted of three analyses for each design section. The first analysis considered a low-water condition, the second a flood condition, and the third a rapid drawdown condition. The rapid drawdown condition governed for most of the design sections. The design high water level was provided to Tensar by the project geotechnical engineer. A 250 psf surcharge was applied to the top of the slope to account for loadings from cars and trucks in the parking area. The design also accounted for a shale formation beneath the bioengineered slope. In the section of the slope where the shale existed, the slope was keyed into the shale. The shale provided a firm foundation for the slope and helped to reduce the amount of geogrid reinforcement required for the stabilization of the slopes. The extent of the shale formation as well as the strength parameters associated with it were provided to Tensar by the project geotechnical engineer.

The slope geometry was extracted from plan views provided in the contract drawings associated with this project. The location of utilities was also provided in the contract drawings. A geogrid layout was developed to accommodate the placement of the utility lines within the reinforced soil matrix. In addition to accommodating utility lines, the slope was designed to accommodate a large set of culverts which connected the channel with a detention pond. Based on the results of the design for this site, Tensar UX1400, UX1500, UX1600, and UX1700 structural geogrids were selected for various elevations within the reinforced soil slopes.

Bioengineering Design of the Reinforced Soil Slopes

The initial bioengineering design required the installation of willow cuttings and dogwood between the five initial lifts of the reinforced soil slope Due to a delay in construction of this system, commercially available bare roots were used instead. As reported by Sotir and Frazier (1997), despite the fact that this stock was significantly smaller than recommended, it was expected to be successful due to good pre-rooting and careful on-site supervision of the installation process. The bareroot willow bundles stored in the stream prior to installation are shown in Figure 2. Completed construction of the reinforced soil slopes with the bioengineered portion is shown in Figure 3. The resulting design cross section, along with the incorporation of the bioengineering design, is depicted in Figure 4.

Evaluation and Monitoring

As reported by Robbin Sotir & Associates (1995), the rooted willow plants installed within the lower five lifts of the reinforced soil slope performed very well, with a plant survival rate of approximately ninety percent. These plants grew two to five feet tall in the first growing season. Despite the fact that two sections experienced a lower survivability rate, the growth of herbaceous pioneer vegetation resulted in a stable slope. The condition of the reinforced soil slope after construction is depicted in Figure 5. Growth after the first growing season is found in Figure 6.

As reported by Robbin Sotir & Associates (1996), there was no evidence of any settling or erosion on the face after two years. They also reported the fact that the structural geogrid reinforced soil slope was effective in stabilization and the willow provided favorable conditions for natural invasion. As shown in Figures 7 and 8, grasses were well established on the upper slopes. As depicted in Figures 9 and 10, the growth of the willows provided cover for shade. This woody vegetative cover provides a natural shelter, and promotes nesting and movement along the stream bank for aquatic and riparian habitat.

Summary & Conclusions

As was reported by Sotier and Frazier (1997), the combination of a reinforced soil slope and bioengineering helped accomplish the following objectives:

1.) Additional Slope Stabilization;
2.) Improved erosion and sediment control for the slope face and stream edge;
3.) Aquatic and Riparian habitat enhancement;
4.) Aesthetic improvements to return the site to a more natural condition, and;
5.) Improvement of water quality

Acknowledgments

The author's would like to thank the work that Robbin Sotier and Lisa Frazier have done to thoroughly document this project from construction through post construction.

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LOCATION MAP

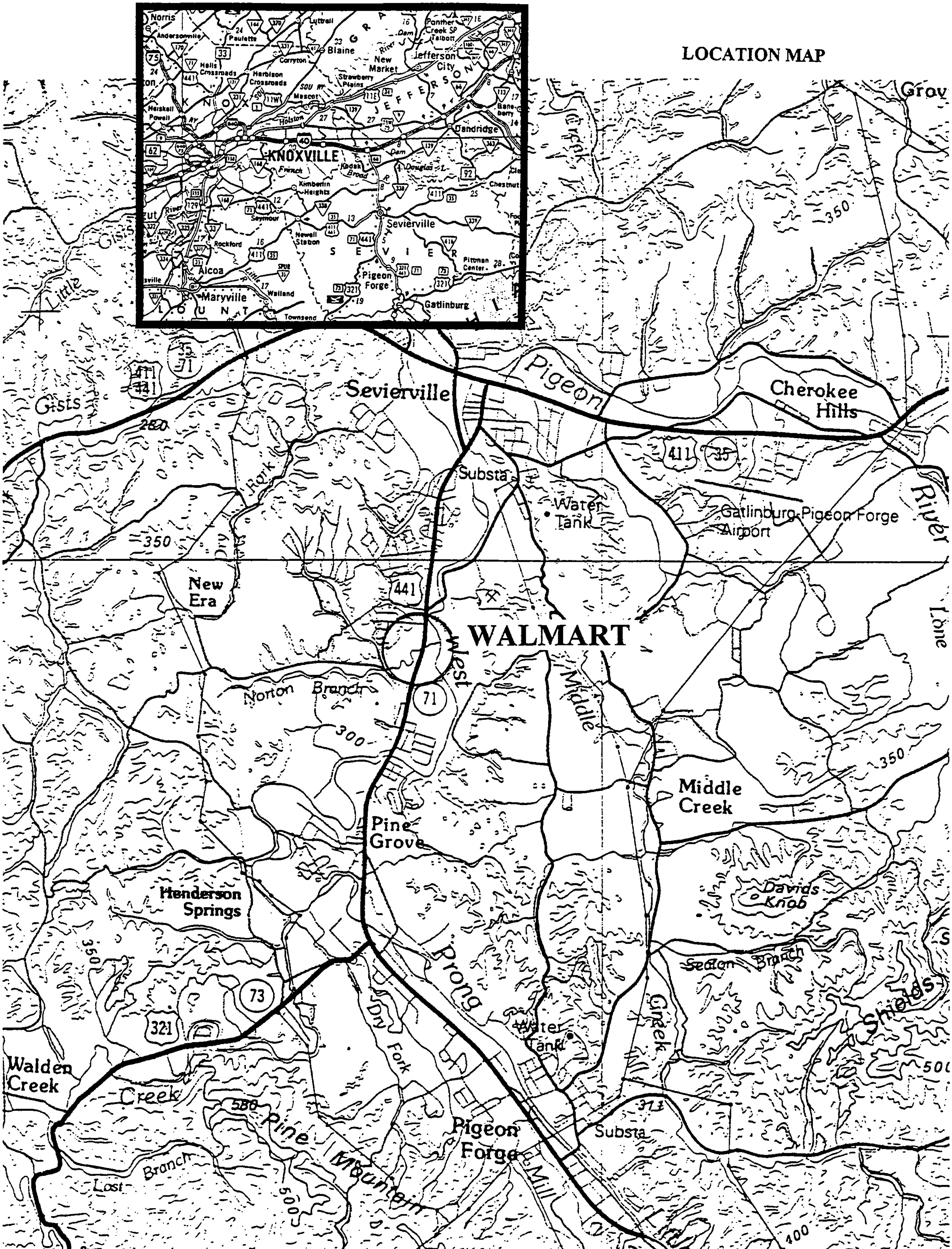

Figure 1 - Site Location

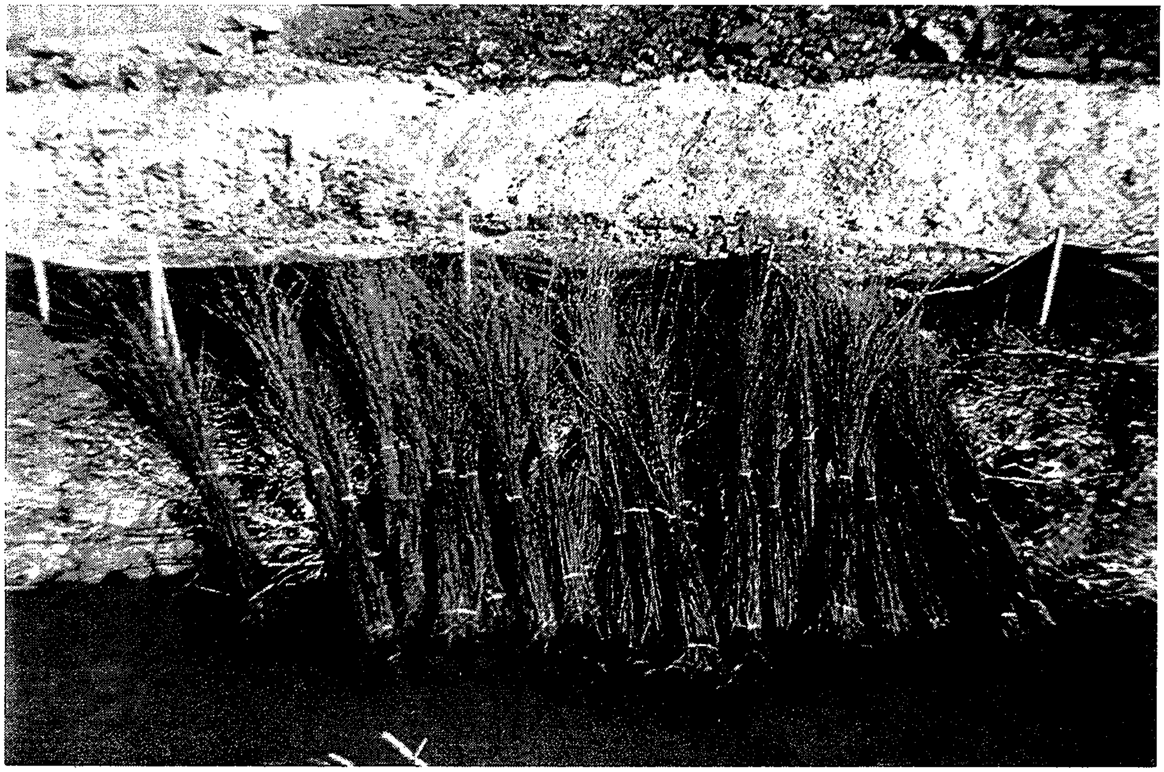

Figure 2 - Bareroot Willow Bundles Prior to Installation

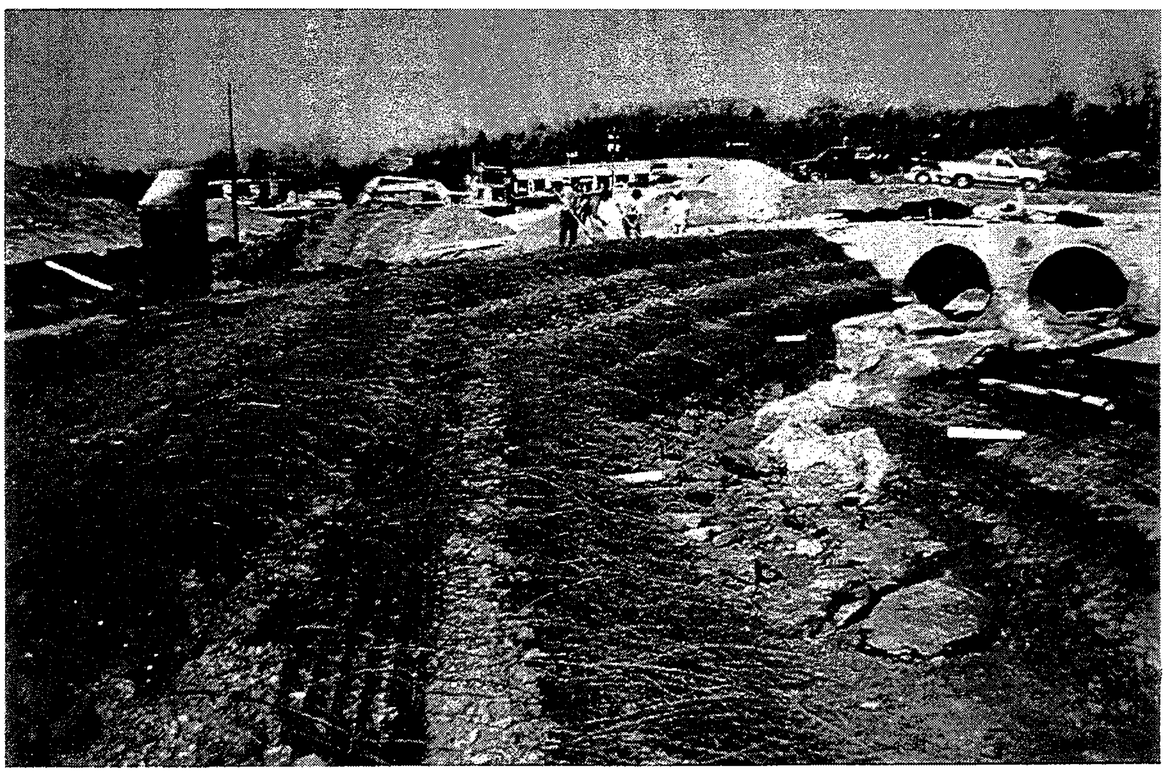

Figure 3 - Completion of the Bioengineered Portion of the Reinforced Soil Slope

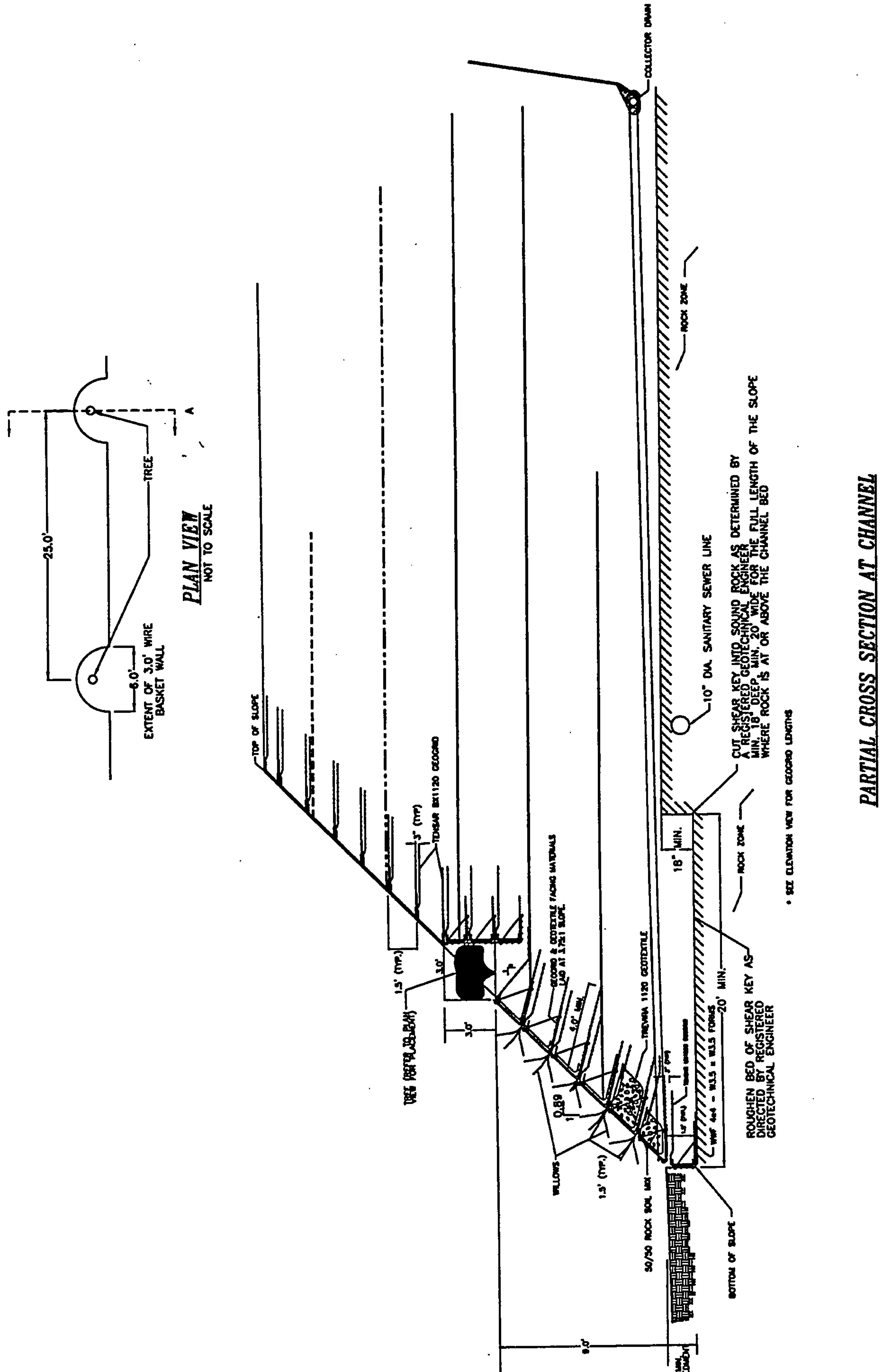

Figure 4 - Design Cross Sections

Figure 5 - Post Construction of the RSS with Lower Bioengineered Section

Figure 6 - Growth After One Year

Figure 7- Growth of the Willows After One Year

Figure 8 - Close-Up of the Stream Bed

Figure 9- Growth of the Lower Slope After Two Years

Figure 10 - Growth of the Upper Reinforced Soil Slopes After Two Years

State of the Art for Rock Cut Slope Design in Eastern Kentucky

by

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Abstract

The Kentucky Department of Highways utilizes benches in most of their cut slope configurations. Special shaped ditch recommendations employing modified "Richie Ditch" criteria are considered for certain situations with differential weathering problems or right of way restrictions.

Coal mining operations in Eastern Kentucky have presented challenging problems and rock cuts are designed so that they may be modified if unforeseen conditions are encountered during construction.

Introduction

The geological formations in Kentucky consist of sedimentary rocks such as sandstone, limestone, shale, siltstone, and coal. With the exception of the Pine Mountain Thrust fault area (Eastern Kentucky) the majority of the formations are essentially flat-lying with local variations of dip.

Guidelines are designed to provide sufficient flexibility to allow the design of slopes in a wide variety of rock formations. Each cut is independently designed by using all subsurface information or field mapping available. Open face logs, rock cores, soil borings, rock soundings, undisturbed samples, etc., are utilized to determine cut slope angles, lift heights, and bench widths. Soil overburden thickness and rock disintegration depths determine the overburden bench requirements.

Rock cut slope configurations are influenced by lithology, but are primarily based on joint inclination and continuity. Benches, where possible, are located at the top of the least resistant lithologic unit in a given rock cut section. Formations that have vertical and/or horizontal changes in lithology are usually designed to accommodate the worst conditions.

State of the Art for Rock Cut Slope Design in Eastern Kentucky

Cut slopes in shale are designed according to weathering characteristics indicated by the Kentucky Method 64-513-78 for determination of Slake Durability Index and the Jar Slake Test (Kentucky method 64-514-77).

Nondurable Shales, Class III, with or without laminations, will usually be in category 1 or 2 of the Jar Slake Test and have a Slake Durability Index range of 0 to 49 percent. Typical cut slope recommendations will vary from 1 vertical to 2 horizontal (1:2) or flatter slope from ground line to ditch line. Normally these slopes are designed without a roadside ditch bench, intermediate benches or overburden benches. Refer to Figure 1.

Nondurable Shales, Class II, will have a Slake Durability Index range of 50 to 79 percent and may be in category 3 or 4 of the Jar Slake Test. Typical cut slope recommendations vary from 1 vertical to 1 horizontal to 4V:3H with roadside ditch benches, intermediate benches typically 5.5m (18') wide, and approximate lift heights of 9m (30') depending on rock competency. Refer to Figure 2.

Nondurable Shales, Class I, will have a Slake Durability Index range from 80 to 94 percent and a category 4 or 5 of the Jar Slake Test. Typical cut slope recommendations vary rom 2V:1H to 4V:1H with approximate 9m (30') lift heights, intermediate benches typically 5.5m (18') wide, and with a roadside ditch bench. Refer to Figure 3.

Durable Shales will have a Slake Durability Index range from 95 to 100 percent and the Jar Slake Test results will usually be in category 6. Typical cut slope recommendations vary from 2V:1H to 4V:1H (depending on fractures) with roadside ditch benches, typical intermediate benches 5.5m to 6m (18' - 20') wide and approximate lift heights of 9 to 14m (30' - 45'). Refer to Figure 4.

Massive Limestone or Sandstone Typical cut slope recommendations vary from 2V:1H to 20V:1H. This material is usually stable; however, presence of joints, fractures, cross bedding etc., will have as much influence on slope design as lithology. Materials placed on 20V:1H slopes may have lift heights of up to 18m (60'), with intermediate benches 5.5m to 6m (18 - 20') wide. It is desirable to design the first lift above grade on a slope flatter than 20V:1H. Refer to Figure 5.

Shaley Limestone and Sandstones Typical cut slope recommendations vary from 1V to 1H to 2V:1H with lift heights from 9m to 14m (30' - 45') and intermediate benches 5.5 or 6m (18' - 20') wide. Flatter slopes may be required depending upon the percent and type of shale present. Refer to Figure 6.

Intermediate and Overburden Bench Widths The elevation of most intermediate benches is determined by changes in lithology, with the bench being of top of the best resistant material, where possible. The width of intermediate benches may vary from 4 to 8m (15' - 25'). Typical bench widths are 5.5m (18'). Intermediate bench widths may be 6 - 8m (20' - 25') when lifts exceed 9m (30') in height or in situations where shale is expected to weather rapidly and undercut a massive bedded material. Unstable slopes with an anticipated heavy rock fallout may also require wider intermediate benches.

Intermediate benches which intercept ditch grade should be transitioned out within a distance of 4.5 - 6m (150' - 200') to avoid leaving a transverse rock wall in the cut slope.

Overburden benches are placed on top of rock cuts at the base of the weathered rock. Typical overburden benches are 5m (15') wide and may be wider in areas where instability is anticipated. The depth to the base of RDZ is measured vertically from ground line and may be highly variable. Overburden benches are drawn horizontally on cross sections and will have some grade through the cut depending on variations in depth of material. These benches are sometimes omitted in mountainous terrain or in cuts where overburden is less than 3m (10') deep.

Roadside Ditch Bench When cut slopes are steeper than 2V:3H and the 9m (30') safety clear zone from edge of pavement to the cut slope is not required, a roadside ditch bench is recommended. Typically, the width of the roadside ditch bench from outside edge of shoulder to the cut slope will be 3.5m (12') for cuts less than 9m (30') in height, and 4.3m (14') for cuts over 9m' (30') in height.

Serrated Slopes Serrated slopes are utilized as a means of controlling erosion and establishing vegetation on soft rock formations, shale or other material that can be excavated by bulldozing or ripping. Serrations may be recommended for 1V:1H or flatter cut slopes that are 6.1m (20') or more in height. Typical step risers will vary from 0.6 to 1.2m (2' to 4'). Refer to Figure 7

Special Ditch - Catchment Areas When the cut slope design does not contain intermediate benches, a specified ditch-catchment area, modified after Richie's design criteria, may be required. Wire mesh on the slope face, Jersey barrier walls or high impact retaining systems can be utilized to provide added safety. Refer to Figure 8.

Continuous Cut Slope Design Continuous cut slopes, without intermediate benches, may be recommended when the following existing conditions are present:

- Rock in the cut slope is homogenous
- Joints are discontinuous and massive failures are unlikely
- Intermediate benches will accumulate debris and become ineffective
- Rock consists of limestone or sandstone with small RQD numbers (less than 25%) that are interbedded with shales of low slake durability (SDI) values (less than 50%).

Typical continuous cut slopes vary from 4V:1H to 2V:1H but may be flatter, and are generally less than 30.5m (100') in height.

Cut Slopes in Highly Tilted Strata Lithologic variations will be more complicated in highly tilted strata depending upon the apparent dip. Complete field reconnaissance of each cut section is required prior to slope design. Recommendations are influenced by apparent dip of strata along centerline and cross sections as well as lithology and character of the strata.

Normal design criteria for slopes may be utilized when the apparent dip along centerline is less than two (2) degrees and apparent dip on the cross section is away from the roadway. Intermediate bench elevations should follow apparent dip and will have a slight grade. These benches are drawn horizontal on cross sections and will cross cut strata in one direction.

Intermediate benches with widths from 5.5 to 8m (18 - 25') may be utilized and should be designed as horizontal in cuts where the apparent dip along centerline is more than two (2) degrees and the apparent dip on the cross section is away from the roadway. Then benches will cross cut strata in two directions. Cut slopes with a maximum vertical lift of 18m (60') are recommended according to the strata encountered in that particular lift.

Intermediate benches are to be omitted and one slope should be recommended from the top of rock to grade in cuts where the apparent dip on the cross sections is toward the roadway. This slope is determined by lithology and character of the strata. In some areas, where a large mass of material could create a major landslide, the design slope should follow the dip of the strata.

<u>Cut Slopes in Coal Mine Areas</u> Design considerations relating to mines include a determination of whether the mine is below, at, or above grade. Mines below grade that do not show signs of subsidence are generally left undisturbed. Mines at or near grade may be excavated and replaced with suitable backfill. Cut slope design for mines above grade utilize wider benches, shorter lifts, and conservative cut slope angles of 2V:1H or less. By utilizing conservative recommendations, the slopes can be revised during construction without a complete redesign of deeper cuts. Mine openings are backfilled a minimum distance of 6.1m (20') from the face of the cut. Pneumatic backstowing using No57's gradation material is preferred and is usually included as a cost item in the bid prices. The last 1.5m (5') of back stowed material contains five (5) percent cement and positive drainage is provided.

ACKNOWLEDGMENTS

The writer would like to express appreciation to Henry Mathis and the Geotechnical Branch personnel for their assistance in preparing this paper, Jackie Overbey for her secretarial work, and Terry Gash for his graphics.

Typical Slope Configuration
Class I Nondurable Shale

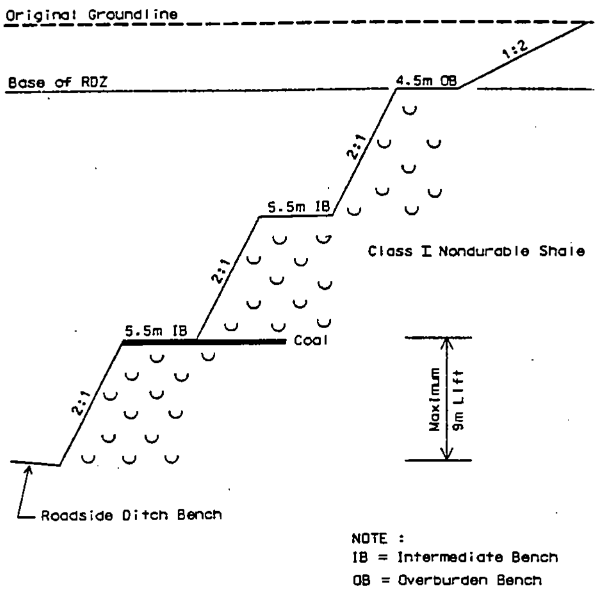


Figure 3

Typical Slope Configuration
Durable Shale

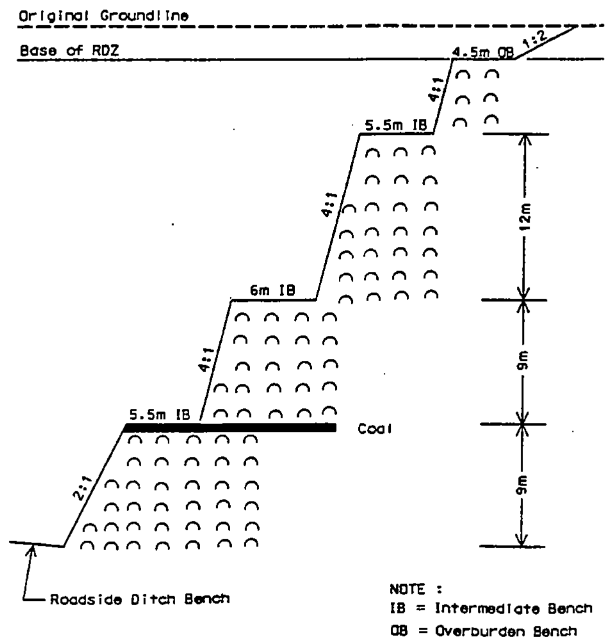


Figure 4

Typical Slope Configuration
Class III Nondurable Shale

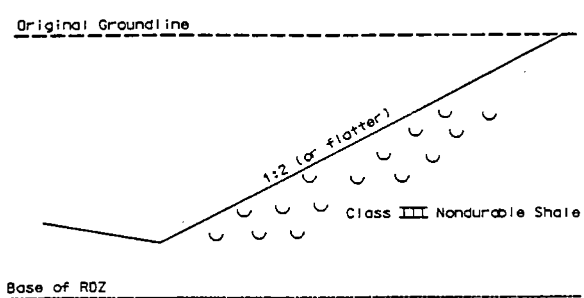


Figure 1

Typical Slope Configuration
Class II Nondurable Shale

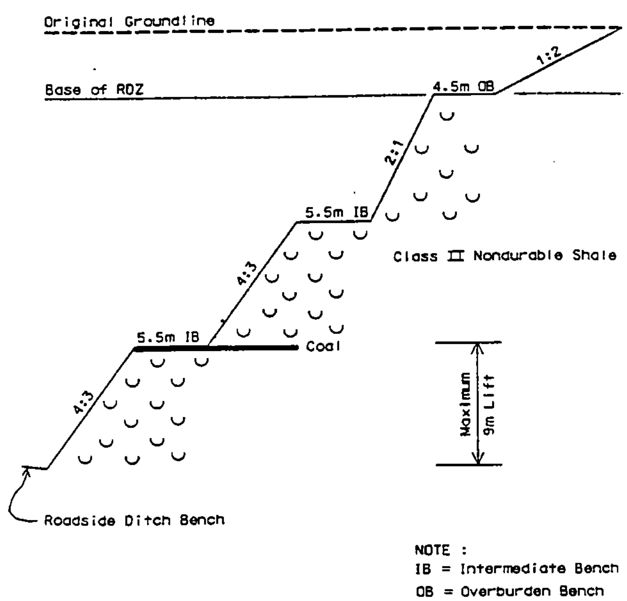
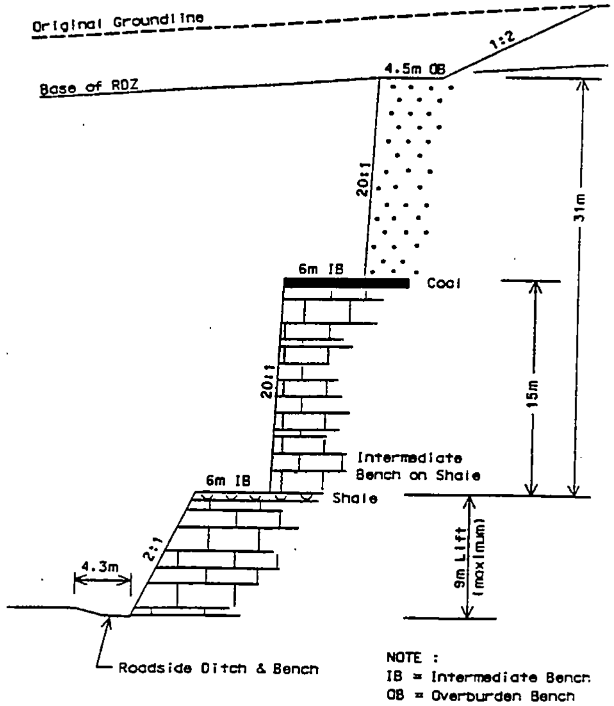


Figure 2

Typical Slope Configuration
Massive Limestone or Sandstone

Figure 5



Typical Slope Configuration
Shaley Limestone or Sandstone

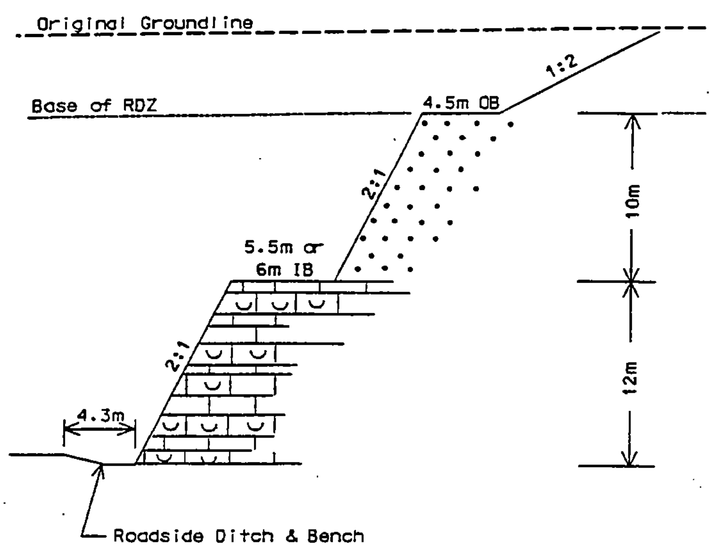


Figure 6

NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Rock Slope	H (m)	W (m)	D* (m)
Near Vertical 20:1	5-10	3.5	1
	10-20	4.5	1.25
	over 20	6	1.25
4:1 or 3:1	5-10	3.5	1
	10-20	4.5	1.25
	20-30	6	2
	over 30	7.5	2
2:1	5-10	3.5	1.25
	10-20	4.5	2
	20-30	6	2
	over 30	7.5	2.5
4:3	0-10	3.5	1
	10-20	4.5	1.25
	Over 20	4.5	2
1:1	0-10	3.5	1
	10-20	3.5	1.5
	Over 20	4.5	2
(*) If catch fence is used this dimension may be 1.25			

Design Criteria for
Roadside Ditch Catchment Area

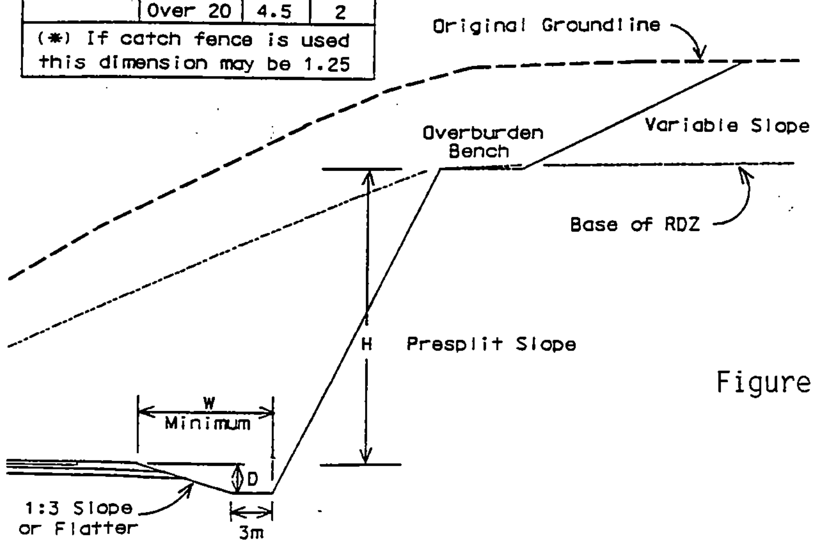


Figure 8

Typical Slope Configuration
1:1 Serrated Slopes

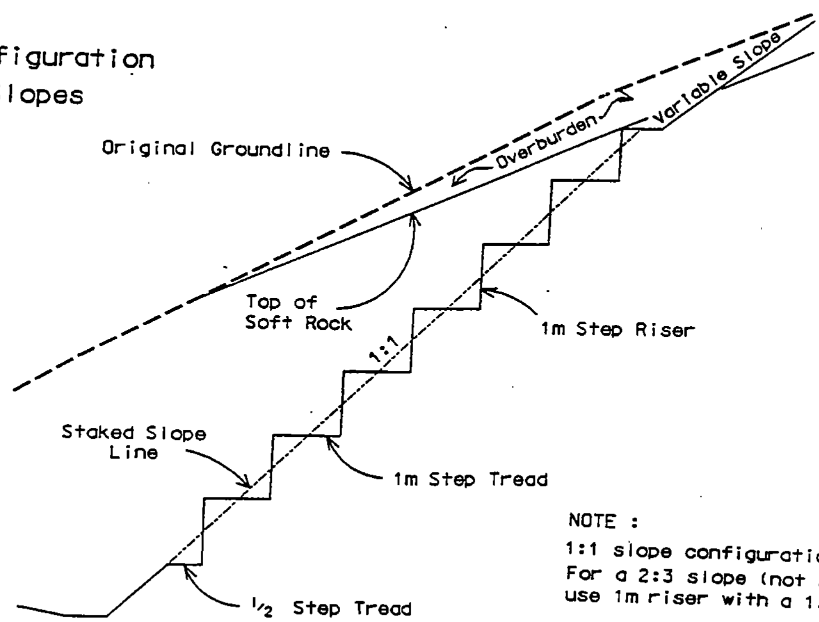


Figure 7

NOTE :
1:1 slope configuration shown.
For a 2:3 slope (not shown)
use 1m riser with a 1.5m treads.

Rock Stability Analysis Based on Potential Energy

by

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Abstract

Analyzing the stability of rock slopes involves predicting the sliding stability of blocks formed by planes of weakness within the rock mass. Plane and wedge sliding analyses, for which there are one and two sliding surfaces, respectively, have been widely used for the last thirty years. For these slides, given certain assumptions, the distribution of normal forces on the contact planes is statically determinate. The resisting and driving forces may then be calculated by limiting equilibrium methods and the factor of safety determined directly. Certain geologic environments, however, especially those of folded rocks, produce potential failures which cannot be modeled as either plane or wedge slides. In these slopes the sliding of blocks may occur along non-planar surfaces. Standard rock stability analysis methods presently available are not adequate to treat these folded rock slopes. For this reason, a new energy model has been developed that can analyze the stability of a rock block with any number of contact planes or a curved contact surface. This paper demonstrates the model on a slope in Carter County, Tennessee, an area of a past failure in cylindrically folded shale. Results indicate that this failure could have been predicted with the model and computer program. The results also show that treating this failure as a wedge with two sliding surfaces greatly overestimates the factor of safety, possibly leading to an unsafe design.

Introduction

Determination of the stability of a rock slope involves predicting the behavior of a discontinuous rock mass where failure surfaces generally occur along preexisting discontinuities. These discontinuities may be bedding layers, which provide a plane of weakness at the interlayer contacts, or later occurring fractures. The classic methods of rock slope stability analysis - wedge analysis and plane slide analysis - are based on limiting equilibrium and are usually evaluated with graphical methods, such as stereographic projection, or with computer programs.

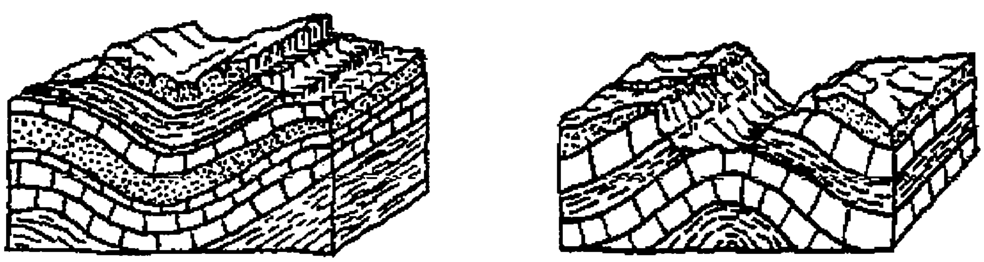
These methods assume that all shear stresses in the contact plane(s) directly oppose motion. This makes the distribution of normal forces on the contact planes statically determinate, given that there is one or two contact planes. The resisting and driving forces may therefore be calculated directly and the factor of safety determined directly as the ratio of the resisting force over the disturbing force.

Certain geologic environments, however, produce blocks which cannot be modeled accurately as wedge or plane slides and cannot be evaluated with the methods discussed above. An example is a block formed by cylindrically folded sedimentary rocks. As shown in Figure 1, folds form synclines and anticlines, with synclines being concave upward and the anticlines concave downward. When these rocks are folded the interlayer contacts are weakened, analogous to bending a deck of cards, and interlayer slip may occur. When slopes are cut through these folded rocks, failures may occur along the contacts of these bedding layers. The sliding surface in such cases cannot be described as single plane or an intersection of two planes, but is a curved surface. Studies have shown that if these blocks are idealized as a two plane wedge, as is commonly done in practice, the factor of safety may be overestimated by the use of the wedge model (Mauldon and Ureta, 1995, 1996; Ureta, 1994).

For this reason, a model based on potential energy has been developed to analyze the stability of this prismatic or curved blocks. This model was developed at the University of Tennessee through a research program funded by the Tennessee Department of Transportation (Mauldon, 1995). Further details and the mathematical development of the model may be found in Mauldon, Arwood, and Pionke (1996), Arwood (1996), Mauldon and Ureta (1995, 1996), and Ureta (1994). A computer program, *ROCKSLIP*, was also developed to implement the model. A study site in East Tennessee was selected to demonstrate and test the model and computer program. The site is a series of cut slopes along US Highway 19E near Biltmore in Carter County, upper East Tennessee (see Figure 2). The cuts were made through thinly bedded Sevier Shale approximately ten years ago.

Site Geology

The southern segment of the Appalachian Mountains can be divided into four belts or provinces (Rodgers, 1970). These are the Appalachian Plateau, Valley and Ridge, Blue Ridge, and the Piedmont Plateau. The Appalachian Plateau province is characterized by flat-lying or nearly flat-



(a) (b)

Figure 1 Two fold structures (a) a syncline, and (b) an anticline (Wilson, 1981).

lying sedimentary rocks, mainly Carboniferous. Most of the plateau is dissected by deep narrow valleys. The Blue Ridge province stretches from northern Georgia to southern Pennsylvania and forms the main divide of the Appalachians. This is the most mountainous area of the region, containing the Unaka and the Smoky Mountains, among others. The Piedmont Plateau is a nearly featureless upland plain underlain by crystalline schist, gneiss, and gneissic granite (Rodgers, 1970).

The Biltmore study site lies within the Valley and Ridge geologic province. This province is typified by narrow ridges, a kilometer or less across, persisting for tens or hundreds of

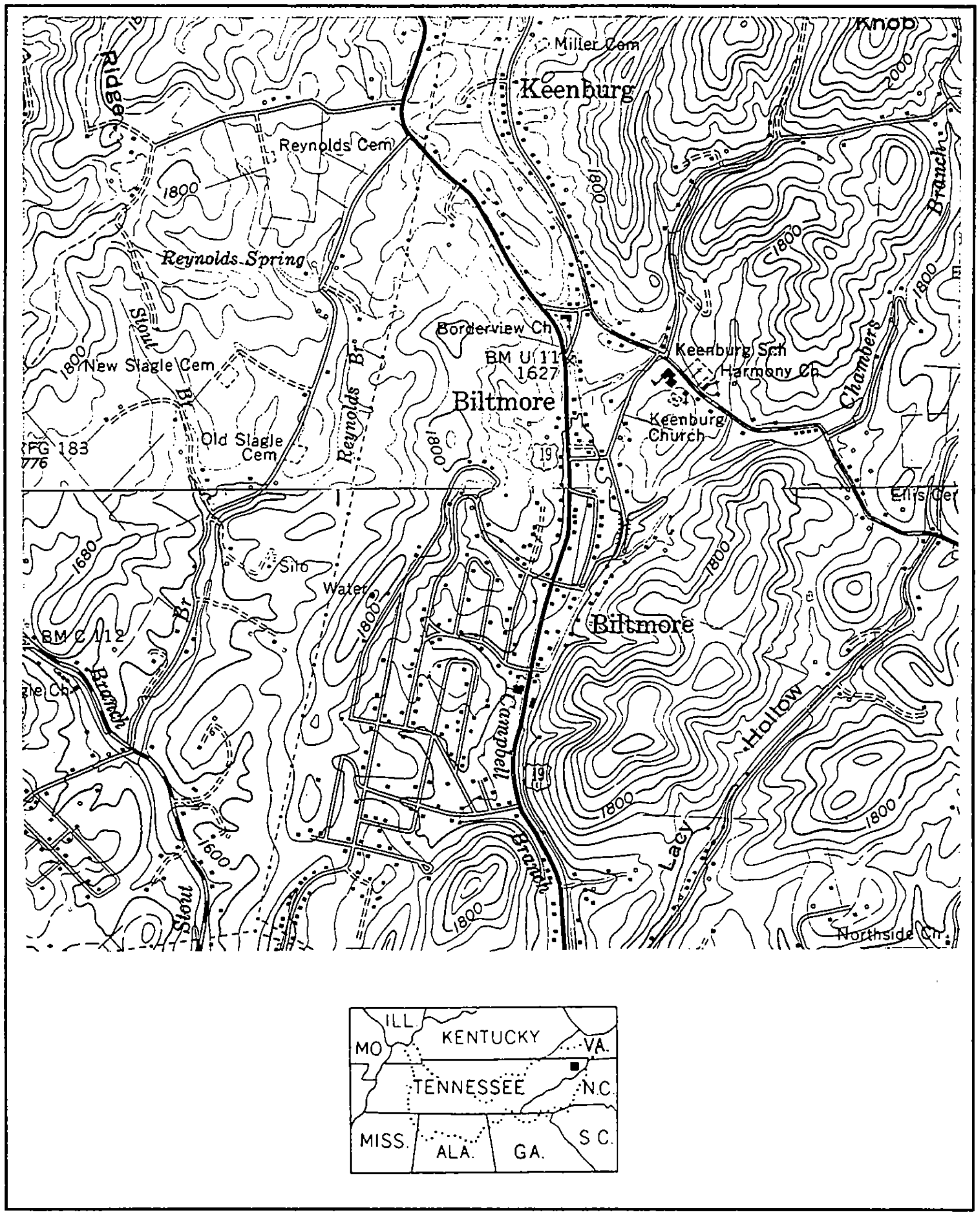


Figure 2 Study site location in Carter County, Tennessee (Elizabethton and Keenburg Quadrangles, 1969.)

kilometers. They are strictly parallel to each other and to the trend of the chain. Between the ridges are extensive parallel valleys, some narrow but others very broad. The province is underlain by a diversified sequence of Paleozoic sedimentary rocks (Cambrian to Carboniferous) that have been strongly folded and faulted with little metamorphism (Rodgers, 1970). These include limestone, sandstone, siltstone, shale, and dolomite. Nearly all rocks are faulted and tilted to the southeast, the direction of the massive tectonic forces that formed the structures.

Several of these thrust faults are in Carter County, where the southeasterly dipping thrusts have produced folds at the study site with southwesterly trending fold axes. The Sevier Shale which is exposed at the site is of Ordovician age (400 to 500 million years) and is of the Chickamauga Group. Published geologic information indicates that the Sevier Shale has a thickness of 2000 to 7000 feet (Thornbury, 1965). This is a bluish-gray shale with thin gray limestone layers that can have a yellowish weathered appearance. The rock at the site is thinly bedded, with layers no more than a few centimeters in thickness. Structurally developed cleavage which is perpendicular to bedding is readily evident on the slopes. This additional plane of weakness results in small rock blocks spalling off the slopes, most of which are well less than a half meter across, but a few larger. Rock fences have been constructed along the bases of some of the slopes to stop these blocks before they reach the roadway.

The particular slope chosen for analysis is an area of a past slope failure near the community of Biltmore, Tennessee. This large syncline failed shortly after construction of a new section of State Route 37. The bedding is readily visible on the slope and it appears that the failure occurred generally along the curved bedding planes (see Figure 3). The model and *ROCKSLIP* were applied here as "post-mortem" analysis, to determine if the failure could have been predicted and to determine the stability of the block that was there.

Orientation Data Analysis

The orientations of seventy bedding planes and bedding fractures were measured on the analyzed slope. A lower hemisphere stereographic projection of the poles to these planes is shown in Figure 4. These poles lie approximately along a great circle, indicating that this may be a cylindrical fold. The orientation of the cut slope face was also measured for input into *ROCKSLIP*. The dip direction was found to be 225° azimuth and the dip to be 75° from horizontal.

An input file was created in order to calculate the orientation of the fold axis using *ROCKSLIP*. The program calculated a fold axis with a trend of 228° azimuth and a plunge of 11° from horizontal. These values are very near the field estimated measurements of 225° azimuth and 15°. The program also determined that this can be classified as a cylindrical fold based upon criteria proposed by Ramsay and Huber (1987, pp. 333-336).

Figure 3 Field sketch from the Biltmore site showing measurement locations, bedding, and cleavage.

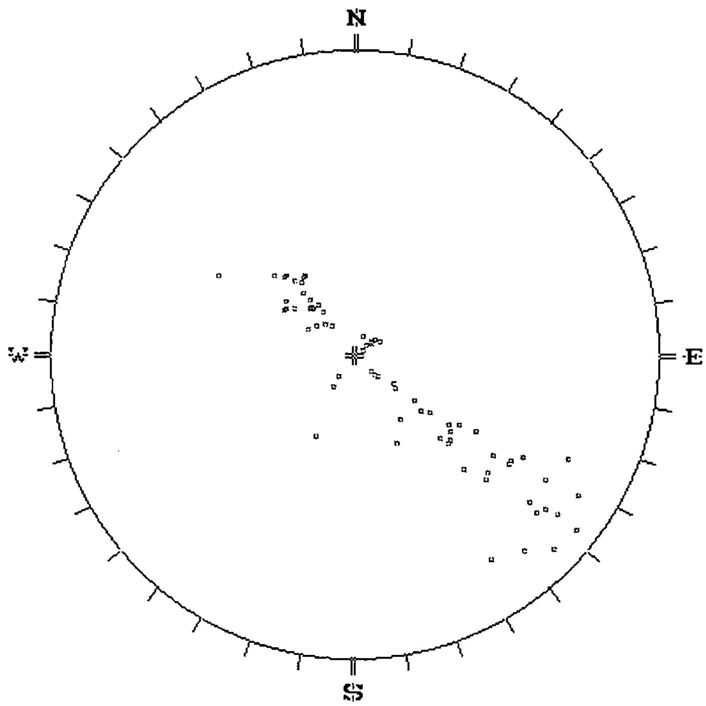


Figure 4 Lower hemisphere equal angle projection of the poles to the bedding.

Analysis

An image of the Biltmore slope was used in the *ROCKSLIP* analysis of the past failure surface of the slope. The curved surface of the failure block was modeled in the program as a prismatic block with a finite number of planes. The block was idealized as several different shapes for the sake of comparison; these were a two sided wedge, a three sided block, and a five sided block. Variations of each case were also used in the analysis. Figure 5 shows the three wedges, three 3-sided blocks, and three 5-sided blocks for which factors of safety were calculated.

ROCKSLIP was run to analyze each of the nine failure blocks shown. For each block the calculated fold axis and the measured slope face orientation were used as input. The traces of the blocks were corrected to a true section using this information. The exact friction angle on the contact surfaces is unknown. Therefore, a range of friction angles was used, from 15° to 30°. Published angles for shale are in the range of 20° to 25°(Goodman, 1989; Hoek and Bray, 1981). Results of the analysis are shown graphically in Figure 6.

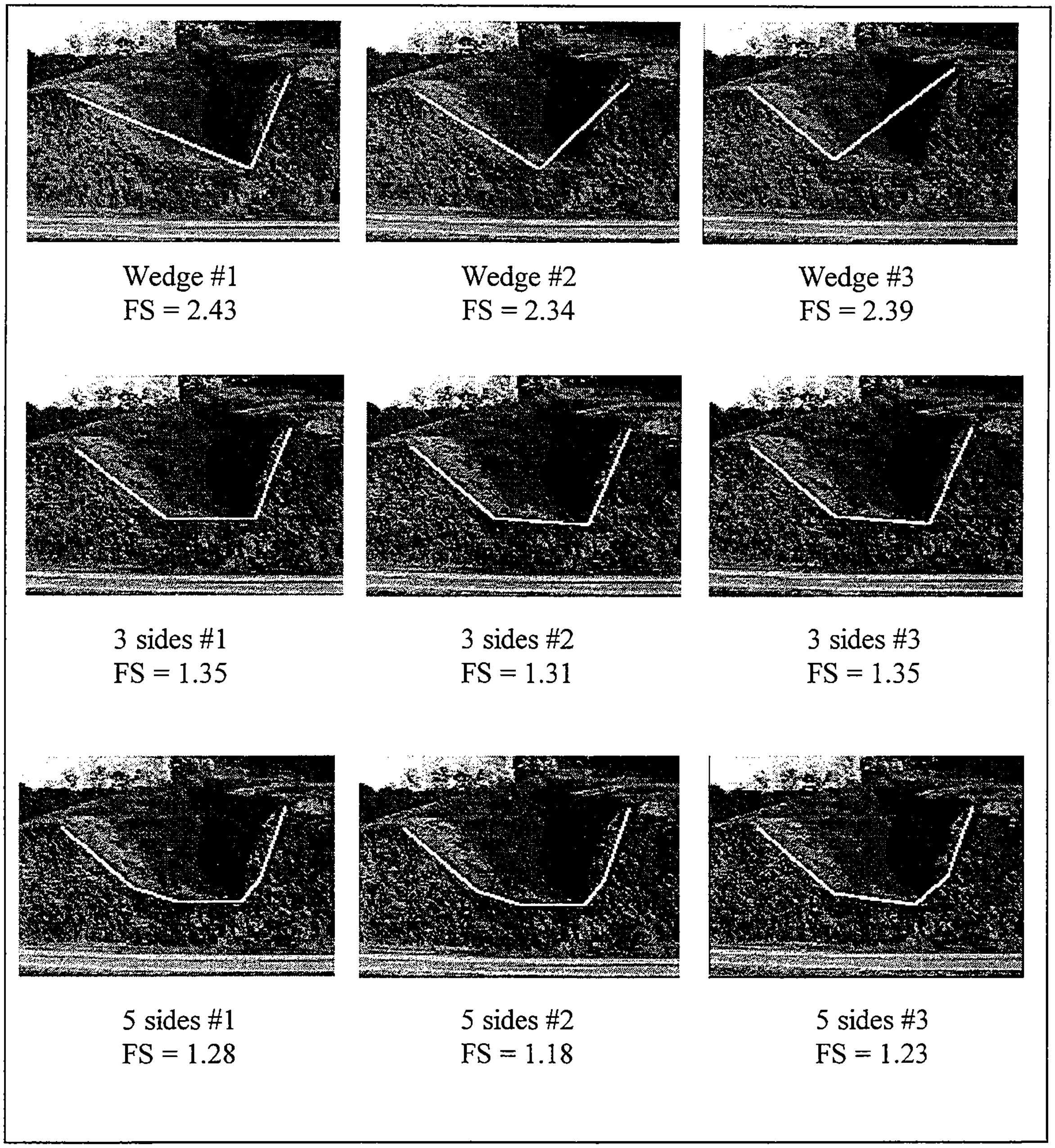


Figure 5 The failure surfaces, all with a fold axis plunge of 11° and a friction angle of 20°, used in the analysis of the Biltmore site with their factors of safety.

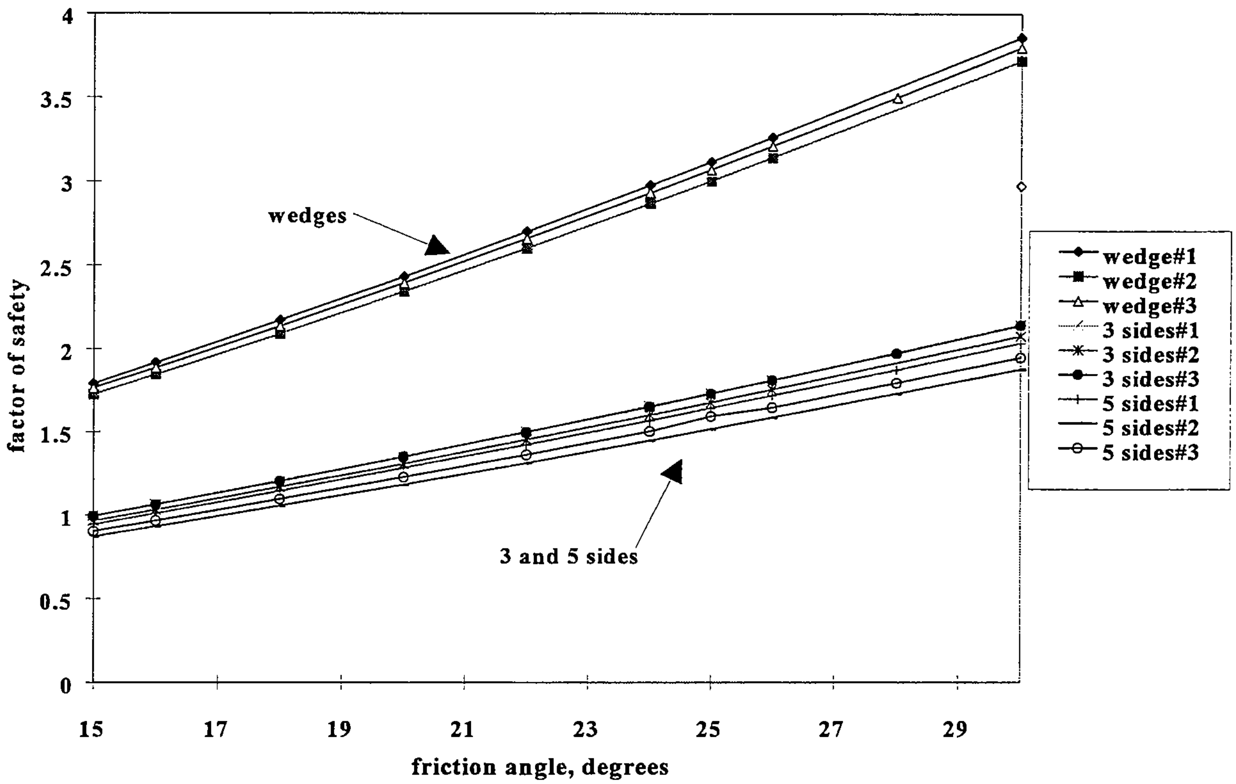


Figure 6 Factors of safety for each analysis of the Biltmore site for a range of friction angles.

Discussion

From Figure 6 it can be seen that the factors of safety for the three wedge analyses are nearly the same, ranging from a low value between 1.5 and 2 to a high value over 3.5. The plane sliding analysis gave factors of safety less than the wedges, but above the three and five sided blocks. The wedge values are well above those of the three sided and five sided blocks, most in the range of 75 to 100 % higher. The factors of safety for these six blocks range from less than one at a friction angle of 15°, to a little less than two at a 30° friction angle. This demonstrates clearly that the wedge model overestimates the factor of safety and could lead to an unsafe design. The factors of safety for the five sided blocks are slightly lower than the three sided blocks.

The factors of safety for the three and five surfaced prismatic blocks are less than the desirable 1.5 up to a friction angle of about 25°. This indicates that this slope, and in particular this block, is potentially unstable. Therefore, it can be concluded that this past failure could have been predicted using this model. However, in actuality, the syncline feature was not detected in the field investigation prior to the road construction. It should also be noted that this analysis did not include a consideration of water forces, which would only reduce the factor of safety further.

CONCLUSIONS

In summary, a new model for slope stability in folded rocks has been presented. This model can calculate the factor of safety against sliding for a prismatic or cylindrical rock block, commonly produced in folded rocks. The analysis of slope stability in folded sedimentary rocks presents unique problems and situations which to date have not been adequately dealt with in the geotechnical literature. The folding of the bedding planes produces potential failure blocks with curved sliding surfaces. Traditional limiting equilibrium stability techniques are capable of the analysis of plane and wedge failures only. These one and two sided blocks are not adequate to model a slope failure in folded rocks. The main purpose of this research program is to develop a model for the sliding stability of folded rock blocks and to develop a computer program to implement this model. The computer program *ROCKSLIP* uses an image of the slope as its major source of input along with field data from the rock mass.

A case study from East Tennessee was presented in which the model was tested and demonstrated with favorable results. A slope with a cylindrical fold in shale was analyzed. The slope was an area of a previous failure, with the failure occurring along bedding planes in a syncline. Results presented here indicate that this failure could have possibly been predicted.

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The Role of a Contractor's Blasting Consultant - A Case Study

By

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Abstract

This Project is in the environmentally sensitive Sonoran Desert northeast of Phoenix, Arizona. Coordination with Tonto National Forest staff was critical. Design plans and specifications reflect the reduction of impacts to natural resources in a corridor where the existing roadway paralleled this project.

The roadway rock cuts are in the following rock types: quartz monzonite, basalt, andesite and sandstone. Dikes and sills intersected the primary rock type reducing the bulk rock strength.

Tombstone Hill, a visible geomorphic feature with a cut slope of 130' ± on the east side and the shoulder of the existing roadway on the west side. Approximately one million cubic yards (cy) of excavation was the primary consideration of requiring a contractor's consultant.

The use of explosives involved the reduction of 20+cy boulders contiguous to the operational roadway.

Additionally, wet holes, hole diameter, use of detonating cord in pre-split and cushion holes, buffer hole charges, and slopehole angles that could change every ¼ station made developing stable rock slopes a challenge.

The project specifications required the contractor employ a blasting consultant. A limited number of consultants are available with blasting expertise and knowledgeable in availability of various types of explosives, drilling and excavation equipment, rock slope stability, safety, highway planning principles, surveying, environmental considerations, contract scheduling and management and blasting and drilling personnel. The consultant must be part of the contractor's group preparing the bid.

A major problem was the state's approval of blasting at midnight. The contractor could obtain a bonus of $500,000+ based on the cost differential in the time of day lane rental. Blasting at night is not consistent with OSHA regulations.

In spite of the difficulties encountered, the project was made safe by the blasting sub-contractor's careful on-site management.

Introduction

The State of Arizona has embarked on a significant statewide highway improvement program. This highway, State Route 87, is part of the program and will provide four lane service between Mesa and Payson. A portion of the traffic is to recreation areas northeast of Phoenix.

This eleven mile section is in the Tonto National Forest Sonoran Desert environment. Preservation, environmental concerns and visual compatibility were coordinated with National Forest Staff from planning to construction. Excavation and fill specifications were normal but the drilling and blasting were detailed and lengthy. The contractor was required to retain a consultant after the preconstruction meeting and the state had a blasting consultant for the design phase through construction. The design addressed visual impacts with rugged rock slopes, staining of rock slopes, shaped fill slopes and relocated boulders and cactus.

It should be noted the location of major highways in National Forests require coordination at the federal level government involving US Departments of Transportation and Agriculture.

Engineering Geology

Field modifications required slope angle, buffer holes and amount of explosives to vary for each lift which made blast plans difficult to prepare. The toe of rock slopes were fixed while the top varied. Excessive ripping by the excavation contractor created short and sharp elevation differences and loose rock that impacted the drilling. It is difficult to collar a borehole in loose rock. Accurate drilling especially for slope boreholes was difficult. The upper surface of the rock was not uniformly weathered and a level bench was not developed by the excavation contractor. Pioneer access to rock cuts was rough and this was critical in two cuts that required significant boulder reduction. The strong weathering along joints reduced drilling accuracy with increased bit and steel wear. These joints produced angular slope breaks that were not consistent with the natural landscape. Some of the measures for visual mitigation utilized in the design were:

- Controlled blasting including pre-splitting.
- Rounding of the top slope.
- Use of cushion blasting to provide rock slopes that were generally free from borehole traces.
- Create ledges and pockets for vegetation (Figure 1).

In all cases any slope elements that could deflect rockfall towards the roadway were to be removed.

Figure 1 - Finished portion of a slope consistent with the Natural landscape. Light color in left corner will be stained.

Rock cut slopes in highly weathered bedrock were 45° and 81° in competent rock. Field observations of rock defects in the previous lift were of some value relative to the dominant joint pattern. Even if the clay filling could not be measured, it was assumed to be present. Drilling was not an indicator of clay filled joints. Joints in the quartz monzonite and granite developed a rugged slope that when stained, had the appearance of a weathered slope. The basalts were not uniform and contained polished joint surfaces that produced a less angular slope. Contact zones produced by dikes and sills varied from near glass to high mineralized or fractured (Figure 2).

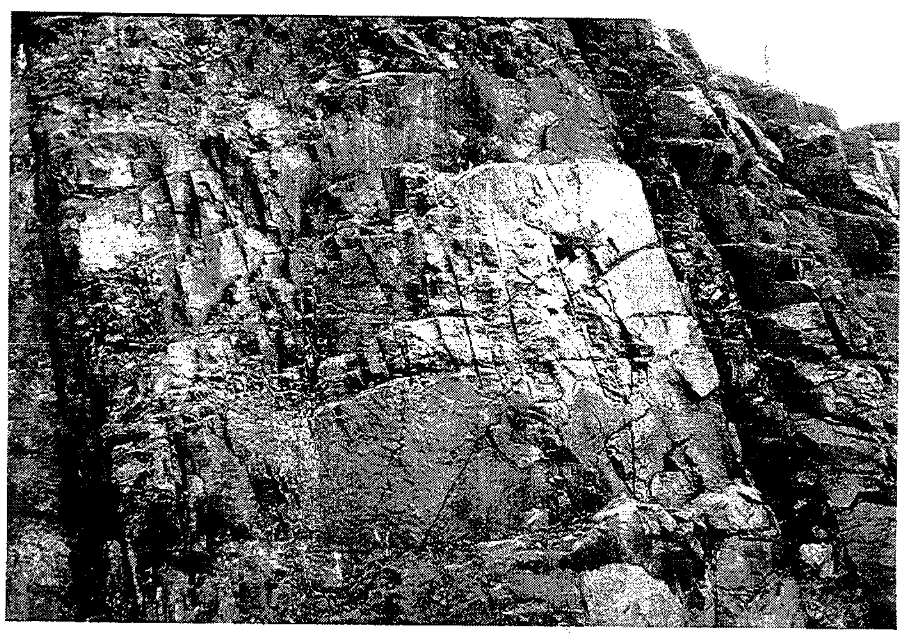

Figure 2 - Dark grey basalt dikes vary in thickness from 0 - 2-1/2 feet. Parallel fracturing has reduced rock strength.

The following are rock strengths obtained by testing rock cores:

fresh to slightly weathered	13,000 psi
moderately weathered	7,400 psi
strongly weathered	1,800 psi

(certain fresh quartz monzonite samples tested to greater than 20,000 psi)

The rock testing relative to drilling and blasting was useful but did not reflect the joint set frequency fracturing. A rock cut of approximately one million cubic yards required more than three core borings. The subsurface exploration program should have been supplemented with geophysics, as cross-hole tomography. It is possible to utilize every piece of design data to estimate optimum borehole diameter and explosive type to determine what impact rock fragmentation will have on the slopes during construction. The main body of holes would utilize a 4.5" diameter borehole based on the data available. A top hammer hydraulic drifter drill was selected that would provide reasonable bit, steel and coupling life. The selection of the drill type was necessary to proved a cost estimate for the drilling. The core boring reports and field work provided an indication of the abrasive quality of the rock. It was assumed basalt would be more abrasive. The actual vertical borehole drilling was near the projections, while the wear in drilling slope holes was greater. Drilling slope holes at angles less than 60° increases the consumption of bits, steel and couplings. Technical data relative to the drilling process is very important and should be obtained by the state as a contractor would need a long lead time to obtain access to test drill in a national forest. The exception was the drilling of the large boulders utilizing a track backhoe excavator with a hydraulic drill mounted in place of the bucket. Pioneer drilling and boulder reduction required drilling holes at all angles. The equipment had to be able to drill on difficult set-ups. The use of this equipment was also a strong safety consideration as drilling from ropes exposes worker to falls. Some of the boulders were nearly 30 feet in diameter.

My goal was to ensure safe stable rock slopes would be constructed consistent with the visual mitigation requirements of the plans and specifications (Figure 3). The demonstration test blast and ripping made in the summer of 1995 was of some value in assessing controlled blasting, fragmentation, hole trace acceptability, rock sculpting and boulder reduction. The information on drill rates was useful. The project specification did not limit borehole size and that one element, determines the success of fragmenting rock with explosives. Based on the test blast, a 4" maximum diameter borehole would have been compatible with the visually compatible slope requirements.

The 30' lift thickness was restrictive since it did not consider the dominant joint sets and had the potential of dislodging rock between joints. The specifications did not specify the amount of stemming necessary to minimize flyrock with an operational two lane highway within the project limits.

An unusual note was found on the plans, obliterate existing pavement and the top of the slope was located in this area. Concrete barriers were moved to accommodate drills. Drill masts extended into plane of the existing roadway. Traffic control was not provided since it was a period of low traffic volume. An unsafe practice (Figure 4).

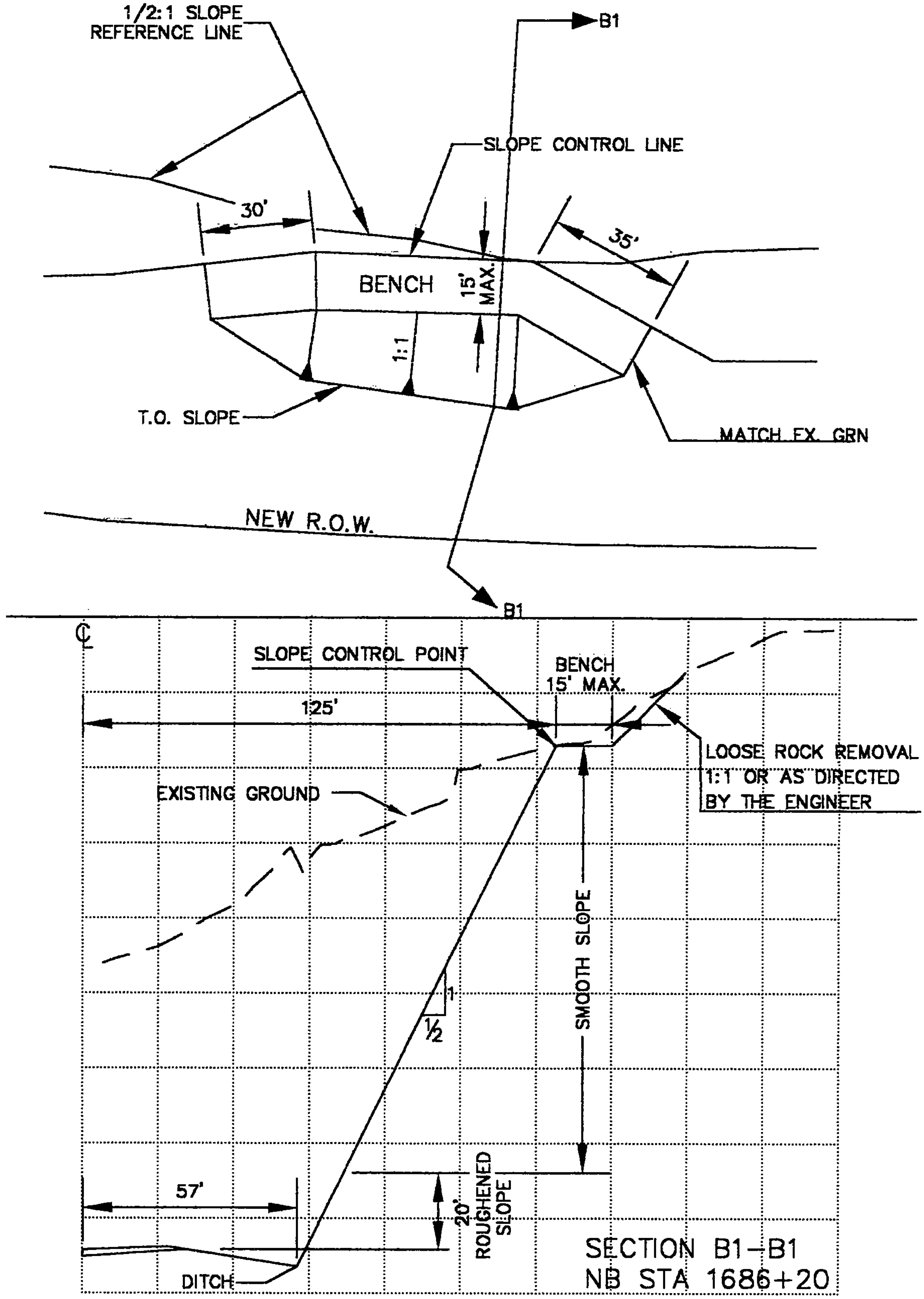

Figure 3 Typical cross section, modified, upper slope benches can be used for establishing vegetation. Note the slope control point.

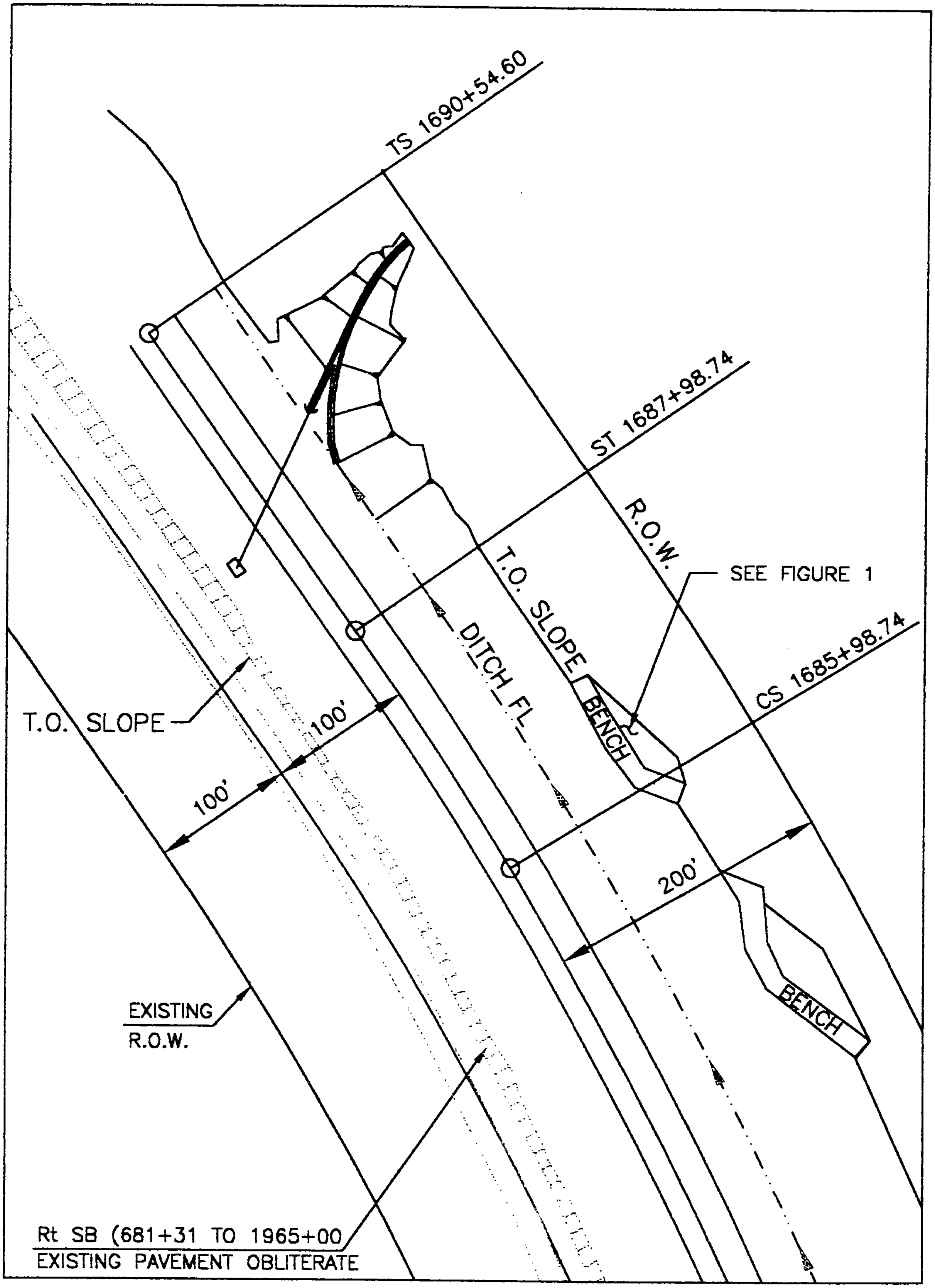

Figure 4 Section of the plans, modified. Slope control point is within cross hatched area. Staking of the toe of the slope is critical as the slope ratio varies.

Slopes were developed with pre-splitting or cushion holes. In some cases it was necessary to use two rows of buffer holes and breaker holes to minimize the amount of energy transmitted along rock defects (joints, fractures, weathered areas and contact zones) that could cause slope failures. The use of detonating cord in controlled slope holes was preferred because of the low amount of explosive per foot and very little gas product. Some concern was expressed because of staining. Organic compounds in the detonating cord can leave a carbonaceous residue. The was observed in 1963 when pre-splitting highway slopes was in its infancy. Removing the residue was not practical and the staining would moderate the problem.

The spacial relationship of a loaded blasthole to a controlled slope hole is important to produce stable slopes. Presplit slope holes must be detonated before the main body of production holes. Cushion holes must detonate after the main body of holes. The rock quality will cause the distance to vary, but two four-foot distance is adequate where the slope hole is loaded with less than 0.2 lbs./ft. The thermal energy must be accounted for in developing slopes with controlled slope holes. This combined with the brisance of detonating cord can enhance fracture development with a preferred direction between boreholes in the same plane.

Project

This project utilized blasting at approximately midnight which is not consistent with OSHA or OSM that requires blasting to take place between sunup and sundown. In the Rock Excavation Demonstration Report, "Because of the critical need to maintain traffic flow on SR 87 during construction, it was decided to provide an incentive for the Contractor to minimize the closure of the road.", Engineers International, 1995. The amount was approximately $550,000. There was no mention of limiting blasting to the daylight hours in the report. The state's blasting consultant approved all night blasting plans. The Arizona Division of Occupational Safety and Health received an inquiry from US Department of Transportation and agreed that this was inconsistent with current OSHA, but the blasting work was completed. Any person preparing specifications must be familiar with safety regulations to avoid problems. Blasting at midnight is not safe and site security difficult, especially with an operational roadway adjacent to the work area.

The project did not utilize a grading plan that was consistent with a project schedule. Construction activities were not compared to computer routine. Drilling and blasting appeared to move from one cut to another to provide fragmented rock for a particular activity. With lack of pioneering it was not possible to overhaul the necessary material increasing drilling and blasting costs. The lack of prepared fill areas caused oversize to be reduced when it could have been incorporated into the fill side slopes. The majority of the oversize material occurred due to the contractor ripping to the top of rock producing numerous high areas. The lack of a level bench impacted the accuracy of the drilling. In spite of the difficulties the drilling and blasting contractor performed admirably. The last lift of the Tombstone cut was competed by the excavation contractor using a closely spaced drill pattern that fractured rock beyond the slope lines, created highs in the subgrade, damaged existing rock and used electrical blasting caps.

Future Considerations

The acquisition of subsurface data must be commensurate with the degree of difficulty of the project. A rock cut with limited subsurface data was excavated successfully as the blasting subcontractor was willing to use extreme care to develop rock slopes consistent with the plans and specifications.

Continuous blasting at midnight is not safe. Project specifications must clearly reflect OSHA requirements. The blast times should be scheduled during this period, ½ hour after sunrise and ½ hour before sunset. Rock excavation should not single out drilling and blasting. Punitive requirements should apply to all construction phases. Where the public is concerned, the specifications should require that the project will be managed in the best interests of the public and State. Traffic operations that are delayed more than 12 minutes from the detonation of explosives may be penalized at the same rate as other construction operations.

Slope ratios that change every quarter station probably will not produce the desired result as rock fragmentation is controlled by the dominant defect. It was nearly impossible to convey the concept that a "saw tooth" slope will be the end product of fragmentation whether explosives or other means were used. The long-term slope stability will adjust to the dominant rock defect structure. These usually reflect the natural landscape.

Slopes that reflect the natural landscape can be developed with controlled slope holes if they are detonated at least 24 hours prior to the main body of holes. Cushion holes and buffer holes must be designed with the main body of holes. It should be obvious that utilizing large diameter boreholes on 24" centers for slope development may not substantially alter the slope, but does not ensure that slopes will be stable in the long term. In the Tombstone Hill cut, residual stress precludes long term stability. Block fall failures are not acceptable where the slope height is greater than 100'. Other problems that occur are pinnacles of rock that develop between loaded boreholes due to inadequate subdrilling. New York State paid for the additional subdrilling on a square yard basis as the pavement performance far exceeded the cost of additional drilling.

Survey layout is critical. Staking the top of slope and toe allows a driller to drill to a target. The consultant must be able to rough stake areas to determine the amount of rock that has to be excavated. Drainage is difficult to lay out. This is due to the differences in elevation if the pavement is superelevated and the drainage structure is at acute angle. Highway drainage may require planning prior to excavation and all field personnel be aware of conflicts.

A 40' cut with a grade and superelevation can be a problem if rock conditions change substantially from the design. It is incumbent that project staff be notified and a determination if remedial work is required. If timely notice is not made, delays may be lengthy.

Summary

Is the requirement that the contractor retain a blasting consultant is a viable approach, and it is a qualified, no. The state's construction contracting process does not allow deviation from the plans and specifications. A model exists with the Federal Highway Administration for both parties to fund a blasting consultant. The intent is the for both parties to obtain the best available advice to fragment rock with explosives. A cooperative effort without an adversarial relationship will ensure the rock excavation will be accomplished in a manner consistent with the plans and specifications.

Blasting at midnight would never have occurred if the consultant was employed by both parties to the contract. A consultant acceptable to both parties must be engaged when the contract is signed, but it would be more useful when the low bid is accepted. This will allow the consultant to provide guidance on borehole diameter, explosive products, drill type, blast plan format, reporting, etc. A consultant must be able to make decisions agreeable to both parties, and work with the staffs of each entity and the public.

The contracting process has traveled a long road since the 1960's and we must never regress, but continue to strive for improvements. Projects with significant volumes of rock must have one person on site that is knowledgeable in the use of explosives.

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Elevated Catchment Areas
A Performance Report

By

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Abstract

In 1988, following a rockfall related fatality, the New York State Thruway Authority instituted a rock slope monitoring and remediation program. The initial slope survey identified 35 sites that were deemed to be in need of immediate attention. Among these sites were 4 that lie along a six lane section of roadway in Rockland and Orange counties, 30 to 40 miles north of New York City, along a section of highway that serves as the major commuter road for the area.

The subject slopes which produced 3 to 4 rockfall incidents per year, have heights that exceed 100 feet and setbacks that in some areas were as narrow as 13 feet. The AADT for this section of highway is in excess of 59,000 with speeds of 70 MPH being common in spite of a 55 MPH speed limit.

In the fall of 1989, the Thruway Authority let a contract for construction of a series of walls intended to create rockfall catchment areas on top of the backfilled area behind the walls. The walls, which are a tied back soldier pile and lagging design, vary in height up to thirty six feet and have Brugg cable net catchment fences twelve to eighteen feet high installed in the backfill at the top of the walls. The system was designed to withstand the 3-10 cubic yard rockfalls common at these sites with little or no damage.

In November 1995 a rockfall estimated at 250 cubic yards was experienced at one of these sites. The wall/fence system retained 99.9% of the debris. In spite of the high percentage of material retained 9 vehicles were damaged by running over rock fragments. Vehicle damage claims amounted to $10,000. Damage to the fence was limited to 2 braking elements and some chain link fence.

Introduction

When the elevated catchment areas were constructed(Cross, 1990) the Thruway Authority was aware that several rockfall incidents a year were occurring at each one of the five wall sites. These events ranged in size from individual particles in the two to three inch range to small rock slides, usually less than five cubic yards. Due to the height of the walls and the consequent possibility of unnoticed buildup of debris, the catchment areas are inspected in the spring and fall to monitor debris accumulation. By the fall of 1995, at the conclusion of five full yearly cycles, the accumulation varied from a sprinkling of individual particles to areas where there the debris was eight to nine inches deep and a few spot accumulations of three to four feet. During this same five year interval no debris reached the pavement. Based on these observations it was apparent that the elevated catchment areas were not only successful in retaining the normal range of rockfalls but that the events were far more numerous than had been reported. The only question remaining was what would happen if "the big one" occurred.

The "Big One"

In October of 1995, three days after the fall slope inspection, a rockfall of significant size took place. The slope where the failure occurred is a highly weathered gneiss with an overall slope of three vertical on one horizontal. Considerable fracturing is evident, much of it the result of over blast during the slope construction. The failure involved an area approximately thirty feet long twenty-five feet high and had an in place volume of approximately one hundred seventy cubic yards. The toe of the failure area is approximately seventy-five feet above the pavement and forty-five feet above the catchment area. The total catchment area width at this point is fifteen feet with twelve feet of this behind a Brugg cable net rock fence. The fence is eighteen feet high at this location with an opening between the rockface and the top of the fence of approximately fifteen feet.

Due to the effect of residual benches from the original construction and the overall roughness of the slope face, the rockfall debris' trajectory was such that it impacted the rock fence near the top. As the debris dropped into the catchment area it pulled the chain link overlay down burying it under the pile of rock. The impact of the slide fully deployed the braking elements on the tie back cables at the top of two of the support columns and pulled one the cables loose from its anchorage. The combined effect of the loss of the overlay, sagging of the netting due to debris accumulation and the tipping of the support columns as the braking deployed and pulled loose allowed some debris to penetrate and/or overtop the fence. Three wheelbarrow loads of small rock debris, under six inch diameter, were cleaned from the pavement. Three rocks in the twelve to fourteen inch range were also found and are believed to have cleared the top of the fence. An examination of the fence prior to the start of debris removal indicated that even though the fence was drooping in the impact area it was still fully functional in the surrounding areas.

The result

The failure occurred at 5:30 PM on a Saturday evening during a period of heavy rainfall. A nearby weather station was reporting accumulations in excess of two inches per hour and greatly reduced visibility at the time. Traffic, although not as heavy as during the comparable weekday period, is still quite heavy during this time period. The combination of these factors resulted in nine vehicles being damaged from driving over the small amount of rock that reached the roadway. Six vehicles sustained tire damage with the others sustaining damage to engines and transmissions. No vehicles were struck by the falling rocks. There were no personal injuries associated with this incident. It is noteworthy to mention that even though the rockfall was on the Northbound roadway the most severely damaged car was in the Southbound roadway.

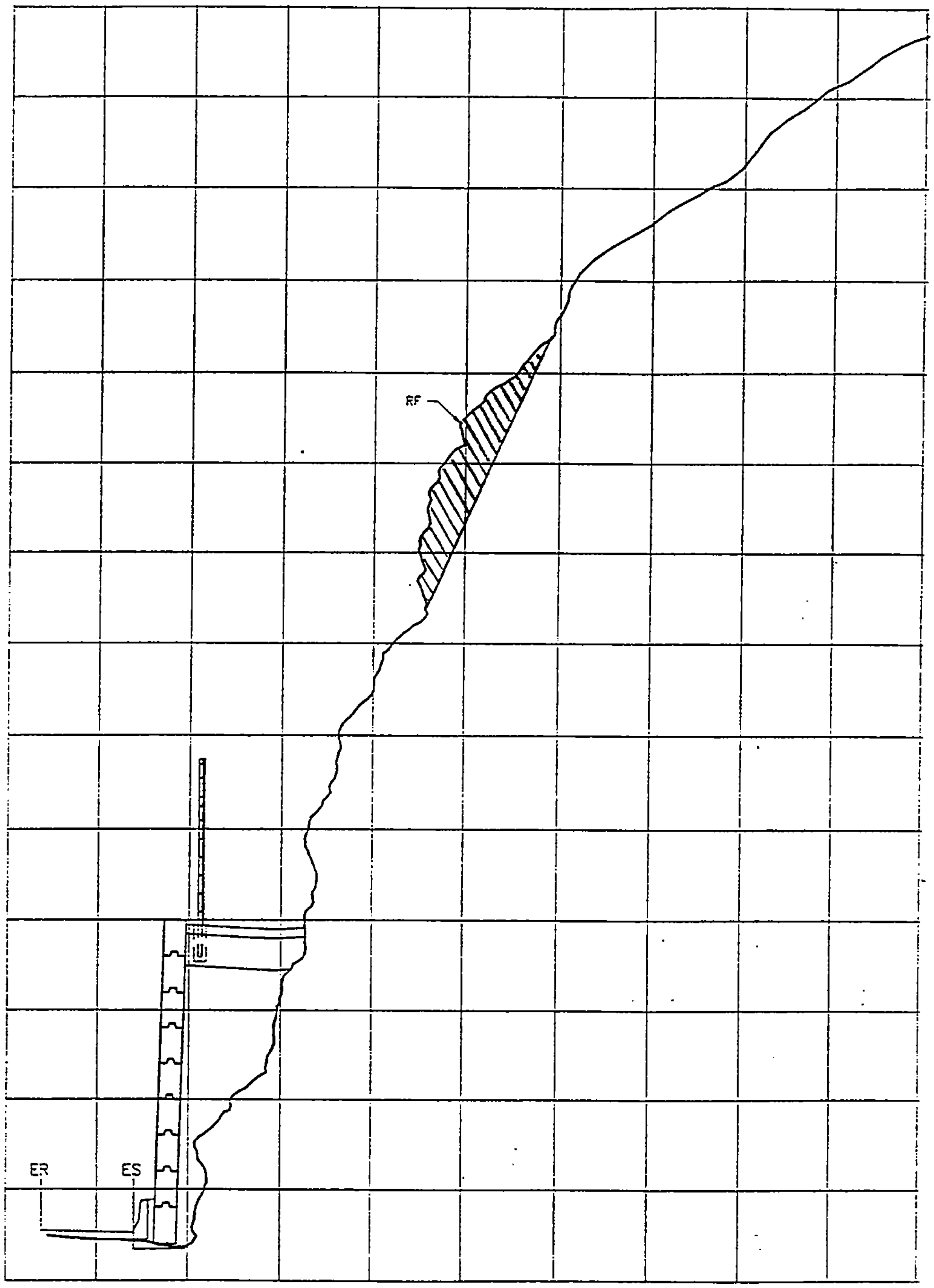

Fig. 1

Photogrametric cross-section of failure area

Conclusions

1. They work - As noted above the elevated catchment areas have had 100% success in stopping the normal size events while sustaining no damage.

2. They work very well - far better in fact than had been anticipated. The failure described here was seventeen times greater than what had been designed for as a zero damage event. Damage to the system was limited to replacing the two upper braking elements and forty feet of chain link overlay. Included in the retained debris were six rocks in the one cubic yard size range. If these rocks had reached the pavement it is almost certain that there would have been personal injuries associated with this event.

3. A few minor changes will provide even greater security - All of the chain link overlay has been seamed both horizontally and vertically to increase the resistance to being pulled down during a rockfall event. It is believed that this change could have prevented most, if not all, of the debris from reaching the pavement.

4. Murphy's Law still applies - stopping 99.95% of the debris will not be sufficient if there is enough traffic.

Epilogue

During the fall 1996 inspection of this slope a six inch wide horizontal crack was observed at the crest of the slope approximately one hundred seventy feet North of this failure. The crack could be traced thirty feet horizontally and an associated vertical crack could be traced for thirty five feet. Assuming a catastrophic failure of the entire affected area the projected in place volume was one hundred ninety cubic yards. The movement appeared to be very recent and is believed to be associated with a period of extremely heavy precipitation six weeks prior to the inspection. In addition to this storm 1996 was a very wet year, at the time of the inspection in November rainfall accumulation was over eleven inches above normal for the year.

Based on the experience with the 1995 failure and the fact that we had prior warning, not only of the possibility of a failure but also the potential effect of even small amounts of debris reaching the pavement, the Authority opted for zero tolerance at this site. In January 1997 nine thousand square feet of wire rope mesh drape was installed over the upper forty feet of this slope.

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Selecting Shear Strength Parameters for Weathered Metamorphic Rock

by

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Abstract

Strength parameters for soils and rocks can be selected from laboratory tests performed on undisturbed samples obtained from boreholes but intermediate materials or partially weathered rock cannot typically be sampled in boreholes. The NCDOT had some success sampling the weathered biotite gneiss from excavated test pits during exploration for the A-10 project, the proposed extension of I-26 to the TN-NC state line.

The weathered rock specimens were classified using a scheme proposed by Lee and de Freitas to classify Korean granites using both geologic and engineering properties to assign materials to one of six weathering categories; fresh (F), slightly weathered (SW), moderately weathered (MW), highly weathered (HW), completely weathered (CW), and residual soil (RS). The foliation strength. The mass strength or foliation strength laboratory results were interpreted for each weathering grade tested using Hoek and Browns model and their parameters σ_c, m, and s. The sliding friction along joints was interpreted for each weathering grade using Barton's model and his parameters, JCS, JRC, and ϕ_b '.

Introduction

To design stable cut slopes in the Blue Ridge and Inner Piedmont geologic provinces of North Carolina, South Carolina, and Tennessee, geotechnical engineers need to select strength envelopes that describe the range of gradually stronger materials derived from the weathering of metamorphic rocks. The soils having SPT N-values less than 20 can be sampled using Shelby tubes and tested in the laboratory by either triaxial or direct shear tests. For the parent rock strong enough to be cored, the discontinuities can be tested in direct shear to measure the sliding friction and in unconfined tests to measure the uniaxial compressive strength. Intermediate materials or partially weathered rock (PWR) cannot typically be sampled in a borehole.. If all the materials in the subsurface profile can be classified based upon degree of weathering then strength parameters for intermediate materials can be selected by interpolating between rock and soil parameters.

Weathering Grades

Generally weathering profiles derived from igneous rock have been investigated much more intensively than profiles derived from parent metamorphic rocks. Igneous rocks are, in general, homogeneous in composition and have more isotropic properties than metamorphic rocks, which typically contain cleavage and additionally may show disjunctive or compositional layering. Therefore, properties of metamorphic rock are anisotropic and foliations influence weathering.

Lee and de Freitas (1989) proposed a scheme to assign a weathering grade to Korean granites based upon both geologic and engineering properties. As shown in Table 1 their scheme consists of six weathering categories; fresh (F), slightly weathered (SW), moderately weathered (MW), highly weathered (HW), completely weathered (CW), and residual soil (RS)..

Table 1: Weathering Grades (Lee and de Freitas, 1989)

Grade	Term	Abbreviation	Type of Material
I	Fresh	F	Rock
II	Slightly Weathered	SW	Rock
III	Moderately Weathered	MW	Rock
IV	Highly Weathered	HW	Rock/Soil
V	Completely Weathered	CW	Soil
VI	Residual Soil	RS	Soil

The rock material is qualitatively evaluated using measures such as ease of breakage with a geologic hammer, scratching with a knife, and friability of NX cores. Index tests such as point load, Schmidt hammer, hand penetrometer, uniaxial compressive strength, and slaking tests provide quantitative measures. Chemical and physical weathering is assessed individually and the rock material assigned to a weathering grade based on the most severe weathering present.

Shear Strength Model

We selected shear strength models that included parameters that could be related to index properties and descriptive features of the weathered rock. The Hoek and Brown (1980) failure criterion was selected for rock mass strength and Barton's (1978) criterion was selected to describe sliding along joints. Hoek and Brown's criterion given in equation (1) was originally defined in terms of principle stresses:

$$\sigma_1' = \sigma_3' + \sigma_c \left(m \frac{\sigma_3'}{\sigma_c} + s \right)^{1/2} \tag{1}$$

where σ_c equals the uniaxial compressive strength of the intact rock and m and s equal empirical constants. Hoek (1983) provided a table with suggested m and s values based upon rock mass quality and mineralogy. For example a fair quality coarse grained metamorphic rock with several sets of moderately weathered joints spaced at .3 to 1 m spacing would have m = 0.5 and s = 0.0001 while a poor quality metamorphic rock with numerous weathered joints at 30 to 500 mm with some gouge would have m = 0.13 and s = 0.00001. These values provided a starting point for curve fitting of the foliation strength of MW and HW specimens.

Bray (Hoek (1983)) derived relationship between τ_{ff} and $\sigma_n{}'$ using the parameters from the empirical criterion.

$$\tau_{ff} = \left(\cot\phi_i{}' - \cos\phi_i{}'\right)\frac{m\,\sigma_c}{8} \tag{2}$$

Equation (2) gives the overall relation that includes the instantaneous friction angle, $\phi_i{}'$, given by equations (3) and (4).

$$\phi_i{}' = \arctan\left(4h\cos^2\left(30 + \frac{1}{3}\arcsin h^{-3/2}\right) - 1\right)^{-1/2} \tag{3}$$

$$h = 1 + \frac{16\left(m\,\sigma_n{}' + s\,\sigma_c\right)}{3\,m^2\,\sigma_c} \tag{4}$$

The friction along joints was described by Barton's (1978) model given in equation (5).

$$\tau_{ff} = \sigma_n{}'\tan\left(JRC\log_{10}\left(\frac{JCS}{\sigma_n{}'}\right) + \phi_b{}'\right) \tag{5}$$

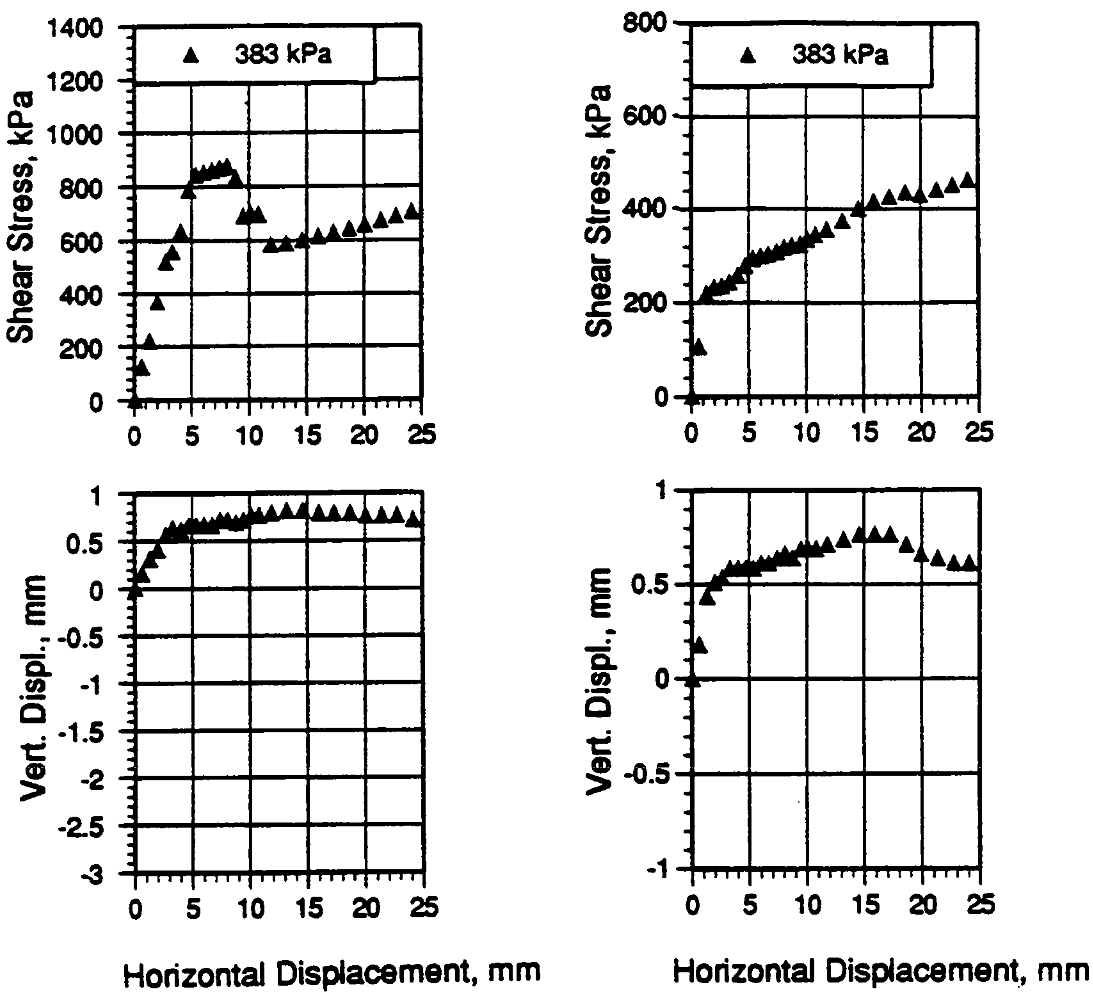
where JRC equals the joint roughness coefficient and JCS equals the joint wall compressive strength. The JRC varies between 0 and 20 with increasing roughness and JRC equals σ_c for unweathered joint walls and $\frac{1}{4}\,\sigma_c$ for weathered joint walls.

Sampling and Testing

The A-10 project will extend I-26 north from Asheville, NC to the Tennessee state line at Sams Gap. The alignment follows existing US 19-23 and includes 9.5 miles of new roadway. There will be 11 areas that require cuts ranging in depth from 100 ft to 300 ft. measured at the roadway centerline. The cuts include soil, weathered rock, and rock. The A-10 project lies within the Blue Ridge Belt which is composed of granitic gneiss, biotite gneiss, schist, quartz monzonite, and migmatitic gneiss. Rocks within the Blue Ridge Belt exhibit extensive metamorphism, faults, and joints resulting from thrust faulting. Outcrops in the area trend NE-SW at 45° - 80° and dip 30° - 60° SE. Colluvial and saprolitic soils are classified as SM and ML

Sweitzer (1995) described the exploration and investigation of weathered rock performed for the A-10 project. After trying unsuccessfully to core and then cut block samples of weathered rock the NCDOT subcontractor, Trigon, cut sets of three 102 mm (4") diameter and 152 mm (6") long cylindrical weathered rock samples from 15 different locations selected from 8 of the 11 designated cuts. NCDOT trimmed a face either perpendicular or parallel to the observed foliation and then cut samples using a portable concrete coring drill employing compressed air to flush cuttings. The 76 mm (3") diameter rock samples were taken from both standard and oriented NX cores.

The weathered rock samples were tested at their natural water content by Singleton Laboratories in direct shear to measure the mass strength. The 102 mm diameter specimens fit in 102 mm diameter metal platens placed inside a 300 mm square direct shear box. After trimming specimen ends to fit the test box sulfur capping compound was used to provide a snug fit. Weathered rock specimens were sheared at a horizontal displacement rate of 0.13 to 0.25 mm/min. up 25 mm. Figure 1a shows a sample test on a MW specimen sheared parallel to the foliation at $\sigma_n{}'$ equal to 383 kPa. The sheared specimen was returned to its initial position and then sheared three more times at $\sigma_n{}'$ equal to 96 kPa, 192 kPa, and 383 kPa. Figure 1b compares the fourth shear test performed on the MW specimen tested to peak strength results shown in Figure 1a. The results of the shear stages performed after a discontinuity had formed during the first shear stage were referred to as the ultimate strength and were used to determine the joint sliding strength for weathered rock specimens.



(a) Peak Strength (b) Ultimate Strength

Figure 1 Peak and Ultimate Direct Shear Test Results on MW Specimens

The weathered rock specimens were each visually inspected to identify qualitative factors including:

1. Can specimen be scratched with a knife?
2. Can specimen be indented with a geologic hammer?
3. Can NX size piece be broken by hand?
4. Is specimen mostly mafic or felsic?
5. Are the failure surface or material mass stained?
6. General notes such as grain size, presence of mica, and foliation.

In addition quantitative factors were measured including water content, dry density, compression wave velocity, and uniaxial compressive strength. The compression wave velocity of weathered rock specimens was measured in m/s along the vertical axis using a V-meter that measures the Ultrasonic Pulse Velocity and the uniaxial compressive strength was estimated by a point load apparatus (Broch and Franklin, 1972).

The weathered rock specimens tested in direct shear were classified according to weathering grade based upon qualitative factors. For example one specimen could not be scratched with a knife, indented with a hammer, or broken by hand but because of staining on discoloration on the specimen but not on the failure plane was classified MW. Another specimen was classified HW because it was scratched by a knife, was broken with difficulty be hand, was not indented by a hammer, and the failure plane had some slight staining. Of the 45 cored weathered rock specimens 24 were classified MW, 18 HW, and 3 CW.

In addition the rock cores tested in uniaxial compression and at the NCDOT laboratory and six of core specimens tested by Singleton labs in direct shear along discontinuities that fit together tightly as joints without any soil infilling. The rock joint was oriented at the shear box separation and were snugly fit into the 102 mm diameter platens using sulfur capping compound. Each specimen was sheared at 0.13 to 0.25 mm/min horizontal displacement rate and were tested at three different σ_n' values. The τ_{ff} selected for evaluation was the value measured just past peak at a horizontal displacement of 8 mm. The same procedure was used for selecting the τ_{ff} used in evaluating the direct shear test results on weathered rock specimens. In addition six rock core specimens were cut by a saw, ground flat and then dressed using 240 grit sandpaper to eliminate polished areas and grinding ridges. Of the 54 core specimens tested by NCDOT in uniaxial compression 16 were classified SW, 36 MW, and 2 HW. All 6 direct shear specimens were SW.

Table 2 summarizes the dry densities and compression wave velocities ranges for specimens classified by qualitative factors. The measured uniaxial compressive strengths indicate that SW rock core specimens had σ_c values of 120 to 180 Mpa and point load test measurements indicated MW specimens had σ_c values of 150 to 335 kPa, and HW specimens had σ_c values of 30 to 90 kPa.

Table 2 Dry Density and Compression Wave Velocity for Different Weathering Grades (from Sweitzer, 1995)

Weathering Grade	Dry Density $^{g}/_{cm^3}$	Compression Wave Velocity $^{m}/_{sec}$
I	> 2.8	> 5500
II	2.6 - 2.8	4000 - 5500
III	2.4 - 2.6	1500 - 4000
IV	2.0 - 2.4	500 - 1500
V & VI	< 2.0	< 500

Evaluation of Shear Strength

Figure 2 shows the sliding direct shear test results for specimens of weathering grades II, III, and IV and selected envelopes described by equation (5) using Barton's model parameters JRC and JCS. The basic friction angle, $\phi_b{}'$, was determined from the direct shear test results on sawcut joints. The data show considerable scatter and while the Barton curves follow the general trend of increasing sliding friction with decreasing weathering grade they do not provide a good fit. The measured data for both sliding and foliation strengths had both spatial and parent material variability.

In figure 3 the mass strength of MW specimens as represented by the foliation strength have been compared to a linear regression envelope, the Barton sliding envelope for moderately weathered specimens and the Hoek and Brown envelopes for both MW and HW materials. The Hoek and Brown envelope for MW falls near the upper range of results while the HW envelope represents an average trend. In figure 4 all the measured foliation results have been plotted without any envelopes. This plot shows while the foliation strength generally increases with decreasing weathering grade many points overlap. Table 3 summarizing a set of parameters for both mass strength (foliation) and sliding strength for different weathering grades. These values were not selected by statistical curve fitting and are based upon judgement.

Table 3: Shear Strength Parameters versus Weathering Grade

WG	σ_c, kPa	Point Load kPa	JCS, kPa	JRC	m	s
II	150,000		180,000	7		
III		250	80,000	5	0.5	.0001
IV		60	45,000	4	0.5	.00001
V			10,000	-	0.5	0

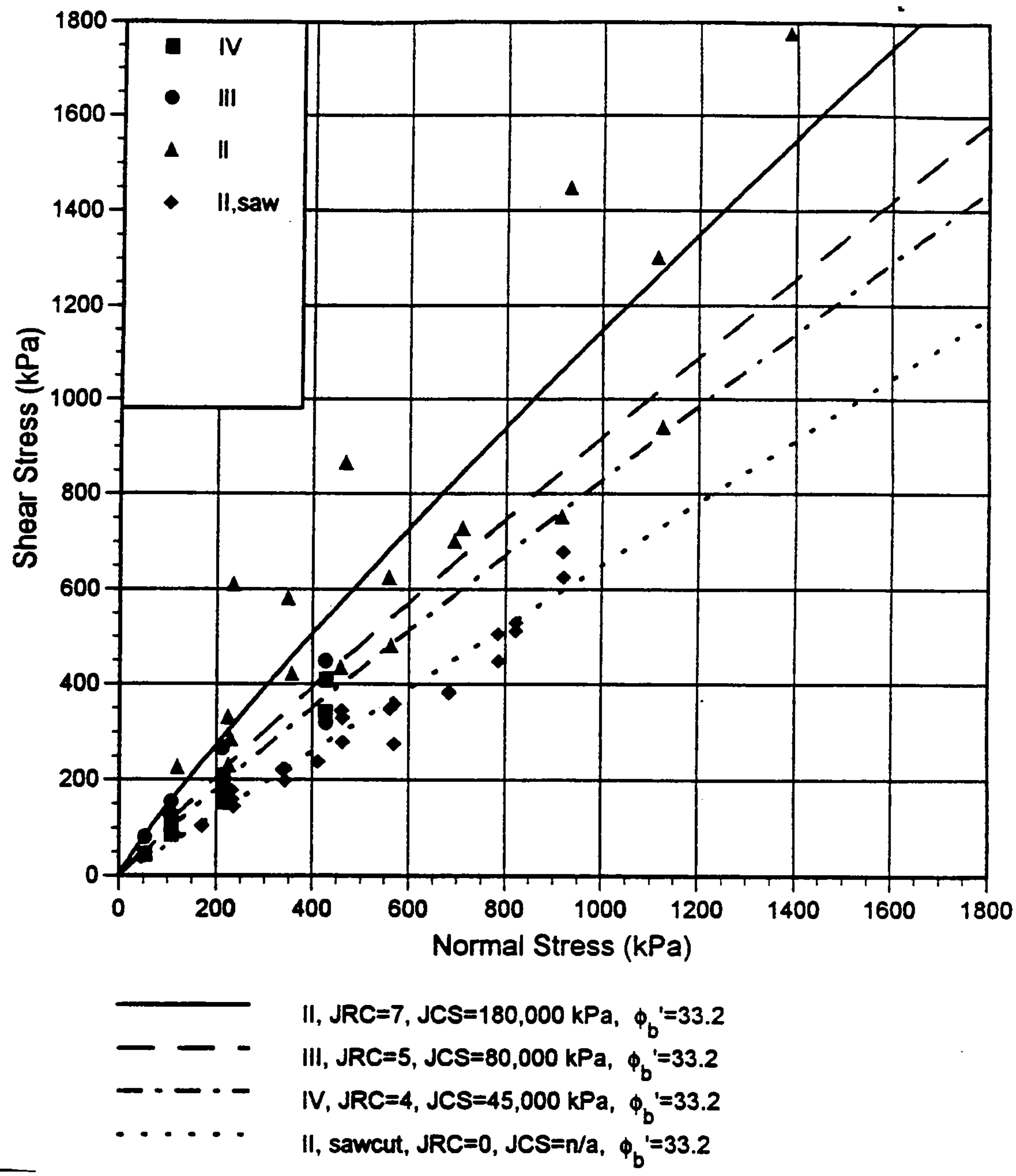


Figure 2 Joint Strength results and Barton envelopes

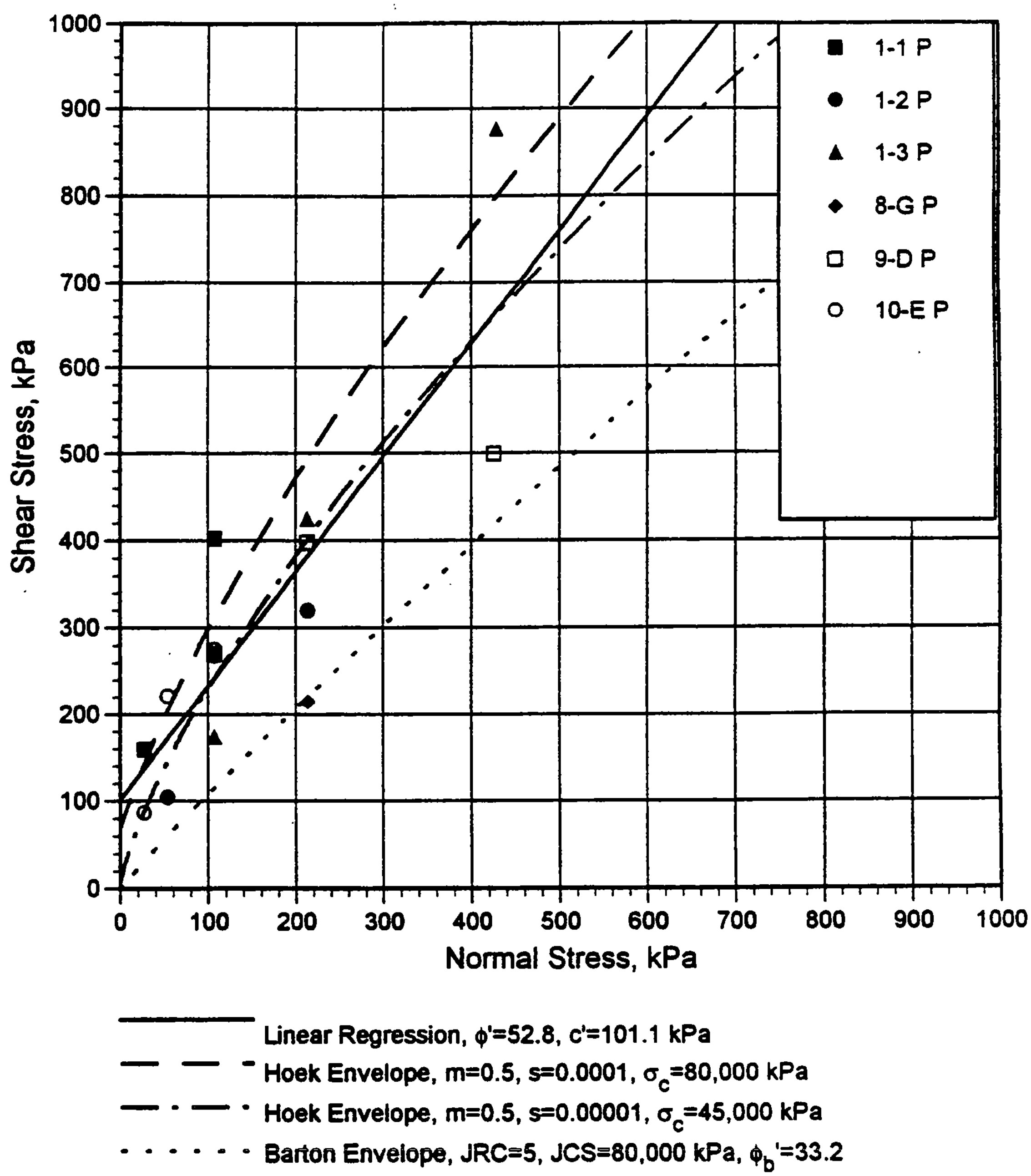


Figure 3 Foliation strength for MW specimens

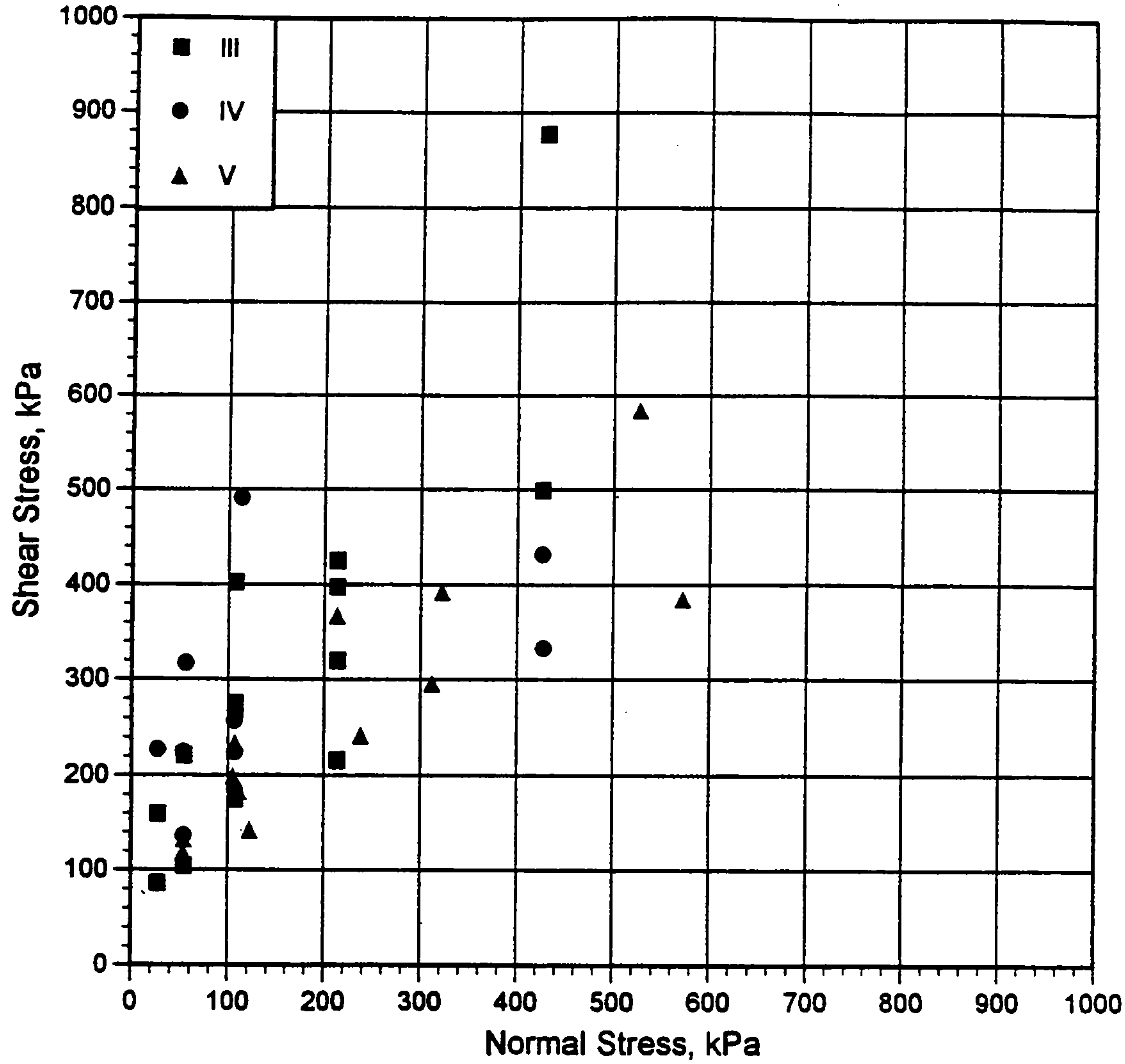


Figure 4 Foliation strength for all weathering grades

Summary and Conclusions

Lee and de Freitas (1989) recommended classifying materials within weathering profiles of Korean granites to help select properties for analyzing engineering behavior. This paper used their proposed framework to assign weathering grades to weathered metamorphic rock samples taken by NCDOT for design of the A-10 project and then selected different model parameters for each weathering grade tested. The test results for eight different cuts were combined to provide a sufficiently large database that correlations could be developed. It was hoped that weathering grade could be used to select parameters for highly weathered (HW) materials by interpolating between strength results on cored samples of moderately weathered (MW) samples and soil samples of completely weathered (CW) materials.

While these results provide a starting point new projects design will still have to be based upon efforts to sample and test the site specific materials. For example Beard (1997) has described how the parameters used in design of A-10 were selected based upon the site specific test results. Further investigation needs to include both methods to obtain good quality highly weathered rock samples from boreholes and improve index tests to use in classifying weathered rock samples.

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PARAMETERS USED FOR SLOPE DESIGN OF I-26 IN NORTH CAROLINA

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ABSTRACT

Interstate 26 will traverse through 13 miles of mountainous terrain with rock cuts exceeding 400 feet in depth. To design the safest and most economical slopes, North Carolina used an innovative probabilistic design procedure. This procedure evaluates the rock cuts for possible wedge failures. A wedge will be formed when two discontinuities strike obliquely across the slope and their line of intersection daylights into the slope face. If the alignment and weight of the wedge is sufficient to overcome the shear strength in the discontinuities, then the wedge will slide out of the rock cut. To investigate the stability of these wedges, a number of input parameters were developed. These parameters were then incorporated into a statistical mathematical model that was used to design the final rock slope angle. The following information discusses how these input parameters were obtained and what values were used for the probabilistic design.

INTRODUCTION

The third section of Interstate 26 (A10-C) was let to contract on September 17,1996. This 5.908 mile section will be another leg in the corridor that will run from Asheville, North Carolina to Erwin, Tennessee. The company of Gilbert Southern was awarded the contract for $105,646,973.35 and is currently mobilizing to begin the massive movement of 25,800,000 cubic yards of material. Four large rock cut areas will be needed to bring the roadway section down 450 feet to grade. This project is currently the second largest dollar amount the state has awarded in a single contract. The contractor has five years to complete the project, which will coincide with the completion date of December 31, 2001 for the entire corridor. The next section (A10-D) will be let to contract on December 17,1997 and is currently being designed by North Carolina Department of Transportation (NCDOT).

PROBABLISTIC DESIGN

An innovative risk assessment and decision analysis procedure was used to help design the slope angles for the rock cuts. This procedure uses decision factors or attributes to quantify the risks associated with a particular slope angle. The state contracted with Golder Associates to develop a probabilistic based program that will evaluate various slope angles based on 1) quantifying the risks of all relevant attributes 2) combining the attributes into dollars and 3) using tradeoffs to come up with a ranking system based on the total cost of the attributes.

Traditional wedge analysis (ref. Hoek and Bray 1974) was used to quantify the number and size of wedges that could be expected during construction and post construction. Slope angles of ¼:1(H:V), ½:1, ¾:1, and 1:1 were evaluated for each cut area. A Monte Carlo Simulation method was incorporated into three spreadsheets that was developed by Golder Associates for the NCDOT (ref. Roberds, W. and Wyllie, D., April 18,1986). The simulation could evaluate 1000 different wedges for each cut area. Each time a wedge was analyzed, new values were selected from input parameters that were defined by a mean and standard deviation. The program would randomly pick numbers for each parameter from these ranges and run a wedge analysis. Output would then include: the average wedge height, average wedge volume, number of potential wedges in the slope, total wedge volume in the slope, and probability of failure. These results were then input into another program that would convert these volumes to dollars and apply tradeoff values to determine which slope would receive the highest ranking. Seven decision factors were chosen to model tradeoff attributes. These factors included construction costs, post construction costs, worker safety, public safety, public relations, aesthetics, and schedule delays. The final rankings, along with the size and volumes of the wedges, were used to design an economical and safe slope.

GEOLOGY

The predominant rock types of granite, gneiss, and pegmatite depict the site geology. Biotite, mica, magnetite, and hornblende comprise the major accessory minerals. Uniaxial compressive strength tests on intact samples yielded strengths that ranged from 4000 psi to 30,000 psi.

The foliations typically trend in a southeast direction with dip angles ranging from 30 to 60 degrees. This orientation creates many planes that daylight into the western slope face which could have the potential for failure. Folding has caused many variations in these directions which is evident in the rock core as it changes depth and location.

Rainfall of 60 inches per year has created landslides that are unstable, whenever they become fully saturated. Many of these ancient landslides contain colluvial deposits which fill almost every hollow with some exceeding 40 feet in depth.

The project was divided into four major cut areas. Rock Quality Designation (RQD) values were taken for the entire core length with each cut area having the following averages: Area I=46%,Area II= 88%, Area III=78%, and Area IV=67%. Some core holes showed a 5 foot weak zone with RQD values of 0 to 20 %. These zones could vary in depth throughout the various core holes.

STRUCTURAL DATA

Structural data were obtained from field mapping of outcrops and logging of rock core holes. Rock core was placed into a goniometer and reoriented, so the dip and dip directions could be recorded. This information was then transformed into stereonets that provided a two dimensional representation of the discontinuities. The stereonets were then contoured so the foliation and important joint sets could be identified. Once identified, each cluster was represented by a mean dip and a mean dip direction. A standard deviation was used to outline the boundary of the cluster.

DIRECT SHEAR

Direct shear tests were conducted on samples containing foliations and on samples containing joints. The shear testing was conducted by three independent laboratories on samples taken from the rock coring. Each sample was tested at three different normal stresses with all the results shown on a shear stress vs. normal stress plot (Fig. I). A best fit line is plotted which shows an angle of 42 degrees. This graph includes bars that incorporate 25% of the data to show how much scatter is outside this range.

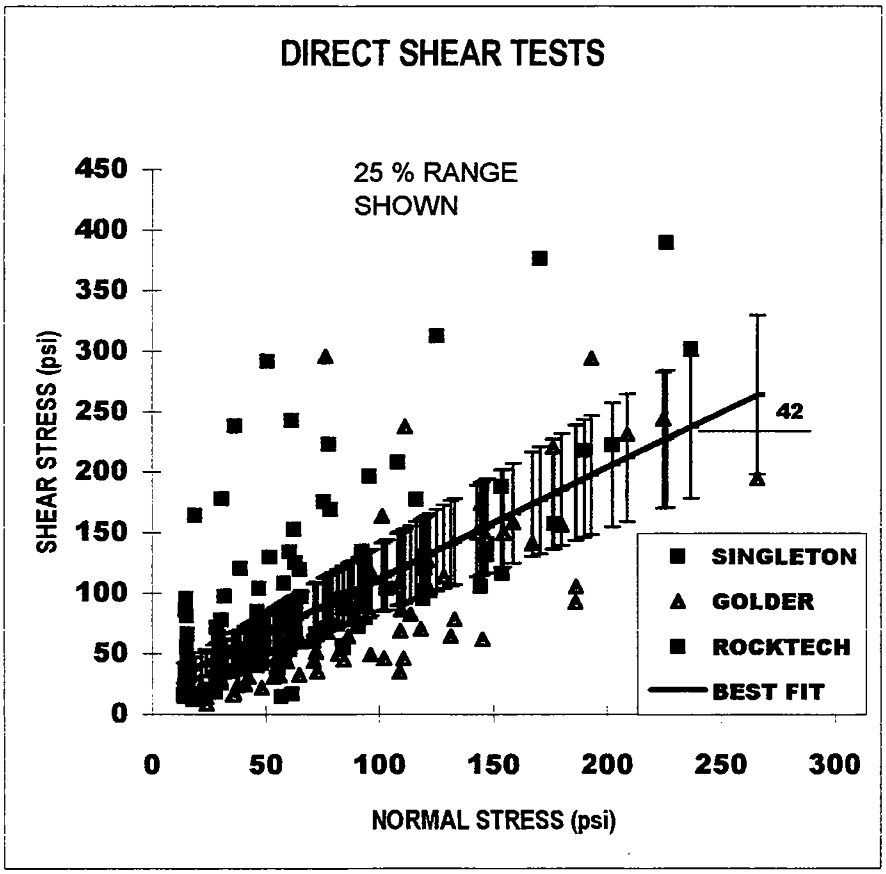


FIGURE I

The next three graphs (Fig. II, III, & IV) group the data by individual labs with a best fit line drawn. There is still some scatter which is probably due to the type of material and the varying amount of weathering in the joints. The samples exhibited very little clay infilling material, so a phi (φ) number was determined from the graphs, with the cohesion assigned a value of zero.

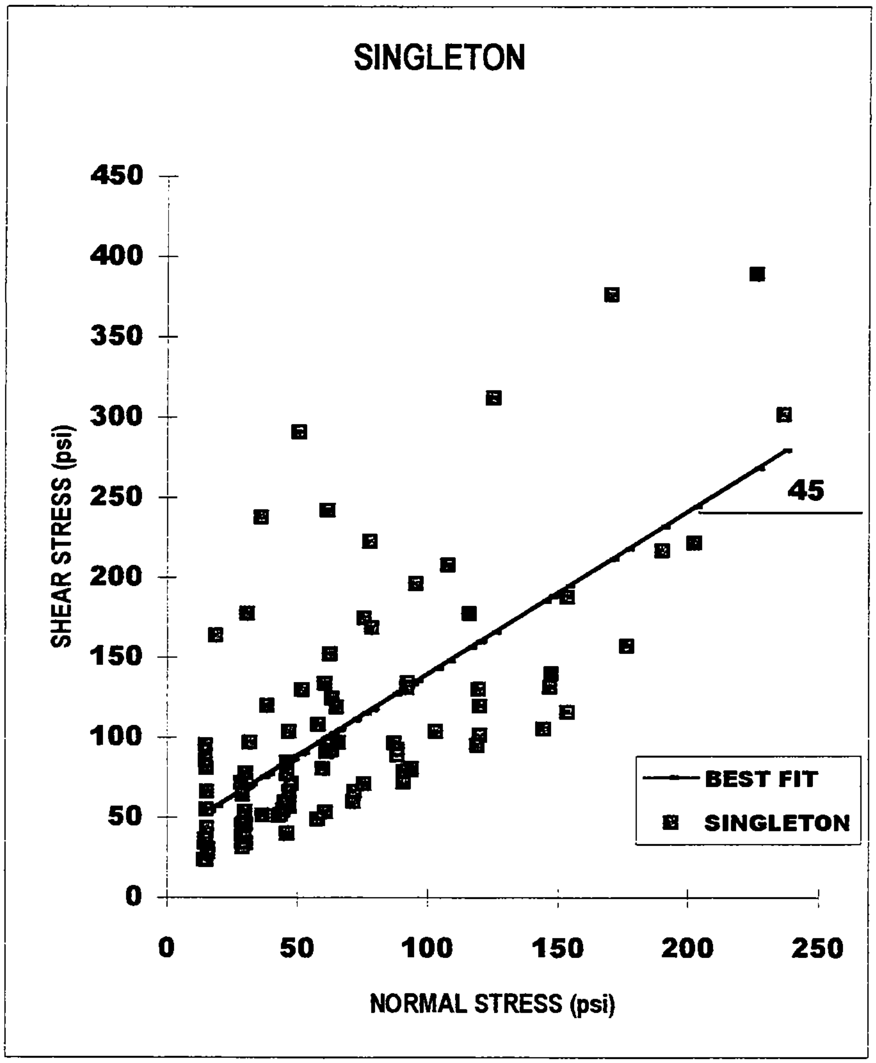
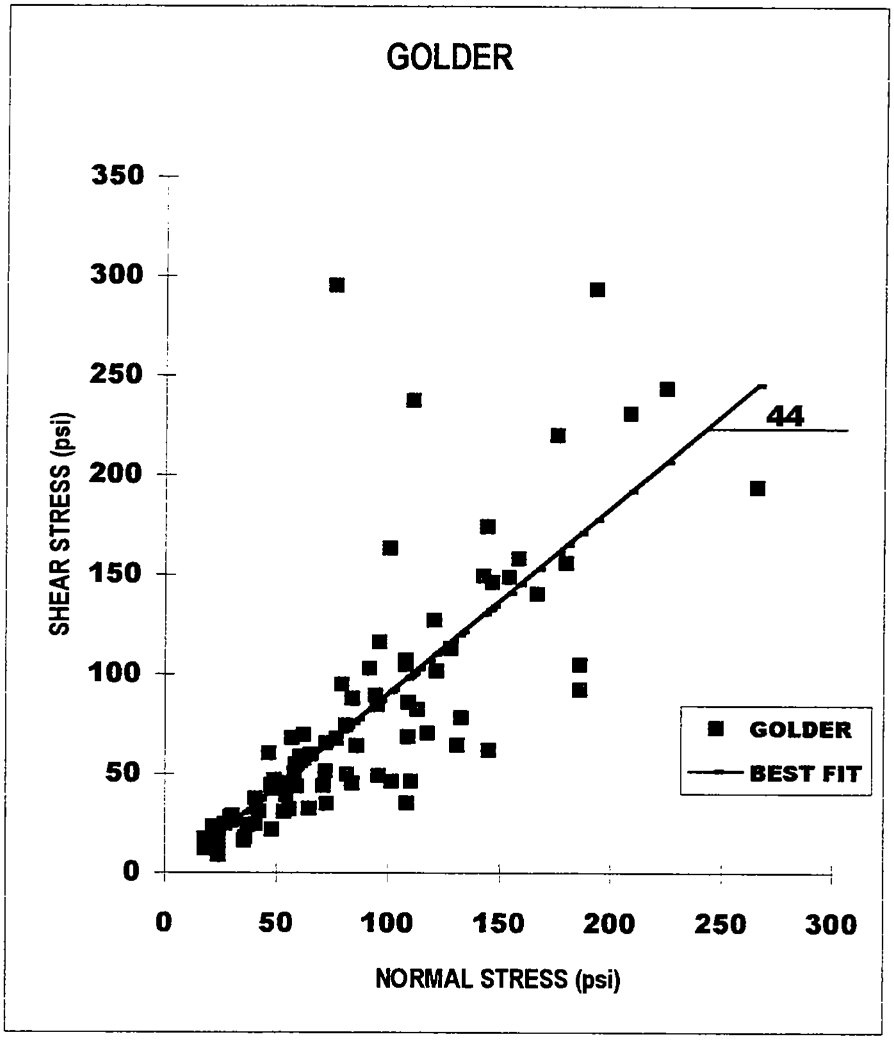


FIGURE II



GOLDER
350
300
250
200
150
100
50
0
SHEAR STRESS (psi)
0
50
100
150
200
250
300
NORMAL STRESS (psi)
44
GOLDER
BEST FIT

FIGURE III

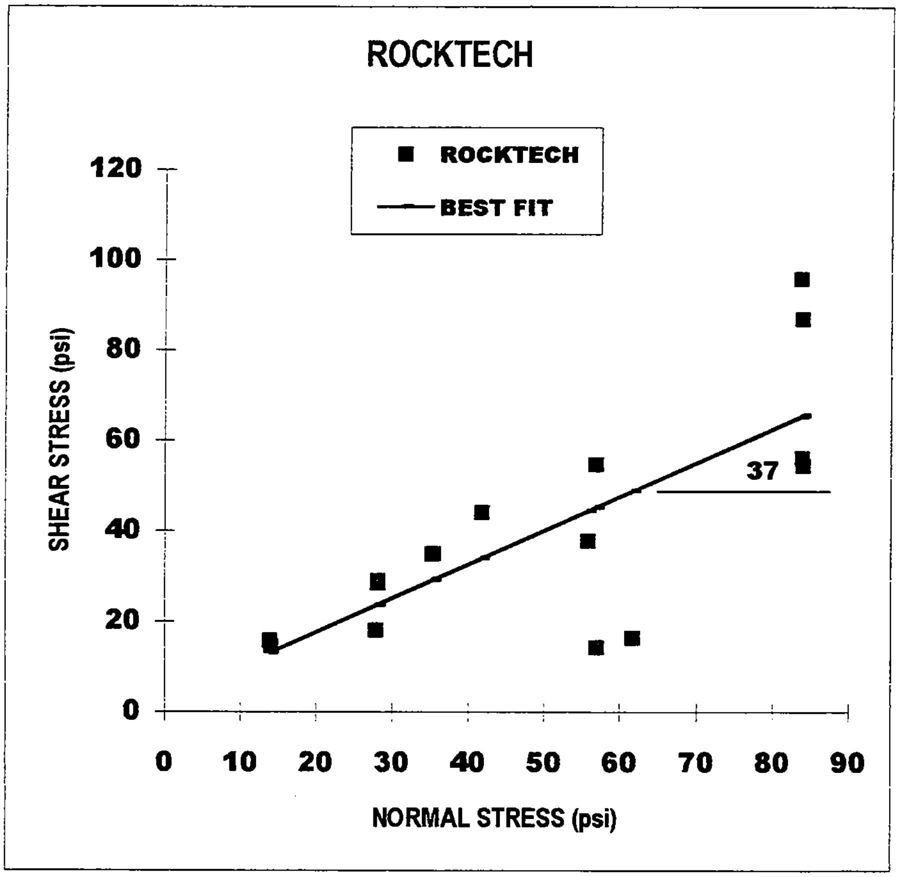


FIGURE IV

Four samples were sawcut and tested in direct shear which gave a ϕ=29 degree value for this biotite gneiss material. The sawcut samples indicated the true friction angle without any effect from roughness. The friction angle (ϕ) used for design with foliations was 29 degrees, with a standard deviation of 4. The friction angle used for design with joints was 40 degrees, with a standard deviation of 5.

ROUGHNESS

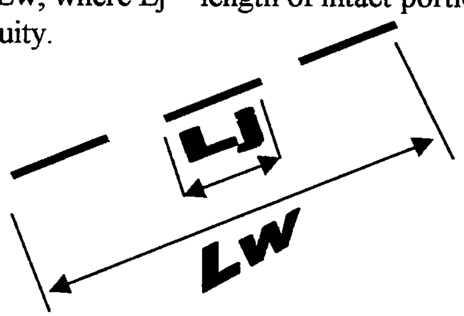
Roughness angles (i) were determined from waviness plate tests performed on nearby I-26 cut faces in Tennessee. These tests, using various plate sizes, determined the roughness of the fracture surfaces. The roughness angle is the difference in the dip angle of the plate compared to the average dip angle of the joint being measured. A discontinuity with a higher roughness angle will impart a greater sliding resistance and increase the total shear strength of the joint. For a wedge to fail, it will have to travel over or shear off these rough projections. The plate tests indicated an i of 6 degrees should be used for the foliations, while an i of 9 degrees should be used for the joints. These roughness angles combined with the friction angles were used for the analysis of the wedges. The combined (ϕ+i) angle for foliations was 35 degrees with a standard deviation of 4, while the combined (ϕ+i) angle for joints was 49 degrees with a standard deviation of 5.

ROCK MASS STRENGTH

Rock mass strength values were used to model the intact portion of rock that formed the sides of the wedge. Intact rock has both cohesion and friction and is significantly stronger than the open discontinuities. It is likely that a wedge formed by continuous open fractures could slide, while a small amount of intact rock would be sufficient to maintain stability of the wedge. To model this intact rock, Hoek and Brown's failure criteria for closely jointed rock mass was used (ref. Hoek and Brown 1988). Two dimensionless constants (m & s) are needed to define the interlocking of the individual pieces of rock within the mass. When laboratory test data are not available, Hoek and Brown recommend using a rock mass classification to develop these m & s values. The Council for Scientific and Industrial Research (CSIR) classification system was used for this purpose. A rock mass strength for intact foliations was calculated to have a cohesion of 1500 psf and a ϕ=30 degrees ;while the rock mass strength for intact joints had a cohesion of 7000 psf and a ϕ=50 degrees.

ALPHA VALUE

The proportion of intact rock in a wedge is unknown, so it can only be modeled by using probabilistic design methods. This uncertainty is defined by an alpha value where $\alpha = \sum Lj / Lw$; where Lj = length of intact portion of discontinuity, and Lw = total length of discontinuity.



The α could vary from totally intact joints to joints that are completely open. The probabilistic model would develop a Lj & Lw number and then calculate an alpha to use for each wedge analyzed. Strengths for the intact rock were developed from the rock mass method, while strengths for the open portions were derived from the direct shear and waviness plate tests.

FRACTURE LENGTHS

Fracture lengths will influence the size of wedges that may be formed. These lengths were determined from both outcrop mappings at the project and open highway cuts ten miles from the site. A negative exponential distribution was utilized to model the joint length that formed the side of the wedges. The foliations had a mean value of 8 feet and a standard deviation of 6 feet while the joints had a mean of 11 feet and a standard deviation of 12 feet. This distribution would allow for a few large wedges to have joint lengths of 150 feet, since some joints of this length can be seen in the Tennessee cuts.

GROUNDWATER

Groundwater fluctuation was measured in standpipes for a period of one year. These standpipes indicated water levels will vary in some holes by as much as 100 feet from the dry to wet season. Based on these large fluctuations, the groundwater was assumed to be a triangular distribution with water heights varying with the size of the wedges. Small wedges close to the surface will be less likely to be saturated when compared to the larger wedges that extend below the water table. Therefore, small wedges were assigned a mean value of 25% saturated, while large wedges were assigned a mean value of 75% saturated.

SUMMARY OF DESIGN PARAMETERS

TABLE 1

Parameter	Comment	Area I	Area II	Area III	Area IV
Uniaxial compressive strength	Laboratory testing on drill core	4000 to 27,900 psi mean (m)=16,700 standard deviation (s d)=7290	14,000 to 26,600 psi m=19,200 sd=4190	14,400 to 26,100 psi m=17,822 sd=3952	1100 to 30,300 psi m=11,608 sd=8694

RQD	Measured from core	45%	88%	78%	67%
Roughness (i)	Field mapping from waviness plate tests in Tenn.	Foliations=6 degrees Joints=9 degrees	Same as Area I	Same as Area I	Same as Area I
Rock mass strength for intact portion of sliding surfaces	Hoek m&s method with allowance for different rock strengths and RQD in each area	Foliations: c=1500 psf sd=500 φ=30 deg sd=5 Joints: c=5000 psf sd=1000 φ=50 sd=5	Fol: same Joints: c=7000 psf sd=1500 φ=50 sd=5	Same as Area II	Same as Area I
Fracture length	Outcrop mapping of open foliations and joints with a negative exponential distribution used	Foliation: m=8 ft. sd=6 Joint: m=11 ft. sd=12	Same as Area I	Same as Area I	Same as Area I
C & φ Strengths for open discontinuity	φ from direct shear tests combined with (i) roughness	Foliation: (φ+i)=35 degrees sd=4 cohesion=0 Joint: (φ +i)=49, sd=5 cohesion=0	Same as Area I	Same as Area I	Same as Area I

TABLE 1 (CONTINUED)

CONCLUSION

Information was gathered from borings, outcrops, and existing rock cuts and used to develop the input parameters for a probabilistic computer program. Table I lists these design parameters.

The analysis indicated many wedge failures were possible on the western side of Interstate 26. Based on the large volume of rockfall these wedges may generate, it was safer and more economical to flatten the western cuts to a 1:1 slope. This flatter angle undercut many of the steeper foliations and reduced the large failure volume. On the eastern cut slopes the analysis allowed for a steeper ½ :1 cut slope to be used. The rockfall volume risk was greatly reduced since the foliations dip into the mountain away from the highway.

These input parameters along with a probability based model helped design the cut slope angles for Interstate 26 in North Carolina. These parameters were invaluable in evaluating the economic feasibility and safety of this future transportation link between states.

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Fractal Analysis of Rock Joint Roughness

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Abstract

The shear strength of rock joints is an important concept in rock engineering. In order to estimate the peak shear strength, the roughness of the rock joint in question must be estimated. The procedure which has been used to estimate rock joint roughness required the visual matching of standard roughness profiles with those for which shear strength is to be determined. Here, fractal geometry is used to quantify rock joint roughness. The variogram fractal method is used to analyze roughness profiles for rock joints taken from the Foothills Parkway project and the US Highway 19E project, both located in Eastern Tennessee. A series of laboratory tests are conducted such that Joint Roughness Coefficients can be determined for the rock discontinuities. Two fractal parameters, the normalized fractal dimension, D_n, and the normalized fractal intercept, b_n, are then correlated with the Joint Roughness Coefficients. The results of this study showed the normalized fractal intercepts, b_n, obtained from the variogram method are positively correlated with the Joint Roughness Coefficients from laboratory tested samples. Equations relating the variogram intercepts to the Joint Roughness Coefficients are developed which show a logarithmic trend. These equations are then used in the prediction of peak shear strength.

Introduction

The shear strength of rock joints plays a key role in considerations related to highway geology. Particularly when dealing with cut and fill slopes through and over rock, tunnels and foundations. The subjective nature of visually estimating Joint Roughness Coefficients leads to subjectivity in Barton's peak shear strength criterion. In order to eliminate the uncertainty, a method based upon fractal geometry will be presented. The focus of this study will be rock discontinuities from the Foothills Parkway and US19E projects, both located in Eastern Tennessee. The joints will be profiled using a digital roughness gage to obtain digitized coordinates to be used in the variogram fractal method. Two fractal parameters, the normalized fractal intercept and normalized fractal dimension are then obtained from the variogram analysis of the digitized profiles. A laboratory testing program developed to back-calculate the Joint Roughness Coefficients using Barton's equation is discussed. The relationship between fractal parameters and Joint Roughness Coefficients will then be examined to determine which is the better parameter for the quantification of rock joint roughness.

Peak Shear Strength Model

As shown by Patton (1966), the peak shear strength of a rock joint is closely related to the surface roughness of the discontinuity. In Patton's model, the joint surface is idealized as a perfectly matched series of saw teeth. When the joint is subjected to normal and shear loadings, several things happen. At low levels of normal stress the asperities must be overridden, resulting in dilation. Dilation is an increase in volume of the specimen due to the overriding of asperities. This involves frictional sliding along the asperity surfaces. The sliding does not occur in the plane of the applied shear stress but occurs in a plane at an angle *i* to the shear plane. After the shear and normal stresses acting on the specimen have been resolved with respect to the angle *i*, we are left with the expression for frictional sliding along rock discontinuities at low levels of normal stress (Patton, 1966).

$$\tau = \sigma_n(\phi_b + i) \tag{1}$$

Equation (1) shows that the shear strength along a joint is a function of the applied normal stress (σ_n), the base friction angle of the material on which sliding occurs (ϕ_b), and the effective roughness angle (i). With increasing normal stress, dilation is inhibited. At a sufficiently large value of normal stress, asperities must be sheared in order for shear displacement to occur. For this condition it is assumed that the Coulomb relationship (eqn. (2)) is valid, where c is an apparent cohesion generated by the shearing of asperities.

$$\tau = c + \sigma_n(\tan\phi_b) \tag{2}$$

The two models presented above are only valid at the extremes of low and high normal stresses since the transition between overriding and shearing is not clear. Furthermore, it would be impossible to predict shear strength with Patton's model since the *i* value and the *c* value are not quantifiable without conducting shear tests.

In order to quantify the effects of joint roughness on shear strength, several sets of model discontinuities were tested by Barton and Choubey (1977). The results revealed that a series of constants corresponding to various levels of joint roughness can be used to model the joint in question. They termed the constant the Joint Roughness Coefficient (JRC). For example, a JRC of twenty represents the upper boundary for rough tension joints while a value of ten represents an intermediate range and a value of five models the behavior of planar foliation joints. The value of zero represents a perfectly planar surface which is for perfect rock to rock contact with no roughness-induced dilation or shearing of asperities. Tests were conducted until a range of ten coefficients, ranging from zero to twenty, matching a set of ten standard profiles were developed. The representative JRC value can then be used for the prediction of peak shear strength according to:

$$\tau/\sigma_n = \tan[(JRC)\log(JCS) + \phi_b] \quad \textbf{(3)}$$

where JCS is the Joint Compressive Strength.

Fractal Geometry

Recently, the subjective nature of visually estimating Joint Roughness Coefficients has been questioned (Miller et al., 1989; Wakabayashi and Fukushige, 1995). Several researchers have turned to fractal geometry to quantify the roughness characteristics of rock discontinuities.

Fractals occupy the borderline between Euclidean geometry and complete randomness. Euclidean geometry is the branch of mathematics that deals with the measurement and properties of points, lines, planes and volumes. Geometry also represents the arrangement of objects made up of points, lines, planes and volumes which, when combined, form a specific shape or figure. Shapes found in nature such as mountains, coastlines, river systems, clouds, trees, and an infinite number of other objects are not easily described by traditional Euclidean geometry.

Fractals allow scientists from a wide range of fields to quantify shapes that had been described with terms such as "*in between, pimply, pocky, ramified, tortuous, wiggly, wispy, wrinkled*" (Mandelbrot, 1983). Elevation profiles of joint surfaces belong to the class of functions that had heretofore been described by terms such as those listed above.

Fractal Analysis of Rock Joint Roughness

Variogram Method

There are numerous methods with which the fractal dimension of a rock discontinuity can be calculated. In this section the variogram method, developed by Mark and Aronson (Russ, 1994), will be presented.

In the variogram method, windows of varying width are positioned along the horizontal axis. The average sum of the squares of the differences in height between the points of the profile are then calculated for each window. Figure 1 shows the use of windowing to evaluate differences in elevations ($\Delta z = z(x_i)-z(x_i + h)$) for two window sizes, h , (figure 1a) and $2h$ (figure 1b). The one-dimensional variogram function, $\gamma(h)$ can be expressed in discrete form as:

$$\gamma(h) = \frac{1}{2N}\sum_{i=1}^{N}[z(x_i)-z(x_i+h)]^2 \quad \textbf{(4)}$$

where N is the total number of pairs of roughness profile heights with elevations of $z(x_i)$ and $z(x_i +h)$ spaced at the lag (window) width h. Several values of h are selected for which the respective values

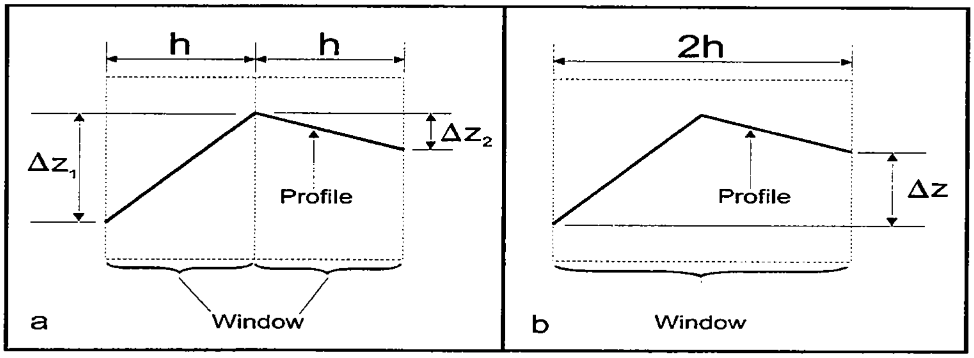

Figure 1 - A diagram showing how windows of varying width, *h*, are used in the calculation of Δz for the variogram method.

of $\gamma(h)$ are calculated. Log $\gamma(h)$ values are then plotted versus the log window widths (h) and the fractal dimension is related to the slope, β, of the resulting line according to:

$$D = 2 - \frac{\beta}{2} \tag{5}$$

where D is the fractal dimension.

Fractal Parameters

The only fractal parameter mentioned to this point has been the slope of the log-log plot, which has been shown to yield the fractal dimension. Obviously two parameters are obtained from the linear fit; the slope and intercept. The intercept of the linear fit has been largely ignored by all but a few researchers (Hsiung, et al. 1995; Kulatilake, et al. 1995). Ignoring the fractal intercept may have been in error, as the intercept yields the magnitude of roughness (Russ 1994). Figure 2 shows two linear fits generated with the variogram method using JRC standard profile number ten. The solid line is for the standard profile and the dotted line for the standard profile with the elevation values reduced by one-half. The plot shows how the intercept of the reduced-elevation standard profile was reduced significantly from the original standard profile intercept, yet the slopes are the same. It should stand to reason that the magnitude of roughness (intercept) would have more impact on shear strength than the variation of roughness with scale (slope). In this study both the fractal dimension and intercept will be examined.

Case Studies

The Foothills Parkway Project was conceived to link an existing portion of the Foothills Parkway near Townsend, Tennessee through Wears Valley to US Highway 321. The project, to be constructed through a portion of the Great Smoky Mountains National Park, was commissioned by the Federal Highways Administration, while the National Park Service had the responsibility of dealing with environmental and aesthetic concerns. The project was initially begun by the Tennessee Department of Transportation (TDOT) but was subsequently taken over by the Federal Highways Administration (FHWA). The geotechnical investigation was contracted by FHWA for certain aspects of the project.

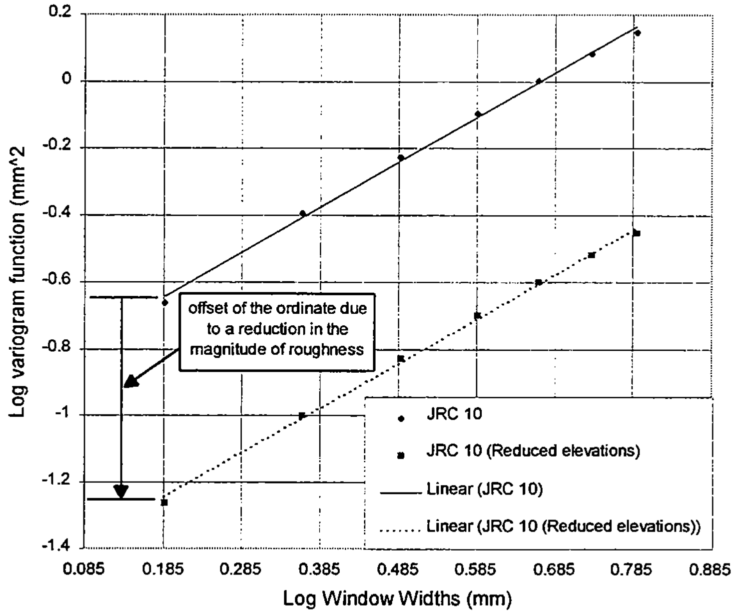

Figure 2 - A plot of log variogram function versus log window widths showing the effect of the magnitude of roughness on the ordinate of the power-law linear fit. The solid line corresponds to JRC profile 10; the dashed line to the same profile with the elevation values reduced by one-half. The slope, β, is the same for both cases.

The geotechnical contractor selected was HDR Engineering (HDR), Pittsburgh, Pennsylvania.

The geotechnical investigation by HDR was obligated to be conducted within the strict environmental guidelines imposed by the National Park Service. An ingenious plan was developed to satisfy environmental concerns. The sampling points, located near proposed abutment and temporary pier locations, were not readily accessible from a previously constructed access road. The solution was to use the access road to get as near as possible to the temporary pier locations and with a bulldozer construct drill pads. Then, using helicopters, drilling rigs were lowered onto the preconstructed drilling pads. Using this system, only small areas of the forest were compromised for the geotechnical investigation as shown in figure 3.

Samples for this study were obtained from rock core of the Shields Formation of the Walden Creek Group. These rocks are Precambrian age and are described as massive conglomerate, sandstone, and slate (Hardeman, 1966). The rocks are all low grade metasedimentary rocks. A typical box of rock core is shown as figure 4.

The next study site examined was a series of roadcuts along US Highway 19E (US19E) near Biltmore, Tennessee. The cuts, some of which measured forty meters in height, were made ten years ago, during the construction of US19E. Sampling was biased toward areas of the outcrop that were readily accessible i.e., less than a height of 1.5 meters. Samples that could easily be removed by hand collection were taken. After the rock surrounding the discontinuity had been loosened, the two halves of the discontinuity could be removed. The samples were then scored with chalk to indicate

Figure 3 - Core drilling on preconstructed drilling pads for the Foothills Parkway Project. Note that only small areas of the forest are compromised for the geotechnical investigation.

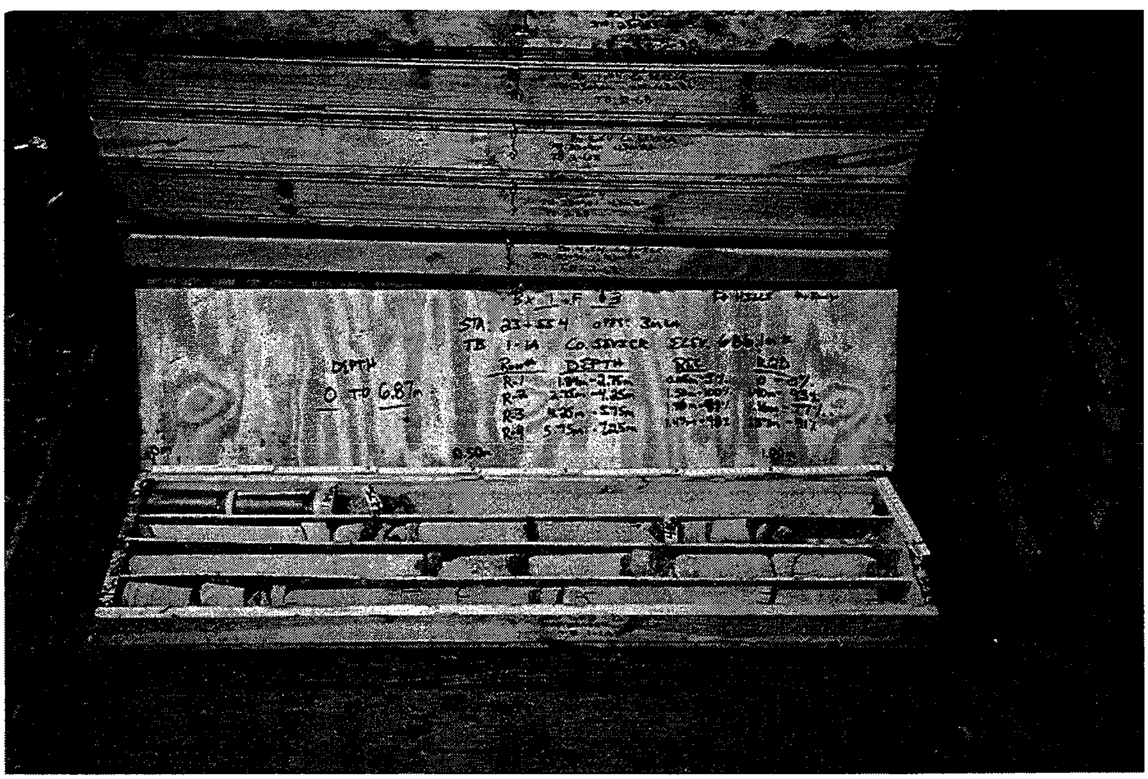

Figure 4 - A typical box of rock core obtained during drilling for the Foothills Parkway Project.

the direction in which they were to be sheared. This direction was determined based upon the presumed direction of the slope failure.

The area where the samples were collected is part of the Valley and Ridge province, dominated by numerous parallel narrow ridges, separated by linear valleys. Rock slopes along US19E were formed when these ridges were bisected by the highway alignment. The Ordovician-aged Sevier Shale comprising the rock slopes has been folded and faulted (Rodgers, 1970); although only low grade metamorphism has occurred. The thinly bedded shale at the site was folded in a large syncline. The bedding surface, in the trough of the fold, corresponds to the failure surface. Low grade metamorphism in the form of rock cleavage was also evident in the rock mass.

Testing

The focus of this study is to relate the fractal dimensions and fractal intercepts obtained the variogram method with JRC values used in Barton's equation. In order to achieve this goal, two testing schemes were developed. The first was to measure the surface roughness of the discontinuities, thus obtaining roughness profiles to be used in the variogram method. The second was aimed at obtaining all values necessary to back-calculate the JRC value in Barton's equation.

The surface roughness of the rock discontinuities was measured with a digital roughness gage provided by Mark McKeown, geologist with the U.S. Bureau of Reclamation in Denver, Colorado. The gage consist of four basic components; the gage, a parallel port interface, computer program, and a jig to secure the gage in position over the sample. The gage was first positioned into the jig over the sample such that the samples were profiled in the orientation in which they were to be sheared. A computer program written to convert the potentials read from the gage via the parallel port interface gage was then started. Elevation values (with an accuracy of 0.10 mm)were taken at 0.77 mm increments along the horizontal axis of the profile. This resulted in a text file containing the coordinates of the points composing the joint profile which can be plotted as shown in figure 5. The gage was then repositioned to the next profiling position and the process repeated.

The coordinates from the digitizing were then input into the variogram function and fractal dimensions and intercepts calculated for each profile. Since there were several profiles for each discontinuity, and each profile was of a different length, the fractal parameters were normalized. This was accomplished using:

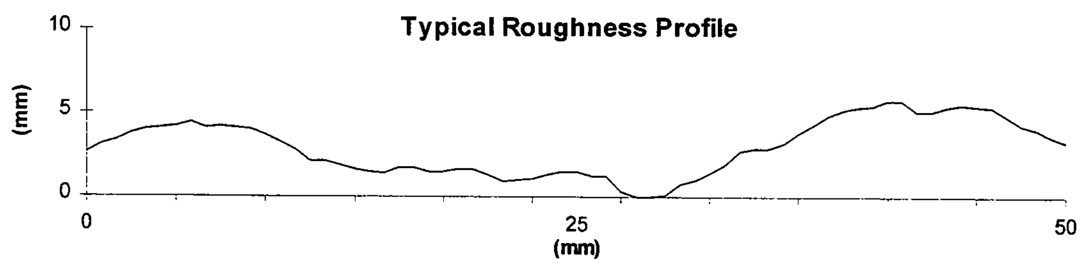

Figure 5 - Typical roughness profile obtained from the digital roughness gage.

$$D_n = \frac{\sum_{j=1}^{j} D_j L_j}{\sum_{j=1}^{j} L_j} \tag{6}$$

and

$$b_{n,\log} = \frac{\sum_{j=1}^{j} b_{j,\log} L_j}{\sum_{j=1}^{j} L_j} \tag{7}$$

where D_n is the normalized fractal dimension, D_j is the fractal dimension of the jth profile, L_j is the length of the jth profile, $b_{n,log}$ is the normalized fractal intercept in log form, and $b_{j,log}$ is the fractal intercept of the jth profile in log form. The fractal intercept was then converted from its log form by:

$$b_n = 10^{b_{n\log}}. \tag{8}$$

The values necessary to back-calculate JRC values from Barton's equation were determined from direct shear and point load testing. Values of peak shear stress, τ, and normal stress at failure, σ_n, were determined from the direct shear testing. Direct shear tests were also performed on smooth saw-cut discontinuities to determine the base friction angles, ϕ_b, of the various rock types.

Point load tests were conducted on the various rock types to determine representative values of the point load indices. The point load index was then used to estimate the unconfined compressive strength, σ_c, of the rock type in question. With the data from direct shear and point load testing the JRC values in Barton's equation could be calculated as:

$$JRC = \frac{\tan^{-1}(\tau/\sigma_n) - \phi_b}{\log_{10}(\sigma_c/\sigma_n)}. \tag{9}$$

Results

The Joint Roughness Coefficient values back-calculated from the experimental laboratory results should be related to some fractal parameter. Two fractal parameters were obtained from the variogram method: slope and intercept. The slope of the fractal log-log plot has been shown as related to the way in which roughness varies with scale. The slope of the fractal log-log plot yields the magnitude of roughness. Here, the parameters from the variogram method will be related to the Joint Roughness Coefficient values.

Normalized fractal dimensions, D_n, obtained from the variogram method were plotted versus the corresponding back-calculated JRC values. The resulting linear regression yielded a R^2 of 0.2066. This procedure showed that the fractal dimension showed poor correlation with the back-calculated JRC values.

JRCs back-calculated from laboratory data were plotted versus the normalized fractal intercepts, b_n. The resulting relationship followed a logarithmic curve given by:

$$JRC = (7.24)\ln(b_n) + 47.69 \tag{10}$$

with a R^2 of 0.7112. This procedure showed that the normalized fractal intercept was positively correlated with the Joint Roughness Coefficient.

To determine if the normalized fractal intercepts could be used in the prediction of peak shear strength; the fractal intercepts for samples of both the Foothills Parkway and US19E projects were substituted into equation 10. This yielded the fractal intercept based joint roughness coefficients. The fractal intercept based JRCs and all other necessary parameters were input into Barton's peak shear strength criterion, which yielded predicted peak shear strengths for the samples. The predicted peak shear strengths were then plotted versus the laboratory measured peak shear strengths (figure 6) . The statistical analysis of the data resulted in an R^2 of 0.9270 with a standard error of 197 kPa.

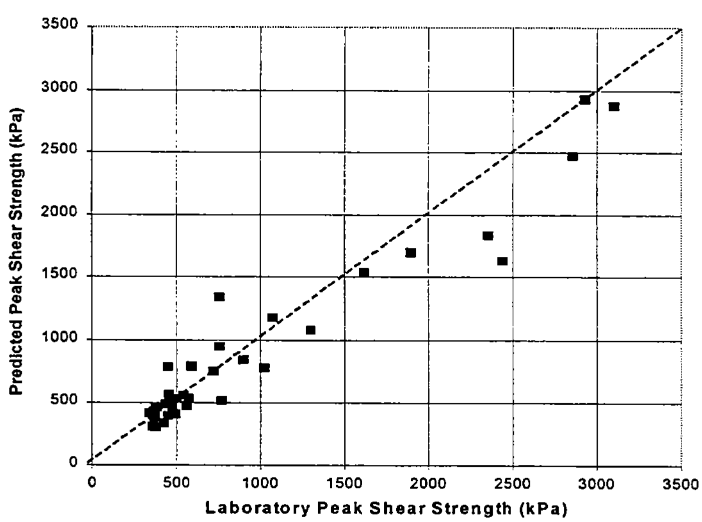

Figure 6 - Plot of predicted peak shear strength versus laboratory peak shear strength. The predicted peak shear strengths were obtained from Barton's equation using fractal intercept based joint roughness coefficients from the variogram method.

Conclusions

The results showed that the normalized fractal intercept was positively correlated with the back-calculated JRC values. An equation was developed to relate the normalized fractal intercepts to JRC values to be used in Barton's equation. Fractal intercept based joint roughness coefficients were then used to predict the peak shear strength of rock joints. The predicted peak shear strengths were closely correlated to the laboratory measured peak shear strengths.

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FIELDTRIP EXCURSION GUIDE

OVERVIEW OF THE GEOLOGY

By
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General

The fieldtrip traverses portions of two Physiographic provinces of the Appalachian Highlands - the Valley and Ridge in East Tennessee and the Blue Ridge in East Tennessee and Western North Carolina. The southern Blue Ridge is commonly referred to as the Great Smoky Mountains or the Unaka Mountains.

The characteristics distinguishing these provinces will be quite apparent on the trip route. These obvious differences distinguishing the provinces include: landforms and general elevation, stratigraphy and structure. The Valley and Ridge is notable for its long continuous ridges with alternating valleys developed through differential erosion of siliciclastic and carbonate sedimentary strata that strike northeast and dip monoclinally southeastward on the average between 30 to 50 degrees. Other than fracture cleavage developed in some argillaceous units, the rocks in the Valley and Ridge have not been metamorphosed. The stratigraphic section, composed mostly of early-middle Paleozoic-aged rocks, is repeated many times across the province due to imbricate thrust faulting associated with the Alleghenian orogeny (collision of proto-Africa with proto-North America about 200 million years ago).

The boundary between the two provinces is ordinarily very pronounced and is marked by the trace of a major fault thrusting late Precambrian or early Cambrian rocks of the Blue Ridge upon younger Paleozoic rocks of the Valley and Ridge. This portion of the Blue Ridge referred to as the Unakas or Great Smoky Mountains embraces most of the highest elevations of eastern North America. Although ridges and valleys of the Blue Ridge are not as distinct as in the Valley and Ridge, the highlands maintain a northeast-southwest trend. Complex structure in the Blue Ridge has resulted in some topographically high areas to be oriented transverse to the general NE-SW grain of the province. This has given rise to the formation of intermontane basins such as the Hot Springs area viewed on this fieldtrip.

In a general sense, rocks in the Blue Ridge are of two types: Grenville crystalline basement granite and granite gneiss around 1 billion years old, and thick sequences of metasedimentary rocks that formed from thousands of meters of sediment that accumulated in structural basins on a seafloor of Grenville Basement along the margin of the North American plate. The rocks have been profoundly deformed over a time span of several hundred million years during tectonism related to the acretion of the supercontinent, Pangea. Manifestations of this tectonism include: thousands of meters of graywacke derived from tectonically uplifted areas, metamorphism to at least kyanite grade, Precambrian glacial and volcanogenic deposits, and many rock discontinuities (joints, cleavage, faults, folded strata, etc.). Analyses of these discontinuities suggest at least three major tectonic events. The presence of these discontinuities has been and continues to be a geotechnical nightmare for the development of slopes during highway construction.

The last 200 million years of history in this region has been largely one of erosion - mostly by fluvial processes. Although none of the high areas in this portion of the Blue Ridge have been glaciated, beginning about 2 million years ago when Pleistocene glaciation commenced, the higher elevations in the Blue Ridge were subjected to periglacial climatic conditions of freezing temperatures for tens of thousands of years during each of the major ice advances. Much of the great volume of colluvium on the steep slopes of the Blue Ridge accumulated during this time. Stabilizing these deposits during highway construction in this province can be as challenging geotechnically as the many discontinuities in the bedrock. About 8 to 10 thousand years ago the last of the Pleistocene glaciers retreated and the climate in the Appalachian Highlands began moderating allowing fluvial processes to become a dominant force in landform development. Vegetation suited to the new climatic regime also returned. Only a few boreal species of trees - spruce and fir - remain as islands at the higher elevations as reminders of a once colder climate.

Valley and Ridge

The bedrock of the Valley and Ridge represents the westwardly thinning edge of a thick wedge of sediment that accumulated on the eastern shelf margin of North America throughout the Paleozoic Era. The stratigraphic column can be conveniently divided into three packages or Sequences bounded by unconformities of regional extent. The thickness' and facies of the strata within these packages reflect the tectonic activity along the eastern margin of the North American Plate (the assembling of the supercontinent, Pangea) during Paleozoic history.

Sequence I strata, representing a westward transgressing sea, is Early Cambrian through Early Ordovician in age and changes upward from terrigenous clastic rocks derived from western sources (Rome Formation) to carbonate rocks (Knox Group). The Rome Formation contains fossils and sedimentary structures (mudcracks, ripple marks, salt hoppers, etc.) that are highly suggestive of deposition in shallow water peritidal environments. The presence of evaporite deposits and the overall argillaceous character of the Rome makes it tectonically very incompetent; thus, it forms the hanging walls of many thrust faults in the province as well as the decollement (fault) separating the sedimentary sequences from the underlying basement - the Rome Formation is the oldest rock unit exposed in the Valley and Ridge.

The Knox Group, the uppermost unit in Sequence I, consists of dolostone and limestone formed mainly by algal buildups as the amount of terrigenous sediment transported to the continental margin continually decreased. Eventually the carbonate platform was subaerially exposed permitting the development of karst. This surface is now preserved as a paleokarst unconformity below the second sequence in the stratigraphic column. East Tennessee has several mining districts as the result of Mississippi Valley-type mineral deposits having formed in breccias associated with the paleokarst. The abundance of chert in the Knox is most evident as float in the residuum. It is the armoring by chert float in the residuum that allows the Knox to be a ridge-forming unit.

The termination of Sequence I is associated with the regression of the sea and karstification of the exposed Knox. As Sequence II stratigraphy developed, the major provenance for terrigenous clastic sediment shifted from the west to a more easterly region which was being tectonically uplifted. Sequence II deposition began in Middle Ordovician and is comprised of two sub-sequences. The lower sub-sequence, forming about one half of Sequence II, consists of sediments influenced by the development of a foredeep basin along the southeastern margin of the Appalachian mobile belt. Within the basin, graptolitic shale and turbidites, derived mainly from eastern sources, dominate. Patch reefs and skeletal sand banks developed west of the foredeep separating the deeper basin from a shallow water carbonate shelf that extended westward toward the Nashville Dome. Ultimately, during the Middle Ordovician, the foredeep filled and shallow peritidal environments prograded northwestward. The close of Sequence II is marked by the deposition of terrigenous sandstone in a shallow near-shore environment.

The base of Sequence III is marked by the Chattanooga Shale of Devonian-Mississippian age. The Chattanooga is typical of black shales that were deposited over much of North America during this time (e.g., New Albany Shale, Olentangy Shale, Ohio Shale, etc.). Transgressing seas later in the Carboniferous (Mississippian Period) gave rise to the deposition of limestone over much of this area as well as the continental interior; however, prograding deltas soon displaced the open marine conditions and terrigenous sand, silt and mud were deposited.

These transitional to continental environments closed out Sequence III and virtually any further marine deposition in the Appalachian Highland region. Other than Tertiary terrace deposits preserved on some ridge crests in the Valley and Ridge, there are no sedimentary rocks younger than the Carboniferous (Pennsylvanian Period) in the stratigraphy of this portion of the Appalachians.

BLUE RIDGE GEOLOGY

By
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Introduction

The purpose of this report is to provide a brief introduction to western Blue Ridge geology in eastern Tennessee and western North Carolina. Emphasis is placed on understanding Late Proterozoic to late Paleozoic tectonic evolution of the southern Appalachian orogen along the southeastern margin of Laurentia.

The western Blue Ridge is one of several geologic provinces in the southern Appalachians (Fig. 1) that are differentiated on the basis of varying tectonic frameworks. Rocks of the western Blue Ridge consist of 1.1 Ga crystalline basement (Odom and others, 1973) and younger (Late Proterozoic to middle Paleozoic) sedimentary cover sequences. The terrane is variably metamorphosed from anchizone along its western fringe up to kyanite-grade in the eastern regions. Most of its cover rocks are traditionally believed to represent syn- and postrift sedimentation along the Laurentian margin during continental breakup between 710 and 575 Ma (Rodgers, 1968; Odom and Fulligar, 1984; Bond and others, 1984).

In comparison, the Valley and Ridge province fault bounds the western Blue Ridge to the west and is composed of thrust faulted, but unmetamorphosed Paleozoic miogeoclinal sediments. The Valley and Ridge constitutes the youngest foreland fold-thrust belt of the orogen. To the east, the western and eastern Blue Ridge provinces are separated by the Allatoona-Hayesville-Gosson Lead faults. The eastern Blue Ridge consists of high metamorphic grade (up to hyperstene) Middle Proterozoic basement and younger Late Proterozoic to early Paleozoic off-shelf cover sediments and Paleozoic igneous intrusives (Hadley, 1970; Rankin, 1975; Hatcher, 1978; Merschat and Wiener, 1988; Hatcher and others, 1990). Farther east beyond the Brevard fault zone, Proterozoic to Paleozoic variable-grade metasedimentary, metavolcanic, and metaplutonic rocks of the Inner Piedmont and the exotic Carolina (Avalon) terrane comprise the remaining provinces of the mountain chain (Hatcher and others, 1990). Thus, the western Blue Ridge represents the transition between the unmetamorphosed exterior and the internal metamorphic core of the southern Appalachian orogen.

Western Blue Ridge Stratigraphy

Western Blue Ridge metasedimentary rocks at this latitiude are divided into two major divisions: A Lower Cambrian sequence consisting of the Chilhowee Group, Shady Dolomite, and Rome Formation, and the Late Proterozoic and younger(?) Ocoee Supergroup. The Ocoee is the dominant lithostratigraphic unit, and consists of a thick (>12 km) succession of fine to coarse clastic and subordinate carbonate rocks sandwiched between crystalline basement rocks and the Lower Cambrian sequence. Rocks of the Chilhowee Group (comprising an alternating sequence of quartzite and shale), Shady Dolomite, and the Rome Formation (fine-grained clastics and subordinate carbonate) underlie the mountainous region along the frontal edge of the province. Southeast of the frontal mountains, the foothills and higher peaks of the Great Smoky Mountains are underlain by Ocoee rocks and crystalline basement.

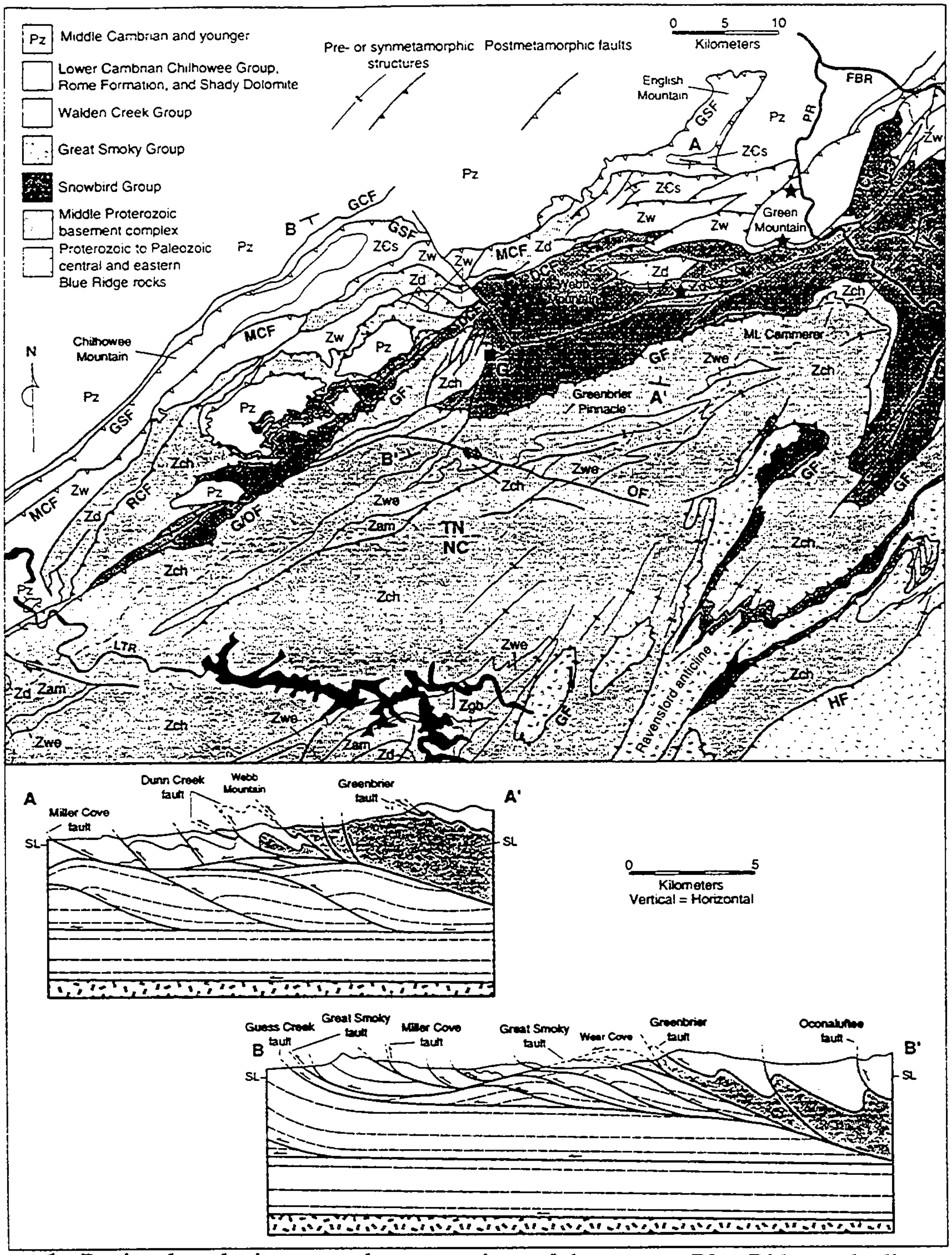


Figure 1. Regional geologic map and cross-sections of the western Blue Ridge and adjacent Valley and Ridge in the vicinity of the Great Smoky Mountains of Tennessee and western North Carolina showing distribution of rock units between the Great Smoky and Hayesville faults. In cross-sections A—A' and B—B', contacts between Paleozoic strata in the Great Smoky footwall are shown by dashed lines. Stars mark aproximate locations of field trip stops. Abbreviations of stratigraphic units: ZCs—Sandsuck Formation; Zam—Ammons Formation; Zch—Copperhill Formation; Zd—Dean Formation; Zgb—Grassy Branch Formation; Zw—Wilhite Formation; Zwe—Wehutty Formation. Abbreviations of faults and geographic localities same as Figure 1 except: GCF—Guess Creek fault; GSF—Great Smoky fault; G/OF—Greenbrier/Oconaluftee fault. Map modified from a number of sources including Hadley and Neslon (1971).

Geologists have long studied the Ocoee because of its anomalous thickness, paucity of fossils, absence of volcanic rocks, and position on the Neoproterozoic continental margin. Although Ocoee rocks have been recognized for more than a century (Safford, 1856), the internal stratigraphic framework of the Supergroup (Table 1) was resolved less than four decades ago by King and others (1958) during their geological survey of the Great Smoky Mountains National Park and vicinity (Fig. 2). Despite the stratigraphic and structural complexity of the terrane in this region, they recognized that two major faults, the Greenbrier and Dunn Creek faults, dissect the folded Ocoee sequence there and that each thrust sheet contains similar rocks. Thus, they subdivided the Supergroup into three groups: The Snowbird, Great Smoky, and Walden Creek Groups.

In the Park, the Snowbird Group mostly occurs in the footwall of the Greenbrier fault and is demonstrably the oldest of the three divisions because it rests nonconformably on crystalline basement (Hadley and Goldsmith, 1963). The stratigraphic position of the Great Smoky Group is less clear because it is bounded by the Greenbrier fault. Great Smoky rocks in the hanging wall of the Greenbrier fault structurally overlie Snowbird strata in the central Great Smoky Mountains (King, 1964), but to the east, Hadley and Goldsmith (1963) mapped Snowbird rocks in stratigraphic continuity beneath Great Smoky strata in the Ravensford anticline. Walden Creek Group rocks crop out in the footwall of the Dunn Creek fault, structurally beneath the Snowbird and Great Smoky strata. The Walden Creek Group is the youngest of the three subdivisions because it lies stratigraphically beneath the Lower Cambrian Chilhowee Group along the westernmost edge of the Blue Ridge (Hamilton, 1961; Neuman and Nelson, 1965; Keller, 1980; Walker and Driese, 1991; Carter, 1994; Carter and others, 1995a; Carter and others, 1995b). King and others' (1958) lithostratigraphic framework is presently applied throughout the province with few modifications (Carter and others, 1995c). Together with sedimentologic characteristics of Ocoee units (Table 1), these stratigraphic relationships first led King and others (1958) to conclude that the Ocoee represents synrift deposition along the Neoproterozoic continental margin.

The inferred Neoproterozoic age of the entire Ocoee Supergroup has been challenged recently with uncorroborated reports of a mid-Paleozoic microfaunal suite from the Walden Creek Group (Unrug and Unrug, 1990; Unrug and others, 1991, 1994), despite a plethera of conflicting stratigraphic evidence. Of course, traditional interpretions of the early tectonic origins of the Ocoee Supergroup (and the later deformational evolution of the terrane outlined below) are equivocal, and supporting stratigraphic data must be concidered spurious, if the age of the these rocks is ultimately resolved to be much younger.

Western Blue Ridge Structural Framework and Orogenesis

Assuming the antiquity of the Ocoee Supergroup, at least two post depositional deformation events are recognized throughout the western Blue Ridge (Rankin, 1975; Hatcher, 1978, 1987). Both circumstantial and direct evidence suggest that polyphase deformation occurred here during the Ordovician Taconian and Permian Alleghanian orogenies. Two diachronous Middle Ordovician clastic wedges (Blountian and Martinsburg) in the foreland of Tennessee (Shanmugam and Lash, 1982), and radiometric age dates for regional western Blue Ridge metamorphism (Rb-Sr mineral and whole-rock, $^{40}Ar/^{39}Ar$, and conventional K-Ar) (Butler, 1973; Dallmeyer, 1975; Kish, 1991; Connelly and Dallmeyer, 1993) suggest Taconian (480-440 Ma—Blountian phase) orogenesis, but rocks in the terrane also record radiometric evidence for a younger metamorphic event around 340 Ma (Acadian) (Aldrich and others, 1958; Long and others, 1959; Dallmeyer, 1975, 1988; Kish, 1991). Many western Blue Ridge structures are related to this peak Barrovian metamorphic event, including the Greenbrier and Dunn Creek faults in the Great Smoky Mountains National Park (Fig. 2), and regional folds with associated axial-planar slaty cleavage in low-grade, fine-grained rocks. The Greenbrier and Dunn Creek faults in the Park are interpreted to be premetamorphic because regional isograds and cleavage trends transect these structures. The terrane preserves many unique structural features of this early event because of its transitional position between the internal core and foreland of the orogen. Connelly and Woodward (1992), for example,

AGE	NORTH OF AND BELOW GREENBRIER FAULT				
Lower Cambrian	Chilhowee Group—Shady Dolomite—Rome Formation				
	Disconformity(?), but probably Conformable				
Late PreCambrian	Ocoee Supergroup	Walden Creek Group	Sandsuck Formation; Wilhite Formation (Yellow Breeches Member, Dixon Mountain Member)		
Late PreCambrian	Ocoee Supergroup	Great Smoky Group	Shields Formation, Licklog Formation (Formerly Walden Creek Group)		
			Dunn Creek—Rabbit Creek Fault Contact—sequence uncertain		
			Western part of area	Eastern part of area	
Late PreCambrian	Ocoee Supergroup	Great Smoky Group	Cades Sandstone	Rocks of Webb Mountain and Big Ridge	Rich Butt Sandstone
			(Formerly unclassified formations by King and others, 1958)		
Late PreCambrian	Ocoee Supergroup	Snowbird Group	Metcalf Phyllite	Pigeon Siltstone; Roaring Fork Sandstone; Longarm Quartzite; Wading Branch Formation	
			Base not exposed	Nonconformity	
Middle Proterozoic				Granitic and gneissic rocks	

SOUTH OF AND ABOVE GREENBRIER FAULT			
Late PreCambrian to Paleozoic		Western part of area	Eastern part of area
Late PreCambrian to Paleozoic		Walden Creek Group (Late PreCambrian)	Rocks of Murphy Marble Belt (PreCambrian to Paleozoic)
		Disconformity(?), but probably Conformable	
Ocoee Supergroup	Great Smoky Group	Dean Formation; Unnamed higher strata — Hothouse/Hughes Gap Formations; Anakeesta Formation — Wehutty Formation; Thunderhead/Elkmont Sandstones — Copperhill Formation	
Ocoee Supergroup	Snowbird Group	Roaring Fork Sandstone; Longarm Quartzite; Wading Branch Formation	
		Nonconformity	
Granitic and gneissic rocks			

Table 1. Stratigraphic nomenclature of the Ocoee Supergroup. Modified from King and others (1958) and Carter and others (1995c).

interpreted the Greenbrier-Dunn Creek fault system, and early premetamorphic folds associated with these faults to comprise the Taconian foreland fold-thrust belt in this part of the southern Appalachians.

The later (Alleghenian) event is better constrained by the Carboniferous-Permian clastic wedge that crops out in the footwall of the postmetamorphic Great Smoky thrust sheet in the Valley and Ridge province and farther west on the Cumberland Plateau (Secor and others, 1986; Hatcher and others, 1989a, 1989b). Alleghanian faults such as the Great Smoky and the Miller Cove faults, and associated folds, are brittle rather than ductile structures. These younger faults comprise the westernmost edge of the composite Blue Ridge-Piedmont crystalline allochthon, which was displaced a minimum of 350 km westward (Hatcher, 1989), producing the foreland fold-thrust belt in front of and beneath as the composite thrust sheet was emplaced. These frontal Blue Ridge faults in Tennessee, including the Great Smoky fault, contain the lowest detachments and easternmost thin-skinned (bedding) thrusts of the foreland fold-thrust belt (Hatcher and others, 1989a, 1989b; Costello, 1993; Carter, 1994; Hatcher, 1994). Conversly, the Miller Cove fault is the westernmost "basement" fault of the composite thrust sheet because it emplaces chlorite and higher-grade cleaved rocks at the surface (Hatcher and others, 1989a, 1989b; Costello, 1993; Carter, 1994). Radiometric-age dates in the more internal parts of the orogen, and possibly a few dates of uplifting sampled from the western Blue Ridge (Kish, 1991), provide additional confirmation of the occurrence of this event in the range of 325-240 Ma.

In the western Blue Ridge of central-eastern Tennessee, subthrust duplexing of foreland rocks produced the antiforms within the composite thrust sheet that were later breeched to create the windows in the Coves region of the Great Smoky Mountains (Fig. 2). From the vicinity of the Pigeon River northeastward, however, the Blue Ridge hanging wall becomes intricately duplexed (e.g., Diegel, 1986; Roberts, 1987). Several of these antiformal stack duplexes, including the Hot Springs window (Oriel, 1950, 1951) in central-western North Carolina and the Limestone Cove window (Diegel, 1986) farther northeast in Tennessee were breeched to expose lowestmost imbricates within the larger structure. In the region from Hot Springs northeastward, basement rocks are intimately involved in most levels of thrusting (Fig. 3), indicating close proximity to the presumed paleogeographic edge of the Ocoee depositional basin.

Although the reported Walden Creek Group microfauna (Unrug and Unrug, 1990; Unrug and others, 1991, 1994) has yet to be reproduced independently, tectonic models spawned from this and other recent western Blue Ridge paleontologic discoveries (Gastaldo and others, 1993; Tull and others, 1993) correlate Ocoee rocks with demonstrably younger western Blue Ridge sequences in the Murphy syncline and Talladega belt (Fig. 1). These models suggest that at least the reportedly fossiliferous part of the Ocoee sequence was deposited above a postulated regional Middle Ordovician unconformity into western Blue Ridge Taconic successor basins which were later intensely deformed during the Alleghanian, but possibly also earlier during the Devonian-Mississippian Acadian orogeny.

Concluding Remarks

In such a tectonically complex terrane as the western Blue Ridge, one needs at least a passing knowledge of the regional geology history to fully comprehend the significance of more local points of geologic and geomorphic interest. The following is a tabulation of the significant conclusions that comprise this report:

•The western Blue Ridge is one of several geologic provinces in the southern Appalachian orogen which are differentiated on the basis of constrasting stratigraphic, structural, and metamorphic histories.

•The Late Proterozoic Ocoee Supergroup is the dominant lithostratigraphic subdivision in the western Blue Ridge. It is divisible into three groups that represent different aspects of synrift deposition along the Neoproterozoic southeastern margin of Laurentia.

•Western Blue Ridge structures include pre- and postmetamorphic faults and folds which are the product of at least two major orogenic events to deform the Laurentian margin. Traditional interpretation suggests Barrovian metamorphism and related structures formed during the Ordovician Taconian orogeny while younger structures such as the frontal thrusts of the composite Blue Ridge-Piedmont crystalline allochthon formed during the Permian Alleghanian orogeny.

•Continued geologic exploration of the terrane will shed new light on conflicting paleontologic and stratigraphic evidence which suggests the need for major revision of our present understanding of western Blue Ridge (and southern Appalachian) Late Proterozoic to late Paleozoic tectonic evolution.

ROAD LOG

Contributions
By
Don Byerly, University of Tennessee
Tommy Douglas, North Carolina DOT
Russell Glass, North Carolina DOT
Harry Moore, Tennessee DOT

0.0 Leave Holiday Inn Worlds Fair Parking Lot.

0.2 Enter Henley Street Tunnel - Turn right onto Ramp to I-40 East

2.2 Enter urbanized karst area ; for next two and a half miles I-40 traverses karst topography underlain by Ordovician limestone; I-40, the Knoxville Zoo, Chilhowee Park, residential areas and Rutledge Pike and I-640 interchanges are all located in this two mile long karst belt..

4.8 Interchange with I-640.

6.8 Cross Holston River.

For the next 27.1 miles I-40 will traverse Valley and Ridge topography underlain by Cambrian and Ordovician limestone and shale; the route of I-40 typically follows the strike of the bedrock trending into long strike valleys and crossing broad carbonate ridges mantled with cherty residuum.

33.9 Interchange with I-81; continue east on I-40; the route now traverses across the regional strike of the bedrock. English Mountain (Blue Ridge foothills) is now visible in the horizon.

36.9 Exit 424: White Pine/S.R. 113 Interchange. Notice that strata of the Knox Group outcrops near the bridge. Note the chert in the residuum.

37.4 Cross over Douglas Lake (French Broad River). Cambrian and Ordovician age strata (Knox Group) are exposed in the bluff on the left side of the bridge along the lake. The medium-bedded strata are limestone and dolostone.

38.7 Rest area on westbound lanes.

38.9 Along both sides of I-40 you can see strata of the Ordovician age Sevier Shale exposed in the cutslopes: light brown to grayish-brown, thin soil and chippy shale.

The Sevier Shale is graptolitic. It is over 1,000 metersthick and represents Middle Ordovician (Sequence II) deposition in a foredeep basin along the eastern edge of the Appalachian mobile belt. The rounded hills, "haystacks" are typical landforms developed on shale in this climate.

42.8 Leave Jefferson County; enter Cocke County.

42.9 This area is underlain by strata of the Sevier Shale.

44.7 Exit 432 B: Leave I-40 and follow exit ramp to U.S. 70 and U.S. 25-E to Newport.

47.3 Junction with U.S. 25-E.

48.5 Junction with TN S.R. 32 and U.S. 321; continue on U.S. 321 north, U.S. 70 east and U.S. 25 south.

48.8 U.S. 321 junction; continue on U.S. 25-70.

48.9 Old Millstone Inn, constructed of Lower Cambrian Chilhowee Group sandstone.

49.2 Hunt/Wolf/VanCamp plant; rock bluff across river is Knox Dolostone - Cambrian-Ordovician Age.

49.7 Cross Pigeon River.

49.8 Junction with S.R. 73.

50.0 Railroad Crossing.

51.2 Brenda's on right!

51.7 Railroad underpass; view of Blue Ridge in distance.

52.4 View of French Broad River on left; carbonate bluffs of Knox Group.

53.5 Cross French Broad River.

55.3 Lower Cambrian Rome Formation.

55.4 Lower Cambrian Chilhowee Group.

56.2 Colluvium exposed along slope on the left of road.

Block-glide landslide occurred in fall of 1995; this dip slope problem was corrected by removing the sliding mass and the construction of a rock fall catchment fence(Brugg Cable, Inc.); work was performed by TDOT Maintenance forces.

57.5 Junction with S.R. 340; continue on U.S. 25-70 which crosses the mouth of Long Creek; note the flood plain deposits.

57.9 Slab Café across rosd from log slabbing operation.

59.1 Leave the flood plain of French Broad River.

59.7 Rome Formation.

60.0 SR 107 to the right leads to Del Rio, Tennessee.

60.5 Enter area of French Broad River flood plain.

Several old barite and iron mines and prospects are located in the vicinity of Del Rio.

61.0 Cross Laurel Creek.

62.6 The route climbs through a shale ridge; Sequence of Cambrian Age Rome, Shady, and Unicoi formations.

63.0 Note cracks in road; fill slide.

63.3 S.R. 107 turns left; continue straight on U.S 25-70.

64.3 Weavers Bend Road on left.

64.5 Cross French Broad River.

65.5 Exposures of Lower Cambrian Unicoi Formation.

67.3 Exposures of shale in the Unicoi Formation.

67.7 Tennessee / North Carolina state line.

A gabion wall located on the right just after crossing into North Carolina was constructed to retain a colluvial deposit.

68.2 The high cut (+/- 140 feet) on left exposes siliciclastic rocks of the Precambrian Snowbird Group.

The median barrier along the shoulder is to intercept small rock fall. The dwelling on the right replaced an older house damaged along with a vehicle, and a satellite dish by a shot gone awry during construction.

70.2 Enter Hot Spring window; cross Mine Ridge fault.

This area contains many old barite, manganese, and sulfide mineral (zinc/lead/iron) mines and prospects.

71.8 Approximate contact between the Cambrian-age Rome and Shady formations.

These units form the decollement of the Southern Appalachian thrusting.

72.5 Turn left and proceed 100 yards along a short side road (a section of older highway U.S. 25/70) to STOP No. 1.

This is an overview of the Hot Springs Window. The town of Hot Springs lies below on the floodplain of the French Broad River (a major headwater tributary of the Tennessee River). See Figure 2.

72.6 Turn left onto main highway, U.S. 25-70.

73.5 Turn left at junction with NC 209; enter central business district of Hot Springs, North Carolina.

73.6 Cross Spring Creek.

73.7 Cross Railroad tracks.

73.75 Turn left into Hot Springs Spa; follow road to parking area.

73.8 Exposure of Cambrian Age Shady Formation, a dolostone.

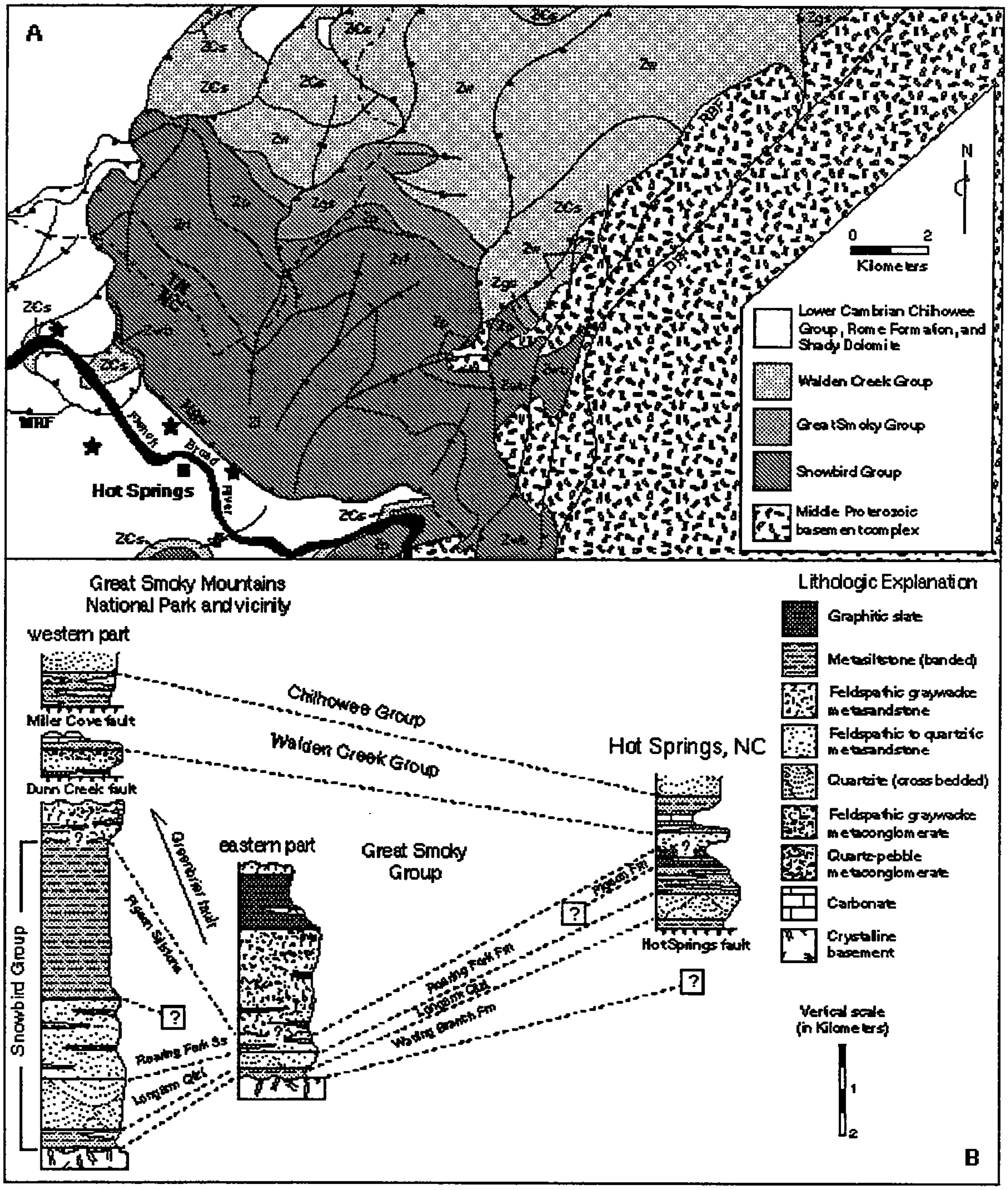


Figure 2: **(A)** Geologic map of the Hot Springs area. Statigraphic abbreviations: Zgs—Great Smoky Group undivided; Zl—Longarm Quartzite; Zp—Pigeon Formation; Zrf—Roaring Fork Formation; Zwb—Wading Branch Formation. Fault abbreviations: DFF—Devils Fork fault; HSF—Hot Springs fault; MRF—Mine Ridge fault; RBF—Rector Branch fault. Map compiled from Carter (unpublished field data), Oriel (1950), Bearce (1966), Burton (1977), and Bolton (1985). **(B)** Lithostratigraphic correlation of western Blue Ridge rocks from the Great Smoky Mountains National Park, Tennessee and North Carolina, to the Hot Springs, North Carolina area.

74.0 Parking area for Hot Springs Spa. STOP No. 2.

The springs were discovered in 1778. Various owners have operated taverns, hotels and spas at the site which was originally called Warm Springs. Usually facilities were destroyed by fire and new ones built to replace them. A railroad was completed through Warm Springs in 1882 and the town became incorporated as Hot Springs in 1886. German prisoners of war were housed in one of the early hotels in 1920 and later in 1933 the C.C.C. was quartered on the site. Currently the springs and campground is owned and operated by Eugene and Anne Hicks.

Analysis of the hot springs water by Chandler and Pellow and recalculated to ions by Dunnington (Watson, 1924):

Ion	Percentage
Cl	2.21
SO_4	48.51
CO_2	15.99
Na	1.00
K	2.45
Ca	18.94
Mg	3.36
Fe_2O_3	0.11
Al_2O_3	0.09
SiO_2	7.34
	100.00

Salinity: 735 ppm

Despite the fact that there has been a number of geologic investigations in this area over the years, the origin of the springs remains a problem. A leading hypothesis on the origin of the is shown in Figure 3 (S.S. Oriel, 1950). Water enters the subsurface along discontinuities such as the Dry Pond Ridge fault and associated fractures, migrates downward (perhaps, 4,000 to 5,000 feet) to be heated by geothermal gradient, then, ascends under hydrostatic pressure along solution channels, bedding planes or other discontinuities in the vertical beds of the folded Shady dolomite.

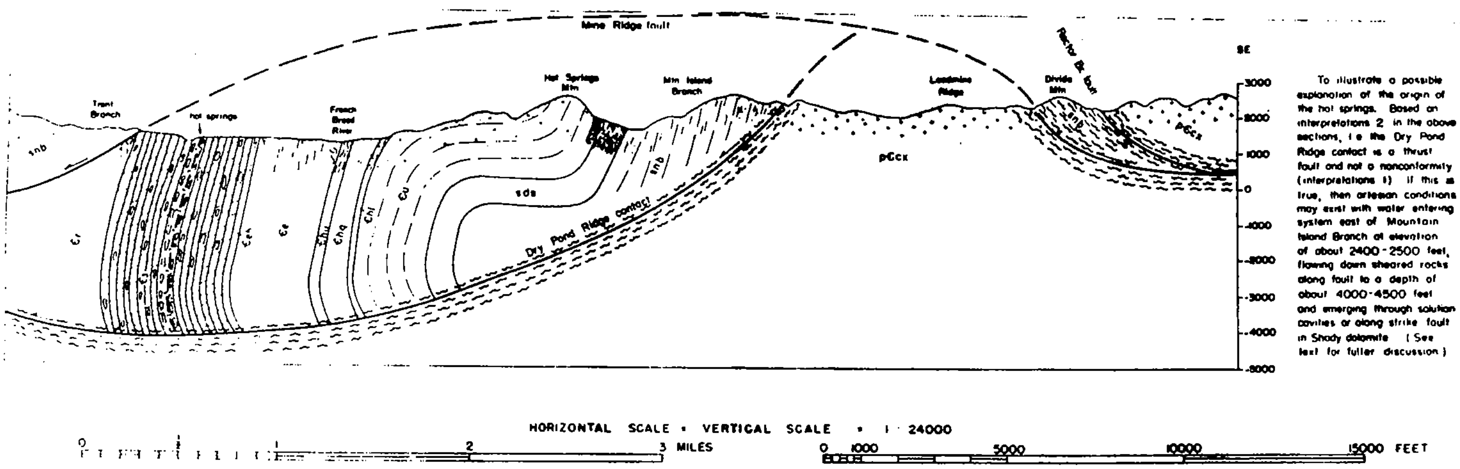


Figure 3. Cross-section illustrating possible origin of the hot springs.

74.3 Turn left onto main highway, U.S. 25-70.

74.4 Cross French Broad River.

74.7 Cross contact between the Snowbird Group and the Shady Formation (fault contact); leave Hot Springs window.

75.0 Cut and fill slope failures.

On the right sagging guard rail indicated a failure in a thin sliver-fill. A pile panel wall is proposed to correct the problem. On the left, a cut slope failure has occurred along discontinuities in "poor" rock material. Failure during construction proved what was feared during design. A wide fall out zone was created to hold material until funds become available for remediation.

77.1 Tanyard Gap. Appalachian Trail crosses above highway.

This trail extends from Cohutta Mountain in Georgia to Mount Katahdin, Maine.

79.2 Millwheel Cafe. Turn right at Yield Sign, on U.S. 25-70; (NC 208 is to the left).

81.4 Top of hill.

84.4 Road to Walnut, North Carolina on right.

85.2 Entrance to NOC Rafting.

86.4 The rounded hilly topography typifies the weathering of the crystalline granitoid rocks in a warm humid climate. Crystalline Precambrian Basement forms the bedrock in this area.

88.4 Intersection with Business 25-70 to Marshall, NC; continue straight on by-pass.

89.1 Traffic light; road to Marshall on right.

90.5 Turn right onto NC 213 ramp to Mars Hill, NC.

90.7 Turn left on NC 213 to Mars Hill.

93.0 View to right of Unaka mountains.

94.2 Enter Petersburg / leave Petersburg.

96.3 View of Unaka Mountains.

97.3 City limits of Mars Hill, NC.

97.7 Mars Hill College, a liberal arts college founded in 1856.

98.2 Traffic light; proceed straight ahead.

99.2 Junction U.S. 19/23; cross over U.S. 19/23 turn right onto ramp for U.S. 19/23 north toward Johnson City Tennessee.

100.4 Begin NCDOT Project A-10BB..

This project is an upgrade and partial relocation of existing Highway U.S. 19/23 to interstate standards. Construction began in July 1996 and completion is expected in the Winter of 1998.

The A-10 Corridor, when completed, will be the primary route between northeast Tennessee and southwest North Carolina. It will be designated as an extension of Interstate 26 which currently runs between Asheville, NC and Charleston, SC. On the three northernmost sections of the A-10 project, there will be about 9.5 miles of new alignment involving large cuts and fills over rugged mountain terrain.

Work is well underway on the A-10BB project (1.7 miles), while grading on the A-10C project (6.1 miles) was started in early 1997. Ther are 5 major cuts on the A-10C project ranging in size from 280 feet to 450 feet. Excavation on the A-10C project is estimated to be 23,385,000 cyds with an additional volume of undercut excavation estimated to be at least 2,347,000 cyds. Work will begin on the A-10D project (3 miles) in late 1997. On this project there are 6 major rock cuts ranging ibn size from about 200 feet to greater than 400 feet. The estimated volume of excavation on the A-10D project is 7 million cyds.

101.1 Veer left on U.S. 23 north; U.S. 19 goes to Burnsville, NC.

101.2 **End Project A-10BB and begin Project A-10C.**

Project A-10C contract was let in October 1996 and completion is anticipated by December 31, 2001. Bedrock in this area consists predominately of Precambrian biotite granite gneiss.

106.3 **Murray Gap.**

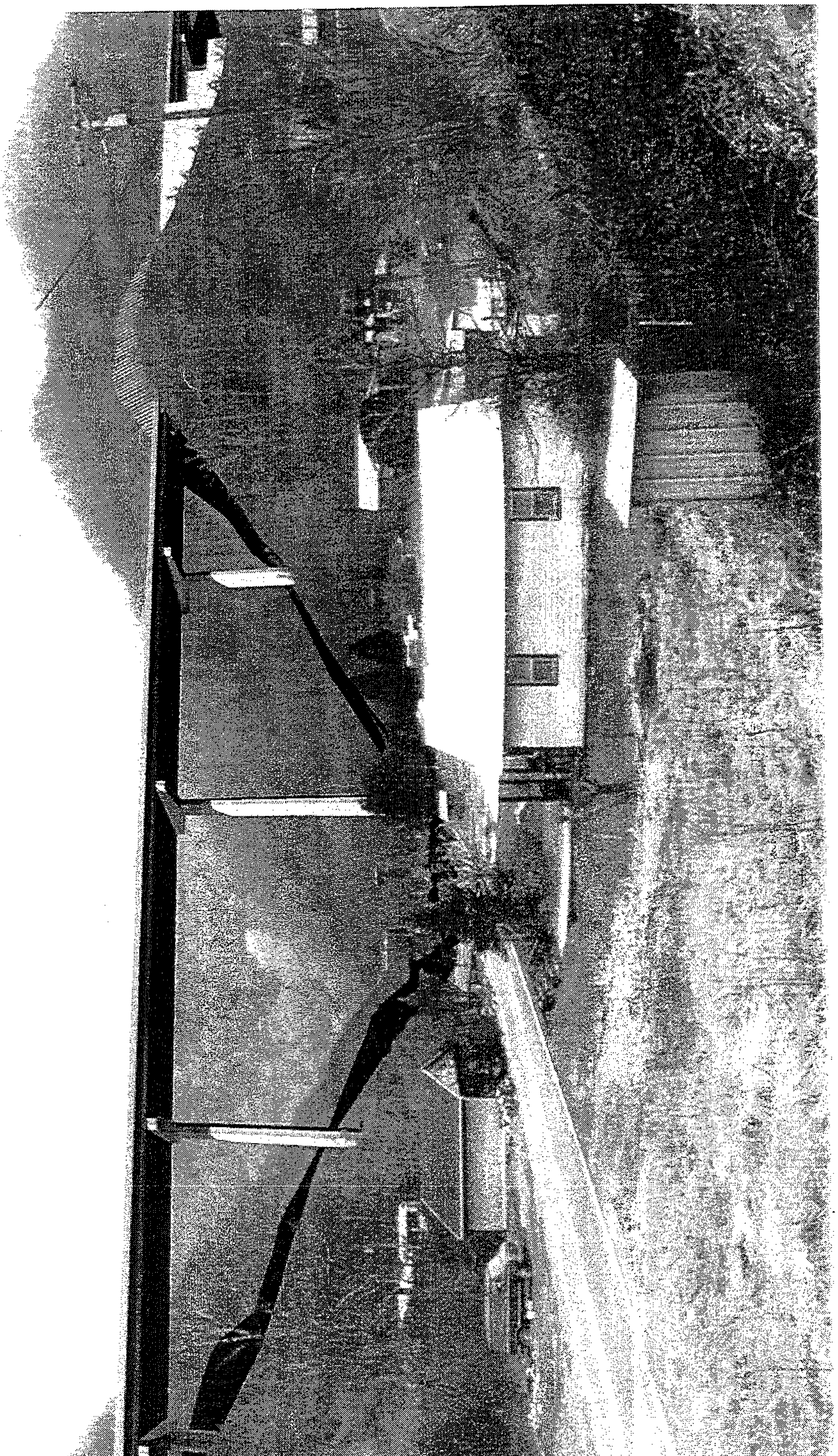
108.0 **Entrance to Wolf Laurel on right; roadway construction project waste area on left.**

108.5 **Location of future Bear Branch Interchange on I-26.**

108.9 **End construction section A-10C; Little Creek Café on right.**

109.0 **STOP No. 3. Laurel Creek Church parking area.**

This the future site of the Laurel Creek Bridge, spanning the gap in the ridge to the west. The bridge will be about 220 feet above the stream bed and nearly 1,000 feet long. At the north end of the bridge the alignment immediately plunges into a side-hill cut and fill. In the early stages of the project, a 1,200-foot viaduct was proposed for this area to avoid removing all of the colluvium in this slope. Later, a three-tiered wall was considered to keep the fill out of the creek. At this time, it appears that a buttress will be placed above the creek to hold up the fill and keep it out of the creek. Figure 4 on the next page is an artistic rendition of what the finished bridge will look like.



111.9 Tennessee/ North Carolina state line (Sams Gap) ; re-enter Tennessee.

This is the end of North Carolina Project A-10D and the beginning of the recently completed Tennessee section of interstate.

The Appalachian Trail previously crossed the highway at this point. It now crosses about 500 feet back into North Carolina.

113.7 Potential lunch stop if raining (beneath bridge).

116.0 Take Flag Pond exit.

116.4 Turn left onto entrance ramp to U.S. 23 south; proceed back up mountain to scenic overlook.

120.4 Exit to scenic overlook; LUNCH, STOP No. 4; short overview of Tennessee construction of future I-26.

120.8 Exit overlook and enter U.S. 23 south ;proceed to top of hill.

121.4 Turn right onto old U.S. 23 (go across gravel area - watch for traffic on old highway).

121.5 Turn left back onto U.S. 23-north (re-enter Tennessee)

121.9 Tie back retaining wall on right (appears as a conventional concrete retaining wall) is retaining very unstable colluvial material that extends upslope to near the top of the ridge. The Appalachian Trail is located at the top of the ridge.

122.6 Habitat (wildlife) Boxes; these boxes are intended to be used by wildlife for crossing the roadway without getting into the traffic lanes.

122.7 Truck escape ramp - The escape ramp is part of a large engineered embankment that contains 1.25 million cubic yards of fill material. A toe buttress, internal shear-key and extensive underbenching and rock pads were used on this embankment.

123.7 Rockfall catchment fence on right; STOP No. 5.

This stop affords the view of two rockfall fences. The upper fence was constructed on the bench to help reduce the effects of rockfall on the roadway and lower catchment area (notethe placement of No. 57 size stone - approximately one inch - on the bench behind the fence; this is to reduce the bouncing effects of falling rock) (designed by TDOT personnel). The lower fence is a Brugg Cable, Inc. fence designed to provide resistance to any falling rocks from above. It has an impact load strength of 75,000 ft. lbs. Rockfall problems were anticipated for this and several other cuts on the 15 mile project. Numerous field tests were conducted by rolling rocks off the finished cut slopes and studying their behavior. These results were compared to rockfall computer simulations to base decisions on where rockfall would be most likely to occur and mitigation would be required. As a result, over 7,000 linear feet of Brugg Cable fence and 1,200 linear feet of TDOT fence was installed at the required sites.

125.4 Thrust fault and catchment fence; STOP No. 6.

This is the largest cut on the project, with 2.5 million cubic yards of rock removal. The country rock is interlayered banded gneiss of intermediate composition and layers and small stocks of basalt. Gabbroic and diabasic textures are developed within the thicker basalts. The contacts between the gneisses and basalts are generally cataclastic with thin gouge zones and pyrite porphyroblasts within the gneiss. A large boudinaged white quartz vein occurs on the west face of the cut near the top. Towards the south end of the cut is a low angle thrust with a sulfide zone in the hanging wall.

The upper sections containing the weathered rock, saprolite, and soil were excavated on 1.5:1 ratios and were serrated. The in-place rock was excavated on a 0.5:1 ratio with a catchment bench located just below the in-place rock -weathered rock zone. The lower 100 feet of the cut slope contains no benches except for a 40 foot wide catchment zone at roadway levcel. All in-place rock was pre-split. The total cut height (vertical) exceeds 300 feet. A zone of acid-producing pyritic rock was encountered near the south end of the cut section. The pyritic rock is located above the relatively low angle thrust sheet exposed in the cut slope. Iron staining on the exposed rock marks the location of the acid-producing rock material.

Approximately 75,000 cubic yards of acid producing rock have been encapsulated just north of this cut section and within the main roadway template fill section. The encapsulation process used 60 mil HDPE geomembrane which was placed in a constructed "trench" in the roadway fill section. After placement of the acid rock and treatment with agriculture lime, the geomembrane was placed over the rock, sealed and then covered with a minimum of 5 feet of embankment material.

Rock consisting of gneiss and basalt was processed to be used as base stone for this section of the project. Approximately 200,000 cubic yards of material was crushed and graded out for use.

125.7 Flag Pond Exit.

125.9 Pyrite encapsulation area in roadway fill. NOTE: leaving banded gneiss/ basalt zone, transition into Cranberry grantie gneiss.

127.5 Cranberry lithology in cut slope on left

128.6 Exit to scenic overlook. STOP No. 7; picture taking area (Kodak moment!).

128.9 Return to U.S. 23 north.

129.8 Clear Branch Road Interchange.

130.2 Unakite cut; <u>STOP No. 8</u>.

This is the heart of the unakite zone, a rock type that is named from this area of the Unaka Mountains and is defined as a granitic rock composed of pink feldspars, green epidote, black hornblende and quartz. The rock is a metasomatic product, in this case, of the Cranberry Granite. The original rock appearance is unknown, but it would typically be a coarse grained, porphyritic granite that may be white or pink. The most notable result of the metasomatism is the introduction of iron and the recrystallization of the K-spars into pink or red porphyroblasts. The plagioclase is saussuritized and the ferromagnesium minerals are chloritized.

The unakitization process begins as the introduction of pink orthoclase metacrysts in zones of mylonite and phyllonite and as small injection gneisses and pegmatites of red or pink color. The process results in the end as a complete unakite.

Hematite is abundant along joints, fractures and faults. It is especially noteworthy along latte stage vertical "streamer" zones of a few inches to several feet wide that cut all other features and introduce quartz and hematite.

A high angle thrust that repeats the unakite zone can be observed along the southern end of the road cut. The fault zone is weathered to a saprolite, but dark zones that represent a mafic intrusion are obvious.

The cut slope left and right of the roadway show entirely different engineering properties. As one looks to the north, the cut slope to the left is constructed on 1:1 ratios with a 20 feet wide bench at 40 foot vertical intervals. This was required due to the very low RQD (0-40) for this interval.

However, on the opposite side of the road the material is very massive with high RQD (80-100). These slopes were constructed on a 0.25:1 slope ratio and were pre-split. The upper portions that contain weathered material were constructed on a flatter 1.5:1 ratio. A 25 foot wide catchment ditch is located at roadway level.

This cut interval involved the removal of over one million cubic yards of soil and rock material. Most of the unakite material was processed into useable products for the project - the coarse material (2 inches to 3 feet) was used for rock drainage pads beneath the embankments, while the finer material was used for the base stone covering the roadbed subgrade.

Please feel free to sample the unakite!

131.2 Cross South Indian Creek.

131.9 **Basement Gneiss/Graphite Phyllonite-Mylonite cut section.**

Fine grained rocks formed by extreme deformation of originally coarse-grained rocks (phyllonite); contains evidence of differential movement on closely spaced slip surfaces; the result is a rock that closely resembles a phyllite but differing in origin and in some details of fabric).

This is a mylonized zone within multiple thrust sheets of Basement gneiss. A wedge shaped structural trap defined by the intersection of a near horizontal thrust and a steeply dipping thrust has trapped remobilized carbon that has subsequently precipitated sulfides. Both pyrite and marcasite are present.

The extreme cataclastic textures of this outcrop make classification difficult, but it should be noted that these rocks do no appear to be as well banked or intruded by basalt dikes like the gneisses that are exposed at Stops 4 and 5.

Also of note, are boudinaged quartz veins and large "rolled" blocks within the phyllonites.

Excavation of the strata in this cut interval required extensive blasting and "ripping". The cut slope was constructed on a straight 0.75:1 slope ratio with a 20 foot wide catchment ditch at roadway level. The rock was also pre-split, resulting in the "neat line" appearance of the exposure. The cut slope is 217 feet in height and 800 feet long.

Opposite the high slope is a cut slope excavated principally along a fault plane. Extensive slick-n-sides can be seen in that exposure.

Approximately 200,000 cubic yards of acid producing rock was excavated from the blackish-gray phyllite and phyllonite portion of this cut interval. A little over 15,000 cubic yards were considered acid producing enough to do environmental damage and were encapsulated on the project.

Prior to this cut section we crossed a major fault zone (small valley to left as we crossed the last bridge) where the structural trend changes 90 degrees;in the last cut interval, the rock structure and the pink lithologies are exposed. These are granite gneisses known as unakite (seen at the previous stop).

132.4 **Exit U.S. 23 at the Temple Hill Exit.**

132.7 **Turn right and proceed to stop sign; turn left onto old U.S. 23.**

133.0 **Turn right on county road and go past Temple Hill School.**

133.4 Giant ripple marks; STOP No. 9.

This is a picture stop for unusual preservation of Cambrian Age ripple marks found in strata of the Chilhowee Group. Note that this is private property!

133.6 Turn left and proceed to stop sign; turn left again onto old U.S. 23.

134.5 Turn right and bear right to re-enter U.S. 23 north; roadway embankment to left is site of pyritic rock encapsulation and treatment.

136.1 Cross South Indian Creek.

136.9 Rome / Shady cut section; STOP No. 10.

This cut exposes Paleozoic age strata that have been severly tectonicly deformed. This stratigraphic zone does not usually form natural outcrops due to its sheared and easily weathered nature, but it is the predominate decollement or thrust zone that dominates the structural style of the Valley and Ridge province. The underlying Shady Dolomite is intensely brecciated and recemented by calcite and to a much lesser extent, quartz veins. The overlying Rome Formation (multicolored shale and siltstone) is deformed by forces apparently directed from the south. Plastic deformation of bedding is obvious, but closer examination will reveal a system of white quartz filled fractures and ladder veins. The yellow or buff zones at the base of the Rome are thought to be mylonite. At the southern end of this cut section one may observe at least two examples of the limits of plastic deformation and the beginning of high angle thrusts. The engineering geology of this cut interval includes the use of 1.5:1 ratios for the weathered shale strata and 1:1 pre-split slopes for the in-place unweathered shale and dolostone. The in-slope catchment benches (20 feet wide) are located at 50 foot vertical intervals. The weathered portions of the cut slope have been serrated ("stair-stepped" benches) in order to reduce sheet washing effects. This permits the rapid establishment of vegetation. The dolostone material (Shady Dolomite) was crushed on site and used as basestone for the lower three mile section of the project.

138.4 Cross Nolichucky River.

The sand comprising the floodplain is very micaceous. The sediment source is tailings from pegmatite mines in the Spruce Pine, NC Mining District. Mica and feldspar are the principal products of the district, but some gems have also been produced.

139.8 On the right the Clinchfield Railroad yard in Erwin, Tennessee.

The low ridge to the right (east) is called Elephant Hill. The name is associated with the hanging of an elephant years ago when it was found guilty of trampling its triner to death during a circus parade in a nearby town. A wrecker crane used by the Clinchfield Railroad as selected to carry out the deathe sentence of, hanging 'til death."

140.7 Intersection with State Route 81.

150.3 Enter Carter County. The large mountain to the left (north) is Buffalo Mountain.

This mountain is underlain by Cambrian clastics of the Chilhowee Group (shale, sandstone, quartzite) which have been faulted and folded. The valley that the roadway follows is underlain by Cambrian and Ordovician Age carbonates and shales.

153.0 Enter Washington County.

154.2 U.S. 23 and S.R. 36 changes to I-181 from this point to Interstate 81.

This entire route of U.S. 23 from the North Carolina State Line to I-81 will eventually be assigned I-26 in the future. The metropolitan area the route is passing through is Johnson City, Tennessee, one city of the tri-cities area. For the next 15 miles the route of I-181 traverses across the strike of Cambrian and Ordovician limestone and shale strata and corresponding ridges and valleys. This gives gives the " hill and valley " effect of the roadway alignment as the route traverses the Valley and Ridge province.

168.1 Enter Sullivan County.

169.2 Exit I-181 north to I-81 south toward Knoxville.

For the next 57 miles the route of I-81 follows the general NE-SW strike of Cambrian and Ordovician limestone and shale; prominent ridges (Bays Mountains) can be seen to the right (north) from Fall Branch to near Baileyton (log miles 176 to 190). These ridges are underlain by resistant sandstone and calcareous siltstone units of the Middle Ordovician (uppermost units of Sequence II, filling the foredeep basin) which have been folded into a synform.

225.8 Leave I-81 south and enter I-40 west.

For the next 27.1 miles I-40 will continue to traverse Valley and Ridge topography underlain by Cambrian and Ordovician limestone and shale; the route of I-40 typically follows the strike of the bedrock, trending into long strike valleys and crossing large broad ridges underlain by cherty dolomite.

252.9 Cross Holston River

254.9 Interchange with I-640.

Enter urbanized karst area; for the next two and a half miles I-40 traverses karst topography underlain by Ordovician limestone; I-40, the Knoxville Zoo, Chilhowee park, residential areas, and the Rutledge Pike and I-640 interchanges are all located in this two mile long karst belt.

255.8 Rutledge Pike Interchange.

Many of the warehouses to the right (north) have been constructed on filled sinkholes. This has plugged natural drainage and caused costly flooding.

257.7 Cherry Street Interchange.

259.2 James White Parkway Interchange.

260.0 I-275 interchange Ramp.

260.6 Take Exit # 387 to 17th Street; cross 17th St. and proceed to Western Ave.

261.1 Turn right on Western Ave.

261.7 Turn right on Henley St. (U.S. 411 south).

261.9 Turn right from Henley St. into Holiday Inn parking lot.

End of Field Trip! Hope you had a good time.

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CONSTRUCTION TECHNIQUES ON U.S. 23 (FUTURE- I-26) UNICOI COUNTY, TENNESSEE

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ABSTRACT

The construction of US 23 (Future I-26) through the mountainous areas of Unicoi County, Tennessee required the use of special design for roadway embankments. The concept of underbenching was used for every major highway embankment along the 24 kilometer (15 mile) length of the subject project.

The highway construction also required special design regarding environmental issues. These issues included acid producing rock, serrated cut slopes, erosion control measures (temporary seeding, rip-rap ditches, siltation dams), bioengineering and habitat crossing boxes.

Structures constructed on the project included conventional concrete and steel bridges, mechanically stabilized earth walls, a tie back wall, and rockfall catchment fences.

Considerations regarding the engineering geology were monumental and included items such as rockslope stability, rockfall, embankment stability, groundwater drainage, acid drainage, and foundation conditions for bridges and retaining walls. This unique project was completed in late spring of 1995 (May- June) and officially opened to the public on July 5, 1995.

INTRODUCTION

As population centers continue to grow, the demand for new infrastructure correspondingly increases. In most instances the areas for proposed new infrastructure are located in "bad ground" includes steep terrain, landslide prone areas, soft soils, karst, and generally unstable rocky ground that has been "skipped over" by cities and developers for more desirable flat stable locations.

The site of U.S. 23 (Future I-26) is located in the rugged steep terrain of the Blue Ridge Physiographic Province of Tennessee (Figure 1 and 2). Beneath the heavily vegetated mountain slopes, the geology provides for interesting and challenging geotechnical considerations.

The project site is more specifically located in the southern half of Unicoi County south of Erwin, Tennessee and between the Nolichucky River and the Tennessee-North Carolina State Line. The existing U.S. 23 alignment is very curvy and dangerous as numerous tractor-trailer trucks use this highway (Figure 3).

The general geology of the subject site involves highly metamorphosed and tectonically deformed strata of Precambrian Age (over one billion years old) and sedimentary strata of early Paleozoic Age. At the base of the mountain near the community of Temple Hill, Precambrian strata of the Ocoee Supergroup (approximately 500 plus million years old) are thrust against Cambrian Age sedimentary rocks.

As one goes south from Temple Hill exposures of pinkish-green granite gneiss (Unakite - popular with rockhounds) are found. This granitic material has been dated at over one billion years in age.

Further south near the community of Flag Pond the rocks become mafic in composition and highly metamorphosed, characterized by layered and banded gneisses and schists. These medium to light gray rocks are intruded by numerous dark grayish-black basalt layers. These schists and gneisses extend across the Tennessee-North Carolina border into North Carolina.

Extreme folding and faulting has extensively fractured the bedrock, emplacing numerous discontinuities. Weathering and erosion of the exposed rock has produced a mantle of saprolite and colluvial material.

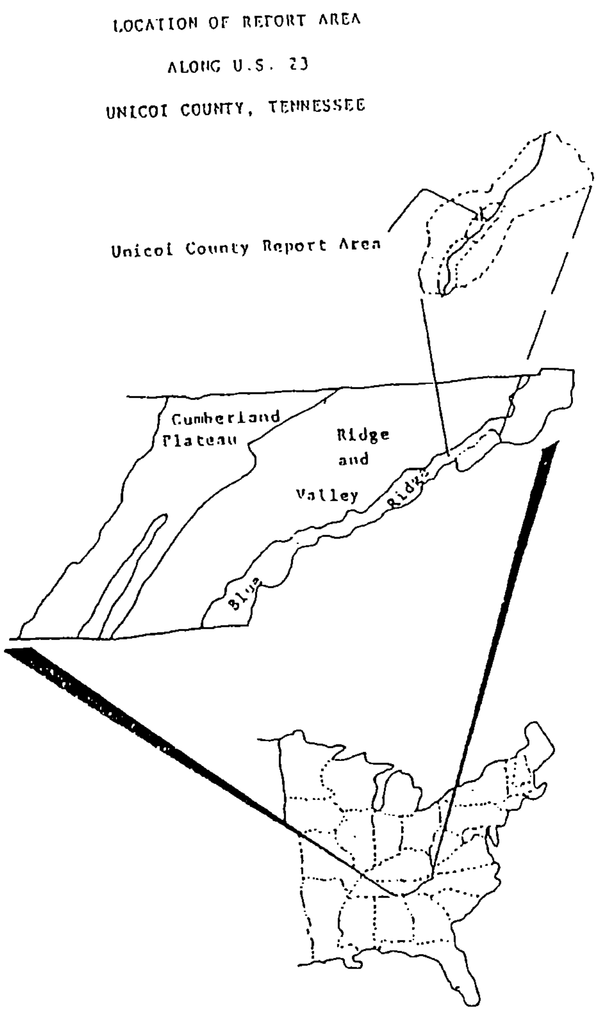

Figure 1. General Location Map of Project.

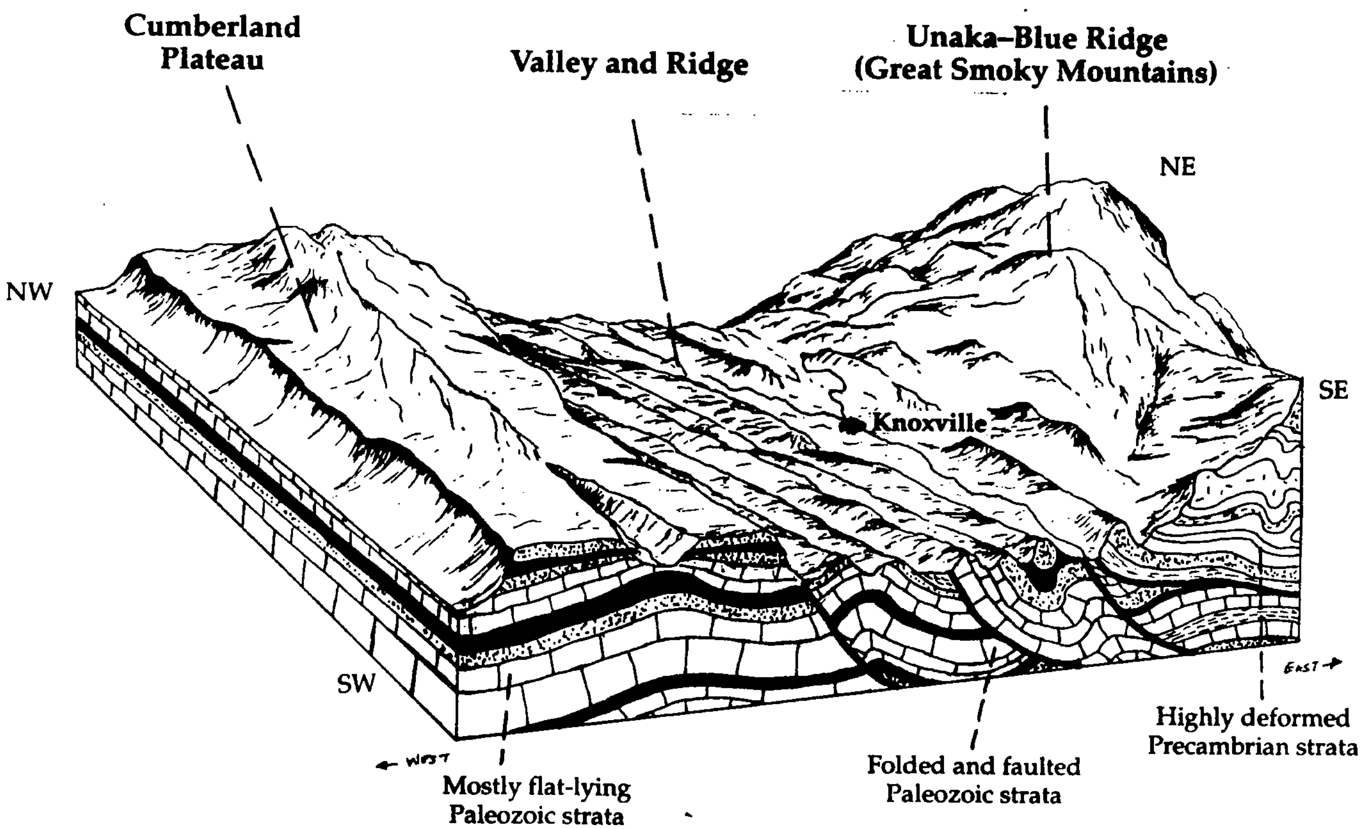

Figure 2. Physiographic Block Diagram of East Tennessee.

The bedrock discontinuities, saprolite, and colluvial material are some of the geologic features that influenced the design and construction of the subject project.

UNDERBENCHING CONCEPT

The concept of underbenching is usually employed in the construction of embankments on steep natural slopes (Figure 4). Highways and railroads are generally where the underbenching concept is most often employed. The purpose of the underbench is simply to prevent the development of a failure plane along the embankment-natural surface boundary in steep terrain (especially where natural slopes are 4:1 or steeper) (Figures 5 and 6).

The underbenching procedure involves the removal of unstable materials (colluvium, soft soils, etc.) by the excavation of benches into firm weathered rock and/or in-place bedrock. The benches are located in embankment (fill) areas and generally will follow the contour of the land.

The bench widths were required to be a minimum of 3.0 to 3.6 meters (10 to 12 feet) and bench heights were kept to a maximum of 3.6 meters (12 feet). In some instances bench widths were as much as 7.6 meters (25 feet), but generally averaged around 4.5 meters (15 feet).

The design of the underbenched embankment requires the placement of a graded rock pad on the underbench area before fill material is placed. The rock pad has a minimum thickness of 1.5 meters (five feet) and a gradation that meets the following specification (Figure 7).

SOLID ROCK MATERIAL SPECIFICATION FOR ROCK BUTTRESS, ROCK KEY AND ROCK PAD CONSTRUCTION

THE SOLID ROCK MATERIAL SHALL CONSIST OF SOUND, NON-DEGRADABLE LIMESTONE, SANDSTONE, GRANITE, GRANITE GNEISS OR GNEISS WITH A MAXIMUM SIZE OF 0.9 METERS (3 FEET). AT LEAST 50% (BY WEIGHT) OF THE ROCK SHALL BE UNIFORMLY DISTRIBUTED BETWEEN 0.3 METERS (1 FOOT) AND 0.9 METERS (3 FEET) IN DIAMETER, AND NO GREATER THAN 10% (BY WEIGHT) SHALL BE LESS THAN 5.0 CM (2 INCHES) IN DIAMETER. THE MATERIAL SHALL BE ROUGHLY EQUI-DIMENSIONAL; THIN, SLABBY MATERIAL WILL NOT BE ACCEPTED.

TO FACILITATE AND INSURE THE ACCOMPLISHMENT OF THIS GRADATION, THE CONTRACTOR SHALL BE REQUIRED TO PROCESS THE MATERIAL WITH AN ACCEPTABLE MECHANICAL MEANS (A SCREENING PROCESS CAPABLE OF PRODUCING THE REQUIRE GRADATION). THE ROCK SHALL BE APPROVED BY A REPRESENTATIVE OF THE GEOTECHNICAL OPERATIONS SECTION, DIVISION OF MATERIALS AND TESTS, BEFORE USE (TENNESSEE DEPARTMENT OF TRANSPORTATION, 1995).

The rock pad may be incorporated into a toe rock buttress or fill "shear key" depending on the design requirements. The rock pad serves as a drainage layer for groundwater and also for surface water infiltration. In addition, the rock pad serves as a stability element in the embankment structure.

APPLICATION TO U.S. 23 PROJECT

The concept of underbenching was applied to the entire 24 kilometer (15.2 mile) length of the subject project. A total of 41 embankments required the underbenching treatment. Several of the embankments were over 609 meters (2,000 feet) in length and approximately 61 meters (200 feet) in height.

The underbenching procedure began as clearing and grubbing and slope staking was completed. Excavation for the underbenching was initiated near the upper most fill line of each embankment section and proceeded down to the fill toe area (Figures 8 and 9). Vertical slopes and benches are excavated in this fashion until the unstable material is removed.

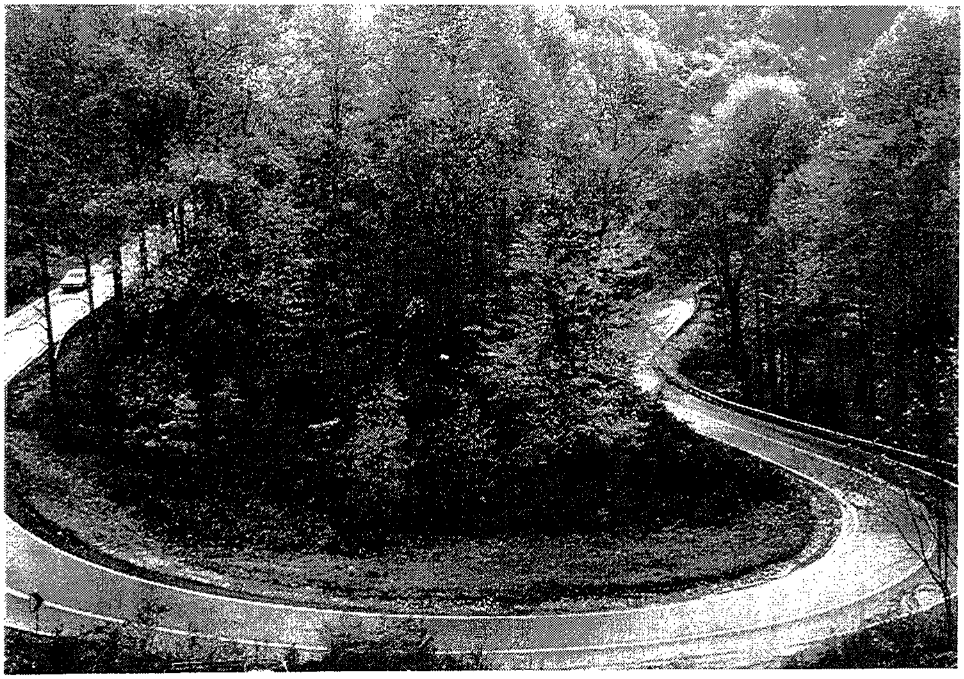

Figure 3. Existing U.S. 23 Near Sams Gap at the Tennessee/North Carolina State Line.

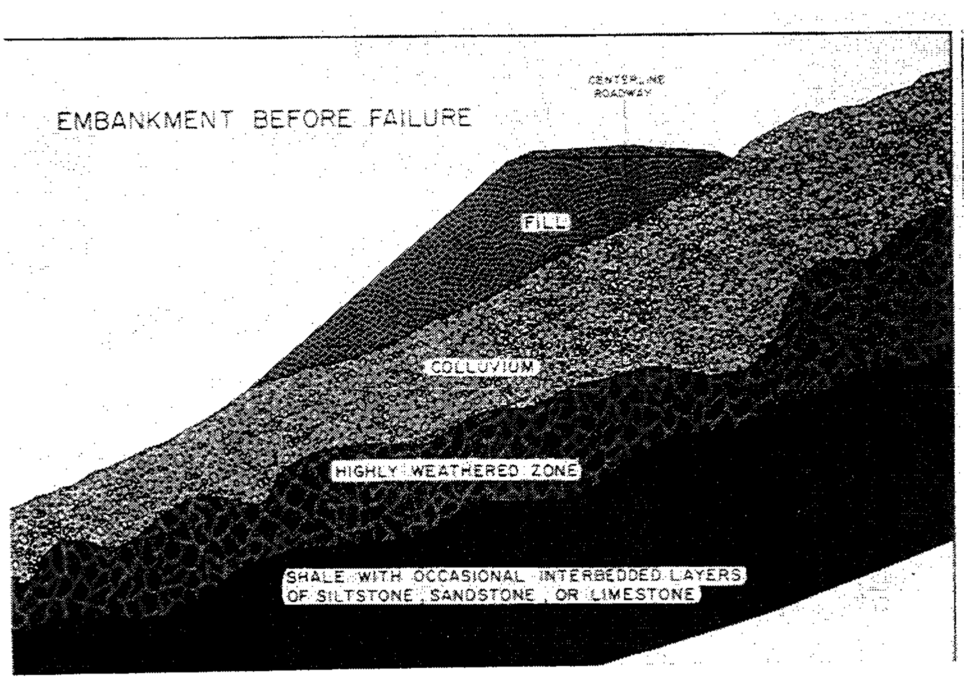

Figure 4. Schematic Drawing of Highway Embankment Constructed on Unstable Material (Colluvium, Highly Weathered Material).

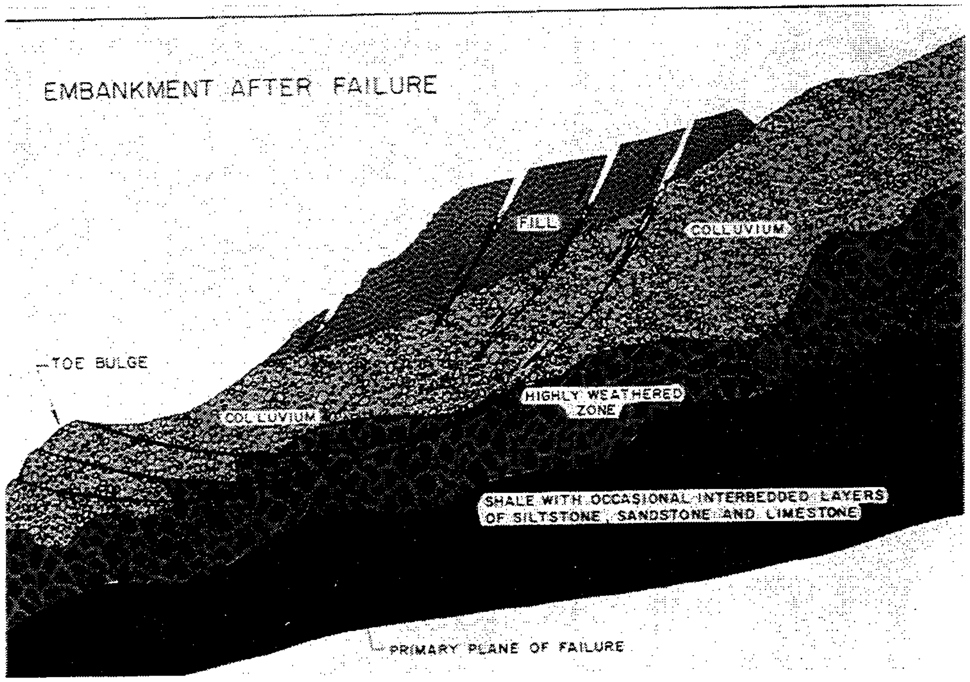

Figure 5. Schematic Drawing Showing Failure of Embankment Due to Unstable Foundation Material.

Figure 6. Actual Roadway Embankment Failure Due to Unstable Foundation Material (S.R. 61, Union County).

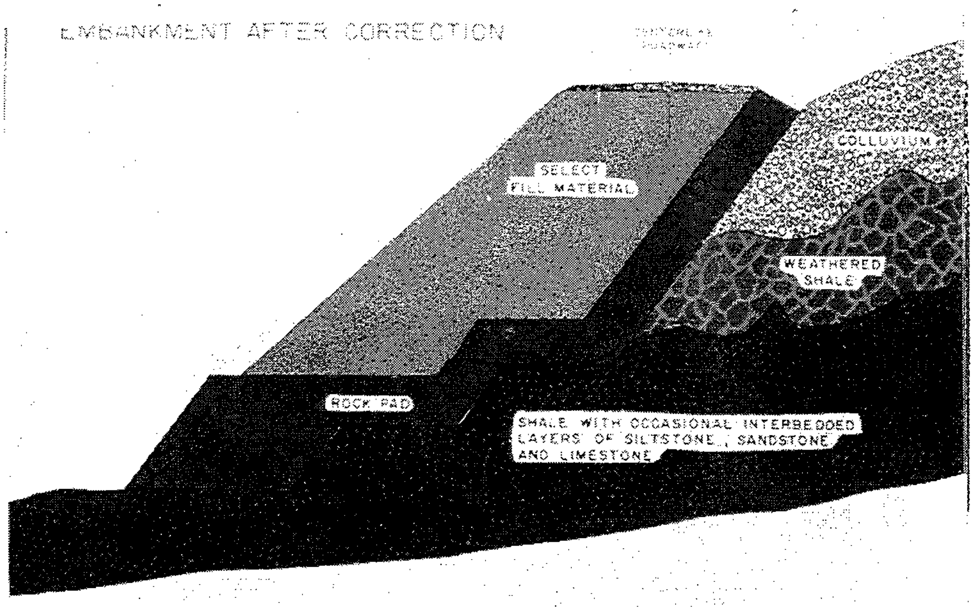

Figure 7. Schematic Drawing Illustrating Underbenching Concept with Rock Pad and Engineered Roadway Embankment.

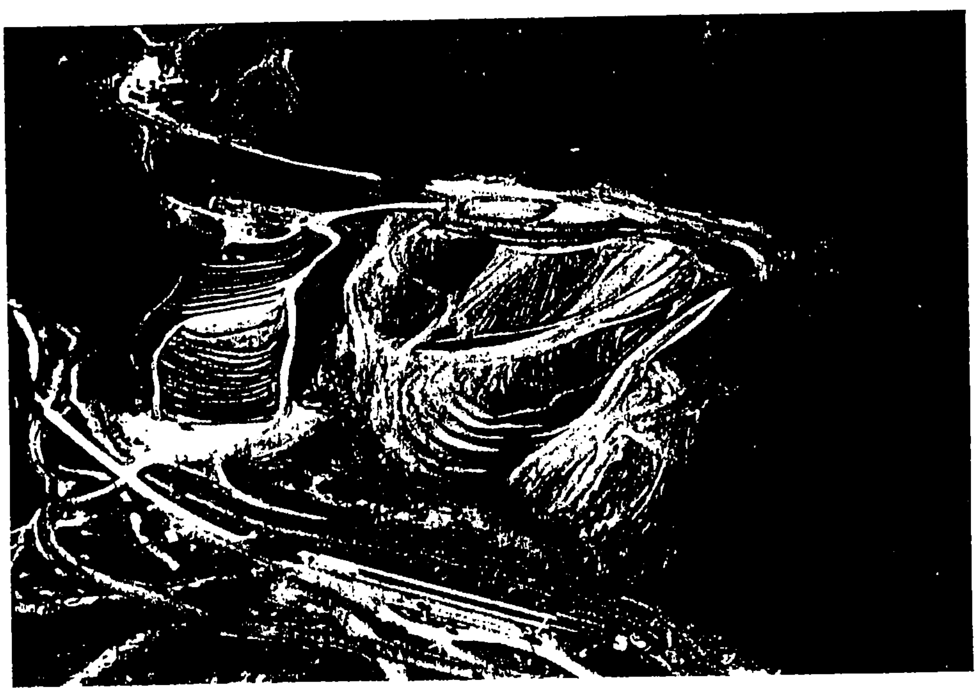

Figure 8. Large Embankment Area Along Future I-26 Construction Showing Underbenching of Natural Slopes Before Placement of Fill Material.

Figure 9. Large Side-Hill Fill Area Where Underbenching for Embankment Construction Has Clearly Marked the Landscape.

In some instances in-place rock material had to be removed to provide for the underbench construction. This is necessary in order to provide continuity for the rock pad and embankment stability.

A total of 2,413,496.6 cubic meters (3,159,027 yds.3) of material was removed during the underbenching process for the projects 41 embankments. Most of the material removed was unstable material (colluvium, soft soils, highly weathered rock). Some weathered and in-place rock was excavated in order to construct the underbenches as designed.

Once the underbenches were completed, then underdrains were installed where necessary and the rock pad construction proceeded. Rock for the rock pads was excavated from cut sections on the project and processed by the contractor on the project site. The rock was processed using a "grissley" or shaker screen system and hauled directly to each subject underbenched fill section.

The rock pad construction required the use of over 1,646,387.7 cubic meters (2,154,957.7 yds.3) of processed rock to complete. The rock consisted mostly of granite, granite gneiss, and banded gneiss and was of excellent quality.

The rock pads were constructed from the toe of each embankment and continued up to the top of the underbenched areas (Figure 10). The embankment construction proceeded in conjunction with the rock pad construction where feasible, and followed standard embankment construction specifications. In some instances toe berms, toe buttresses, and internal embankment shear keys were constructed along with the rock pads in the underbench areas (Figure 11).

ENVIRONMENTAL CONCERNS

The construction of U.S. 23 (Future I-26) in the mountainous terrain of Unicoi County, Tennessee resulted in the implementation of a number of environmentally related mitigation measures. Items of concern to the highway included: acid producing rock, serrated cut slopes, erosion control measures (temporary seeding, rip-rap ditches, siltation dams), bioengineering and habitat crossing boxes.

Acid Producing Rock - The identification and treatment of acid producing rock material encountered along highway construction projects has been gaining more attention and documentation in recent years (Huckabee, et al, 1975; Byerly, 1981 and 1990; Winchester, 1981; Eskensasy and Dunnagan, 1985). The problem with acid producing rock materials is the creation of acidic drainage (usually sulfuric acid along with high metal contents) that results from the breakdown of the sulfidic rock when it is placed in highway embankments.

During the design phase the Tennessee D.O.T. Geotechnical Operations Section investigated the possible occurrence of pyritiferous based acid drainage problems as a result of the proposed construction.

Net acid-base accounting (NAB) analysis was used to estimate the potential for acid drainage. A number of researchers have adequately described the analysis procedure (Byerly, 1981; Skovsen et al, 1987; Brady, et al, 1989; Soobek et al, 1978).

As a result of the investigation, approximately 68,760 cubic meters (90,000 cubic yards) of rock to be excavated was found to contain sufficient quantities of iron disulfides and the possibility of producing adverse acid drainage leachate.

Mitigation of the subject acid producing rock involved treatment or encapsulation (in accordance with Special Provision 107L - see Appendix A). A total of 152,800 cubic meters (200,000 cubic yards) of rock material was treated with agricultural lime @ 3,434 kg (3.38 tons of lime) 764 cubic meters (1,000 cu. yds. of rock) and placed in the roadway embankments.

Approximately 68,760 meters (90,000 cubic yards) of rock was found to have a high potential for producing acid leachate and was subsequently encapsulated.

Two encapsulation methods were employed on the project: A clay liner method and a geomembrane method (developed during the construction project).

Figure 10. Processed Rock from the Project is Placed Over All Underbench Areas to Provide for Drainage and Stability of the Embankment.

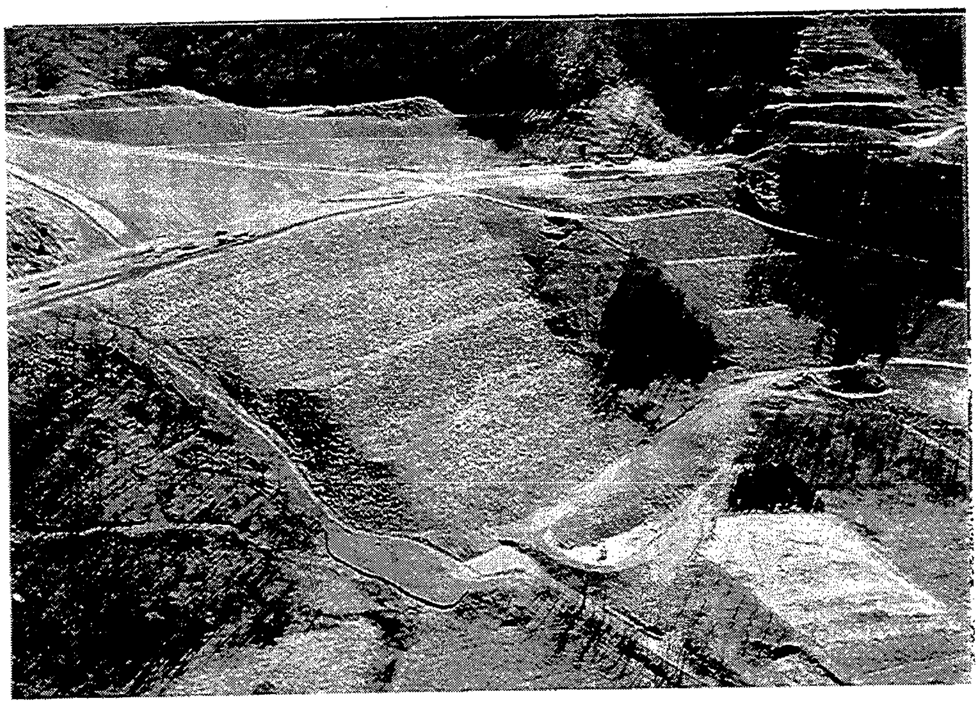

Figure 11. Large Roadway Embankment (in the Carver Hollow Area) that Contains Extensive Underbenching and Rock Pad on Future I-26, Unicoi County, Tennessee.

The clay liner method followed accepted procedures as researched and implemented on previous Tennessee Highway Projects (Byerly, 1981). Details of the clay liner method are as follows:

A 1.5 meter (5 foot) thick limestone rock pad was placed on the natural surface of the site. A thin choker layer of crushed stone and a layer of filter cloth were placed on the rock pad. Next, a 1.8 meter (6 foot) thick layer of clay was compacted into place above the filter cloth and also tied into the natural ground along an adjacent hill. After placement and treatment of the subject rock, the 1.8 meter (6 foot) thick clay liner was brought around and over the top of the pyritic material, encapsulating the material on all sides. Finally, top soil was placed over the clay and seeded.

GEOMEMBRANE METHOD

During the construction project Tennessee D.O.T. Geotechnical personnel investigated the possible use of a synthetic geomembrane for encapsulating the acid producing rock (Moore). Specifications and samples of selected geomembranes were reviewed in preparation of the possible use of the material. The geomembrane was used in place of the 1,8 meter (6 foot) thick clay liner. Using the geomembrane increased the amount of pyritic rock that could be encapsulated.

The following outlines the geomembrane method of encapsulation: The geomembrane method was used in conjunction with the construction of roadway fills.

The lower 4.5 meters (15± feet) of the subject embankment fill was constructed of rock [(lower 1.8-2.4 meters (6-8 feet)] and clay soil [next 2.1-3.0 meter (7-10 feet)]. On top of this material the two outer sides of the embankment were constructed leaving the interior a "hollow" trench (looking much like a ground silo). A 0.3 meter (1 foot) layer of very fine grained soil was placed on the interior of the trench, and compacted. All protruding rock fragments were removed in order to prevent puncture of the geomembrane.

The geomembrane used consisted of 60 mil thick high density polyethylene membrane with a friction textured surface (on both sides). The membrane material came in rolls 122 meters (400 feet) long and 6.7 meters (22 feet) wide. The geomembrane was placed into the trench with the seams of each strip of geomembrane being perpendicular to the roadway centerline. All seams were double welded, wedge heat bonded and vacuum tested. A 0.3 meter (1 foot) thick layer of agricultural lime was placed along the bottom and sides of the trench, on top of the geomembrane. Pyritic material was then placed in 0.6 meter (2 foot) thick lifts and treated with the agricultural lime 3,434 kg/764 cu meters (3.38 tons of lime/1,000 cu. yds. of rock) (Figure 12).

After placement of all the pyritic rock was completed, then a 0.3 meter (1 foot) thick layer of agricultural lime was compacted on top of the rock. The geomembrane was then sealed over the top and end, completely encapsulating the subject material (Figure 13).

Approximately 1.5 to 2.1 meters (5 to 7 feet) of clay material was placed over the top of the geomembrane bringing the embankment up to planned grade elevations.

Serrated Cut Slopes - In an effort to reduce soil and weathered-rock cut slope erosion a method of slope excavation called serrated cut slopes was employed. This procedure involves the excavation of small horizontal benches or "steps" into the remaining cut slope face (Figure 14). This is done as the cut is brought to grade by cutting each bench (or step) as each "lift" is excavated in the cut.

The size of the "steps" may vary depending on the required slope ratio. Most commonly used sizes were 0.6 to 0.9 meters (2 to 3 feet) for the "step" and 0.3 to 0.6 meters (1 to 2 feet) for the "riser".

The serrated slopes were used in areas where a steeper cut slope (1:5:1) was needed in saprolitic soils and weathered rock materials. Once the serrated slopes were seeded and mulched, vegetation established on the horizontal benches without experiencing major erosion problems.

Erosion Control Measures - During the construction project it was determined that a more aggressive erosion control program was needed. Existing erosion control measures were expanded to include temporary seeding, siltation dams, check dams and rip-rap ditches.

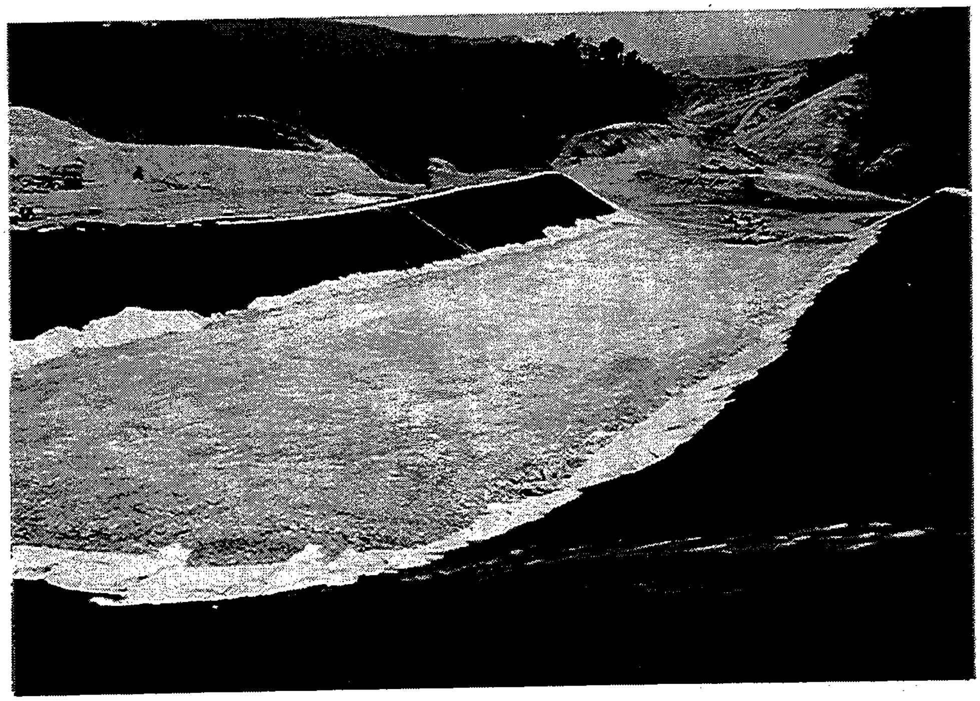

Figure 12. Pyritic Rock Encapsulation Area at the Upper Higgins Creek Road Interchange Illustrating the Geomembrane Placement (Black), Agricultural Lime Location (White) and the Pyritic Rock (Light Gray).

Figure 13. "Closure" of the Pyritic Rock Encapsulation Site Involves Laying the Geomembrane Over the Pyritic Rock/Agricultural Lime Layer and Covering with a Minimum of 1.5 Meters (5 Feet) of Clay.

Figure 14. Serrated Cut Slopes (Stair-Step Benching Procedure of Excavating Steep Soil Slopes) Were Used on the Project to Help Establish Vegetation on Steep Soil Slopes.

Areas that needed to be cleared and grubbed were immediately seeded in order to establish vegetation. This measure was taken in order to reduce the amount of erosion and siltation that was going to occur.

A number of large siltation dams were constructed to catch silt and clay run-off from the grading operations. These siltation basins contained filter fabric linens, rip-rap protection and perforated stand pipes (Figure 15). Special design was required on some of the larger basins where energy dissipaters were needed. One basin was over 0.4 hectare (one acre) in size.

In an effort to reduce and prevent erosion of the ditches, extensive use of rip-rap ditches were needed. The dimensions of the ditches varied depending on the field requirements. Rock was processed on the project by the contractors for use on the rip-rap ditches.

As a result of the erosion control measures used on this project, TDOT has greatly expanded the standard plans for erosion control on all projects in the state.

Bioengineering - An experimental project involving the use of tree cuttings (called bioengineering Gray & Sotir, 1992) was implemented on the Unicoi County Project. The concept involves taking cuttings from trees such as willows, birches, maples, and tulip poplars and creating long bundles 9.1 meter long and 2.54 cm in diameter (~ 30' long and 1" diameter) of the cuttings called a "live fascine". These bundles are then placed in long trenches and partially buried with soil. As new growth emerges from the trenches, vegetative cover is quickly established.

This concept was applied in three different locations to test its effectiveness. The locations were along a relocated creek channel, a soil/weathered rock serrated cut slope, and a soil slope on a selected waste site.

Partial success was accomplished in this trial bioengineering application. The cut slope and waste site locations responded effectively with substantial new growth the first season. The channel relocation plantings were affected by local drought conditions that made the growth spotty at best.

Habitat Crossing Boxes - In an effort to provide safe access to both sides of the new highway for wildlife (including deer and black bear), a type of box culvert referred to as "habitat boxes" were constructed. Since the new roadway used a concrete median barrier to divide the traffic lanes, access across the road for the wildlife was anticipated to be difficult and quite deadly.

The upper two miles of the roadway did not contain any bridges and as such was selected for the use of the habitat boxes. Fencing was used in the vicinity of the habitat boxes to direct the wildlife to and from the crossing areas. Monitoring of these crossing areas will detail their effectiveness.

PROJECT STRUCTURES

A roadway project of this magnitude requires the construction of bridges and retaining walls to compliment the demanding requirements of the terrain. Bridge construction consisted of conventional type concrete and steel design. The unusual aspects of the bridges were their application to environmental and geotechnical conditions.

Where the route crosses South Indian Creek (four places) the bridges were extended lengthwise in order to maintain as much of the Flood Plain environment as possible. These bridges were 244 to 304.8 meters (800 to 1000 feet) in length with very short to non-existent approach fills (Figure 16).

Geotechnically there were three locations where structures were used to bridge over unstable colluvial material. Bridges were constructed over these unstable masses which extend upslope and downslope over 304.8 meters (1000 feet) from the roadway.

Steel girder bridges were constructed over the colluvial ravines and were 183 to 274 meters (600 to 900 feet) in length (Figure 17). All bridge foundations were located on sound in-place bedrock of varying lithology types. These structures were also designed to the new earthquake design load criteria.

Retaining Walls - In order to prevent the construction of long thin "sliver" fills on the steep terrain, it was decided to use mechanically stabilized earth retaining walls on the project. Reinforced Earth, Inc. (R.E.) was chosen as the alternate to conventional concrete cast-in-place retaining wall systems.

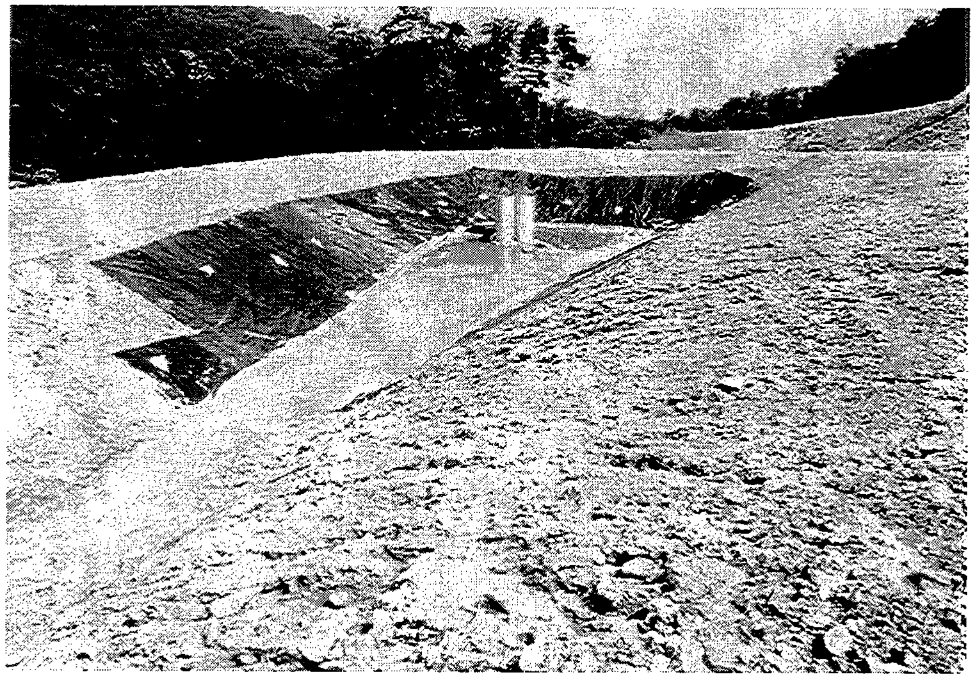

Figure 15. Siltation Dams and Sediment Basins Were Used on the Project in an Effort to Reduce the Water Pollution Effects on South Indian Creek.

Figure 16. Several Bridges Were Constructed Over South Indian Creek and One (Shown) Over the Creek and Old U.S. 23.

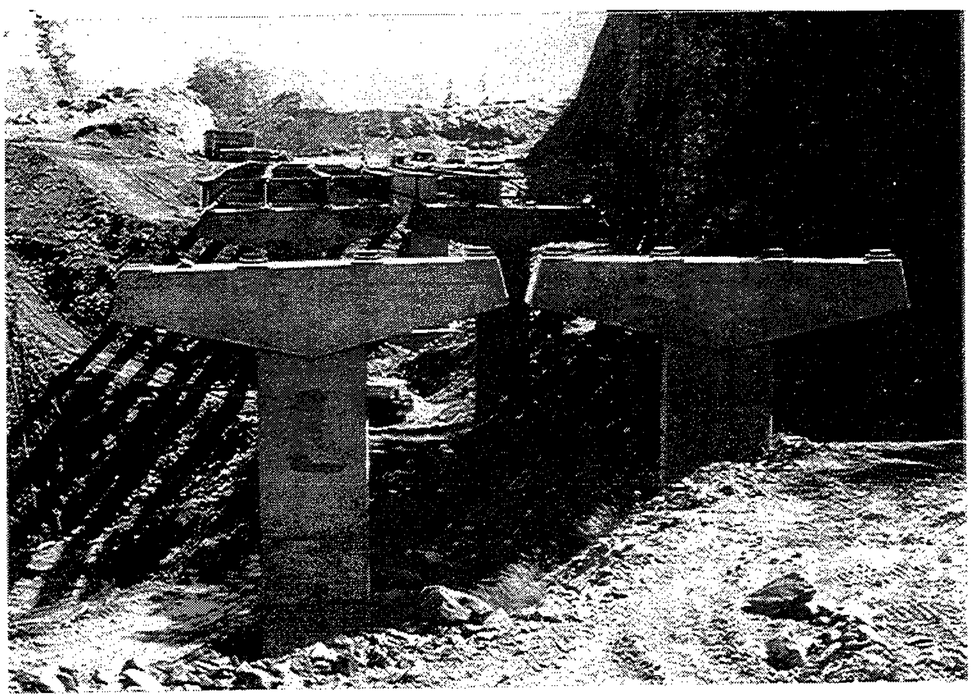

Figure 17. Large Steel Girder Bridges Were Constructed Over Unstable Colluvial Ravines Along the Mountainous Slopes of the Project Area.

The mechanically stabilized earth retaining wall system (commonly referred to as "M S Walls") involves the use of face panels, steel reinforcement backing strips, and granular backfill. The metal strips are attached to the back of the facing panels. Depending on the design there may be five to seven strips attached to each panel. These metal strips extend back behind the face panels at varying distances [3.6 to 12 meters (12 to 40 feet) depending upon design requirements]. The granular backfill is placed over the metal strips in 20 to 25 cm (8 to 10 inch) lifts and compacted. The result is that the weight of the compacted granular backfill produces sufficient friction between the backfill and the metal strips to resist shearing.

The face panels serve primarily to hold the backfill material in place and keep it from eroding out. The entire mass (face panels, metal strips, and granular backfill) acts as the wall system , not just the face panels.

In several locations along the project, the "M S Walls" were "stacked" on top of each other creating a stair-step effect (Figure 18). In two places on the project three walls of 7.6 meters (25 feet) in height each were constructed on top of each other, stepping in toward the slope approximately 25 feet for each wall. In a third area this was done for a two wall system structure.

The intent of this design was to "catch" embankment slopes high up on the ridge surface preventing the "sliver" fill situation. This design also reduced the amount of vegetative cover that was disturbed - a positive effect on the environment.

Another type of retaining wall system employed on this project was a tie-back wall. This type of wall system involves the placement of soldier piles and whalers with rock and/or soil anchors drilled into stable material behind the wall face. The utility of this wall system is that the wall can be constructed from the "top down" instead of the conventional "bottom up" approach.

The tie-back wall system employed on this project involved a multi-step construction approach. First, holes are drilled along the wall face length at pre-determined spacings and depths. Soldier piles (typically steel H-beams) are placed in the holes and grouted. Next, the top 3.0 to 3.6 meters (10 to 12 feet) of ground is excavated down along the roadway side of the wall face. At this point, holes are drilled back into the soil-rock mass, usually at some pre-determined angle from horizontal (15° - 30°) . Steel bars or wire cables are placed into these drilled holes and anchored at the end of the hole, usually with concrete grout.

A whaler is then placed across the face of two soldier piles and the cable tendon or steel bar is extended through a pre-drilled hole in the whaler. The cable tendon/steel bar is then connected to a jack which then pulls the tendons and/or bar to a pre-determined loading. The cable tendons/bar are then locked off at the required loadings as per the design (Figure 19).

All tendons/bars are provided double corrosion protection as a minimum. A concrete facing was placed over the tie-back wall face providing a "conventional looking" concrete retaining wall.

The tie-back wall was used on this project to hold back a large mass of unstable colluvial material near the top of the mountain at Sams Gap. A 7.6 - 9.1 meter (25-30 foot) high cut through the colluvium was necessary for the roadway location. The use of the tie-back wall at this location was most effective in preventing future slope stability problems.

Rockfall Control Fences - With the construction of numerous large high rock slopes on this project, it was decided that some type of rockfall analysis and possible mitigation was needed. The rock mass along the project as a whole is very fractured and contains numerous water seeps. Numerous freeze-thaw cycles are common in this geographic area and usually produce "ice jacking" of the rock faces.

A series of rockfall field tests were performed on the subject rock cut slopes. These tests involved rolling rocks off of the rock slopes and monitoring their trajectories and stopping locations. In addition, numerous computer simulated rockfall trials were performed in the geotechnical office to compare with the actual field tests.

As a result of these studies, a number of locations were identified as having significant rock fall potential as to impacting the safety of the motoring public. To this end over 2,133.6 meters (7,000 linear feet) of rock fall control fence was designed and installed. The rock fence chosen was the Brugg Wire Rope Net Catchment Fence (Figure 20).

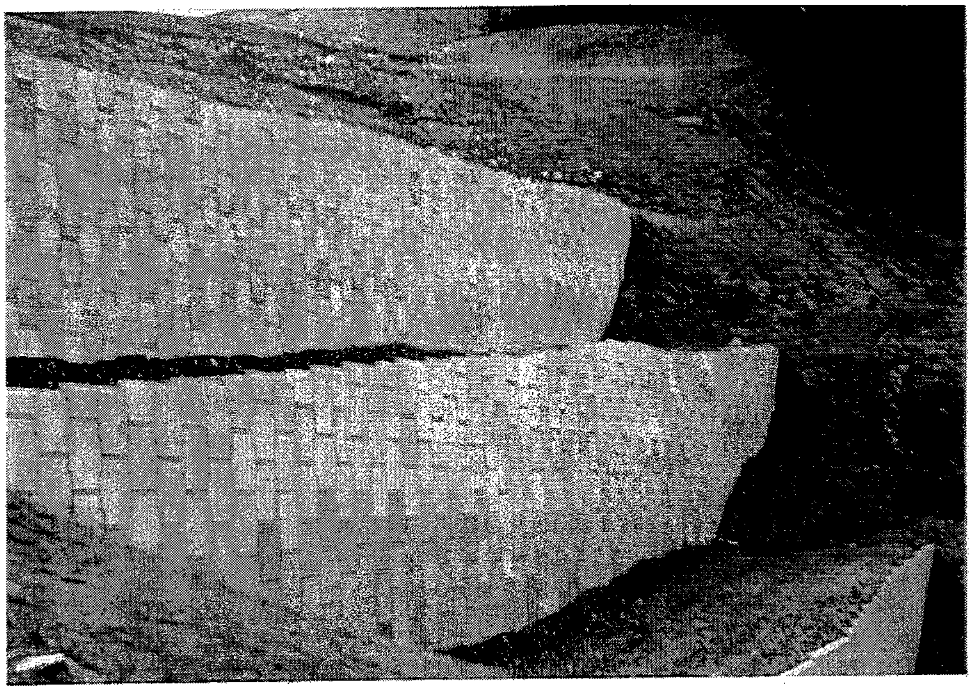

Figure 18. Several Mechanically Stabilized Earth Walls (Reinforced Earth, Inc.) Were Constructed to Prevent Side Hill Fills Extending Down Slope.

Figure 19. This Tie-Back Wall Located Near Sams Gap Illustrates the Soldier Pile and Wood Lagging Concept with the Actual Tie-Back Anchors Protruding from the Soldier Piles.

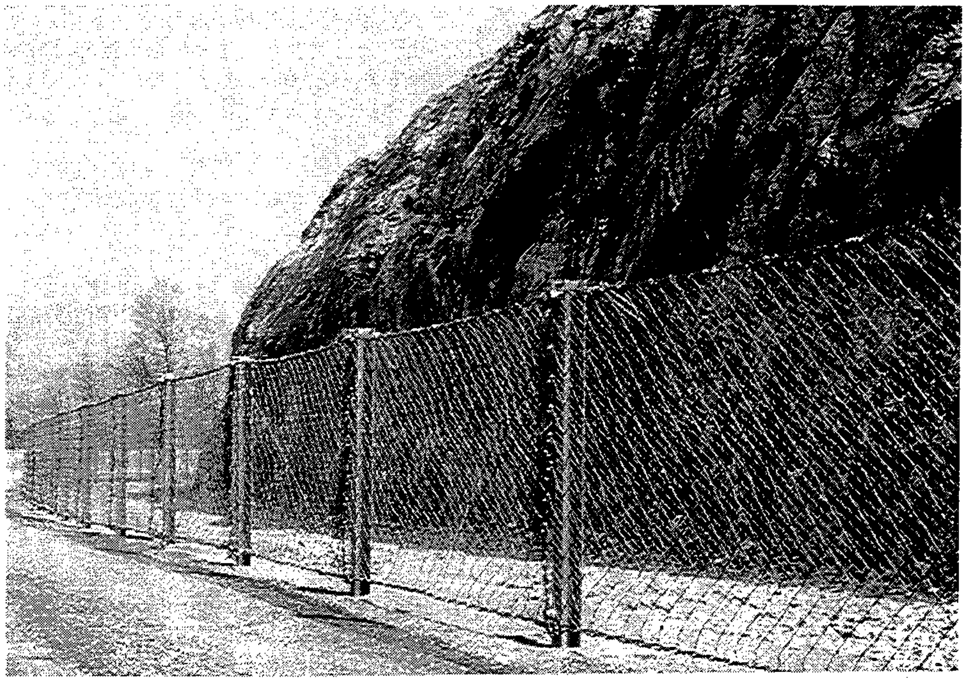

Figure 20. Rockfall Catchment Fences (Brugg, Inc.) Were Constructed in Areas that Were Determined to Have a Potential for Rockfall Events.

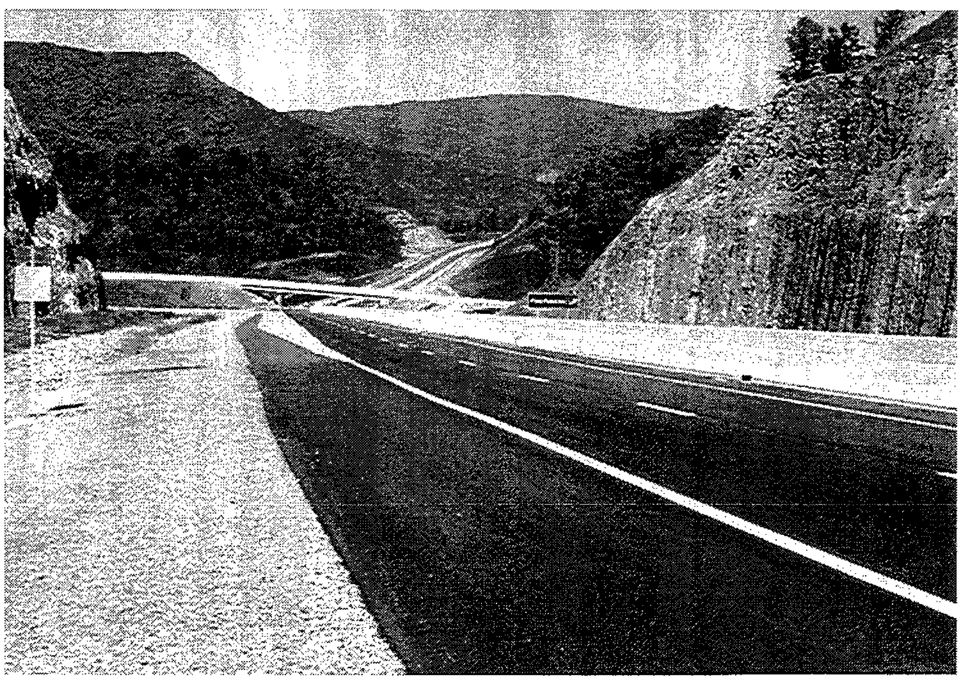

Figure 21. The Completed Roadway Was Open to Traffic July 5, 1995. Shown Here is the Flag Pond Exit near Mile Post 5.

The design by Brugg Cable, Inc. (Santa Fe, N. Mexico) involved the use of a wire rope net connected to a cable system having an energy breaking system of "loops". As a rock impacts the fence, the force of the impact pulls the wire rope net transferring the stress to the supporting cable system. A series of loops placed along the cable contain a torqued clamp that will give as an appropriate force is applied. This breaking device allows the energy to be dissipated slowly as the loop is pulled out by the impact.

These fences were designed to withstand impacts of 20,738 kg-m (75 foot-tons) of force - more than adequate for most rock fall events. Over seven thousand linear feet were installed at various cut intervals on the project in an effort to reduce future rock fall - related traffic accidents and injuries.

Summary - This unique project - the completion of a 24 kilometer (15 mile) section of four-lane controlled access highway through mountain terrain - was completed in late spring of 1995 (May-June) and officially opened to the public on July 5, 1995 (Figure 21). Considerations regarding the engineering geology were monumental and included items such as rockslope stability, rock fall, embankment stability, groundwater drainage, acid drainage, and foundation conditions for bridges and retaining walls. Environmental considerations regarding the project included erosion control, wildlife habitat (particularly black bear and trout) water pollution, and vegetation of the roadway slopes.

Due to the scope of the project, some unique totals of project materials and data were experienced. The following is a list of unique project data:

Length - 24.1 kilometers (15.291 Miles)

Total Cost - $170 Million (design, right-of-way, envir. engineering, and construction)

Excavated Material - 20,271,295 cubic meters (26,533,109 cubic yards)
(enough to fill UT Neyland Stadium 21 times)

Base stone and asphalt mix laid - 907,200,000 kg (1,049,839 tons)

Concrete Poured - 73,483.8 cubic meters (96,183 cubic yards)

Erosion control matting laid - 48.2 hectares or 264 km 1.8 meters wide matting (119 acres or 164 miles six feet wide matting)

Pavement marking materials -9,463.5 liters of paint (2,500 gallon of paint)

Drainage pipe used - 14.5 km (9 miles)

Hay for temporary erosion control during construction - 38,000 bales

Temporary silt fencing for erosion control - 32.2 km (20 miles)

Temporary slope drainage pipe - 18.5 km (11.5 miles)

Total number of bridges - 22

Total length of bridges - 2,393.6 meters (7,853 feet)

Total Erosion Control - $6,537,438.00 (incl. 14 sed. trap struct., 4,549.6 square meters (48,877 sq. ft.) rock check dams, 39,422.2 cubic meters (51,599.7 cyds) of sed. removal)

Total bear boxes (Habitat Boxes) - 2

2,145.8 meters (7,040 Feet) of 3 meter (10') high rock catchment fence

Encapsulation of over 53,480 cubic meters (70,000 cubic yards) of pyritic rock
(cost over 2 million dollars)

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APPENDIX A

Cumulative Index of HGS Proceedings

CUMULATIVE INDEX

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS

VOLUME I - First, Second, Third and Fourth Highway Geology Symposia

First Annual Symposium on "Geology as Applied to Highway Engineering" - April 14, 1950 - Department of Highways - Richmond, Virginia.

Parrott, W. T., "Synopsis of First Highway Geology Symposium." (Papers presented not published)

Second Annual Symposium on "Geology as Applied to Highway Engineering" - February 16, 1951 - Virginia Department of Highways - Richmond, Virginia.

Cornthwaite, A. B., "The Adhesion of Bituminous Films to Highway Aggregates."
Cooper, B. N., Dr., "Geological Enterprise in Virginia -- Present and Future."
Whitmore, Frank C., Jr., "The Importance of Geology in Military Highway Construction."
McConnell, Duncan, Dr., "Detrimental Minerals in Concrete Aggregate."
Granger, A. T., "What Does the Engineer Expect of the Geologist?"
Hicks, L. D., "The Use of Plate Bearing Tests in the Thickness Design of Flexible Pavements."
DeBuchananne, George D., "The Control of Groundwater in Consolidated Rocks."
Woolf, D. O., "The Identification of Rock Types."
Wheeler, Frank W., "The Construction of Highway Bridges and Separation Structures in Unconsolidated Sediments."

Third Annual Symposium on "Geology as Applied to Highway Engineering" - February 29, 1952 - Virginia Department of Highways and Virginia Military Institute - Lexington, Virginia.

Dobrovolny, Ernest, "Application of Geology to Highway Work as Practiced by the States, a Questionnaire Summary."
Nicol, Allen H., "Studies of Aggregates on Okinawa."
Nesbitt, Robert H., "Problem Minerals in Concrete Aggregates of the Southeastern States."
Anderson, Rex S., "Geology and Highway Engineering -- A Continuing Mission."
Baker, R. F., "The Design of the Slopes of Highway Rock Excavations in West Virginia."
Hadley, Jarvis B., "The Geology of a Highway Slide at Gatlinburg, Tennessee."
Scroggie, Everette, "The Application of Geology to Bridge Foundations."
Parrott, W. T., "Geological Problems in the Design and Construction of Highways in Virginia."

Fourth Annual Symposium on "Geology as Applied to Highway Engineering" - February 20, 1953 - Morris Harvey College and West Virginia State Road Commission - Charleston, West Virginia.

Gregg, L. E., and Havens, James H., "Applications of Geology to Highway Engineering in Kentucky."
Marshall, Harry E., "Some Experiences of the Department of Highways with Landslides in Ohio."
McNeal, John D., "The Application of Geology to Highway Subdrainage in Kansas."
Woods, K. B.; Johnstone, J. C.; and Yoder, E. J.; "Some Engineering Problems Associated with the Preglacial Marrietta River Valley."
Price, Paul H., "The Landslide Problem."
Cavendish, Ray, "Landslides Affecting West Virginia Roads."
Eckel, Edwin B., "Contributions by the U. S. Geological Survey to Highway Engineering Research."
Philbrick, Shailer S., "Design of Deep Rock Cuts in the Conemaugh Formation."

VOLUME II - Fifth and Sixth Highway Geology Symposia

Fifth Annual Symposium on "Geology as Applied to Highway Engineering" - March 16, 1954 - Ohio State University and the Ohio Department of Highways.

Melvin, John H., "Correlation of Geological Studies by Various Governmental Agencies in Ohio."
Hyland, John R., "Aids of the Ohio Geologic Survey for Highway Engineers."
Warrick, W. A., "Application of Geology to Highway Engineering as Seen by a Highway Engineer."
Mason, Neil E., "Ohio's Experience in the Use of Geophysical Methods in Subsurface Exploration."
Norris, Stanley E., "Importance of Ground Water Studies to Highway Engineering."
Lewis, D. W., "Effect of Coarse Aggregate on Concrete Durability."
Mintzer, Olin, "Geology in the Engineering Curriculum, the Highway Engineer's Viewpoint."
Supp, Carl W. A., "Geological and Soils Engineering on Ohio Turnpike Project No. 1."
Barger, Lewis, "Three Dimensional Aspects of Landslides."

Sixth Annual Symposium on "Geology as Applied to Highway Engineering" - February 18, 1955 - Johns Hopkins University and the Maryland State Roads Commission.

Singewald, Joseph T., Dr., "The Role of the Geological Surveys in Geology as Applied to Highway Engineering."
Barber, Edward S., "Geology and Foundations."
Smith, Preston C., "Use of Aerial Photographs in Engineering - Soil Mapping."
Melville, Phillip L., "Concrete Aggregate Reaction in Virginia."
Mobley, Arthur B., "Earth Resistivity Indicates Subsurface Geology."
Supp, Carl W. A., "Engineering Geology of the Chesapeake Bay Bridge."
Allen, Alice S., "Sources of Information on Ground Conditions."

VOLUME III - Seventh, Eighth, Ninth and Tenth Highway Geology Symposia

Seventh Annual Symposium on "Geology as Applied to Highway Engineering" - February 24, 1956 - N. C. State Highway and Public Works Commission - Raleigh, North Carolina.

Baker, Robert F., "Engineering and Landslides."
Mather, Bryant, "Application of Petrographic Procedures to Highway Engineering."
Deuterman, Martin, "Investigation of Bridge Foundations."
Laursen, Emmett M., "River Bed Scour at Bridge Foundations."
Laurence, Robert A., "Geologic Features of the Eastern States as Related to Highway Engineering."
Bickel, J. O., "Equipment Used in Geological Engineering."
McCullough, Charles R., "Aerial Photos and Highway Engineering."

Eighth Annual Symposium on "Geology as Applied to Highway Engineering" - February 15, 1957 - the Pennsylvania State University and Commonwealth of Pennsylvania - Department of Highways.

Eckel, Edwin B., "New Developments in the Study of Landslides."
Gary, Carlyle, Dr., "Geological Information in Pennsylvania for the Highway Engineer."
Engel, Harry J., "Bridge Foundation Experiences."

Weeden, Harmer A., "Pedology Helps the Highway Engineer."
Bird, Paul H., "Experience in Designing Rock Slopes in New York State."
Baker, William O., "Photogrammetry in Practice."

Ninth Annual Symposium on "Geology as Applied to Highway Engineering" - February 21, 1958 - University of Virginia and Virginia Department of Highways.

Gay, Archer B., "Highway Geology and the Contractor."
Legget, Robert F., "Geology and Transportation Routes."
Maner, Alfred W., "Soils and Geological Engineering Partners, Not Competitors."
Rice, James M., "A Rapid Method for Determining the Resistance of Ledge Rock to Freezing."
Dole, George J., "Ammonium Nitrate as an Explosive."
Tuttle, Curtis R., "Application of Seismology to Highway Engineering Problems."

Tenth Annual Symposium on "Geology as Applied to Highway Engineering" - February 20, 1959 - Georgia Institute of Technology and Georgia State Highway Department - Atlanta, Georgia.

Felix, George D., "Geology as an Aid to Right-Of-Way."
Furcron, A. S., Dr., "Distribution and Character of Stone for Aggregate in Georgia."
Marshall, Harry E., "Design Considerations in the Treatment of Soft Foundations."
Upham, Charles M., "The Use of Geological Investigations in Foreign Consulting Work."
Seeger, Ralph W., "Highway Material Survey in West Virginia."
Fletcher, G. A., "Geology in Foundation Engineering."
Belcher, Donald J., "Applied Geomorphology."

Eleventh Annual Highway Geology Symposium - February 26, 1960 - Florida State University - Tallahassee, Florida.

Vernon, Robert O., "The Geological Distribution of Highway Base Course Material and Aggregate in Florida."
Mather, Bryant, "Petrology of Concrete Aggregate."
Carey, W. N., Jr., and Schmidt, W. J., "The AASHO Road Test: A Progress Report."
Michener, P. Z., "Preliminary Subsurface Investigation of the Chesapeake Bay Crossing."
Fritz, Axel M., Jr., "The MD Engineering Seismograph and its Application to Highway Engineering."
Bruun, Per, "Beach Erosion and Protection in Florida."
Lynch, S. A., and Weaver, Paul, "Pavement Disruption by Recent Earth Movements."
Radzikowski, Henry A., "Rock and Earth in the Highway Program."

Twelfth Annual Symposium on "Geology as Applied to Highway Engineering" - February 10-11, 1961 - Engineering Experiment Station, Bulletin Number 24 - the University of Tennessee, Knoxville.

Young, Norman C., and Pierce, T. R., "Principal Highway Engineering Characteristics of Some Tennessee Formations."
Goodwin, William A., "Evaluation of Pavement Aggregates for Non-Skid Qualities."

Nichols, Donald R., and Yehle, Lynn A., "Highway Construction and Maintenance Problems in Permafrost Regions."
McNutt, Charles, "Highway Salvage Archaeology."
Officer, Charles B., "Use of Continuous Seismic Profiler (Sparker) in Geologic Investigations for Vehicular Tunnel and Bridge Crossings."
Bailey, Reed W., "Madison River - Hebgen Lake Earthquake and Highway Problems."
Stuart, W. Harold, "Geological Conditions Complicating Highway and Railroad Relocations in the Northwest."
Moore, R. Woodard, "Observations on Subsurface Explorations Using Direct Procedures and Geophysical Techniques."
Dorman, C. W., "The Economics of Natural Resource Valuation."
Gutschick, Kenneth A., "Altering Physio - Chemical Characteristics of Clay - Bearing Soils with Lime."

<u>Thirteenth Annual Highway Geology Symposium</u> - March 16, 1962 - Phoenix, Arizona.

Hall, Bruce M., "Status of Geological Registration."
Gallaher, B. J., "Desert Materials -- Types and Uses."
Eckel, E. B., "Use of Engineering Geology Maps."
Smith, P. C., "Purpose of the Highway Materials Inventory."
Sergent, B. D., "Soil Mechanics in the Southwest."
Mitchell, Stanley N., "Problems Facing the Engineer and the Geologist in a Highway Department Employing an Engineering Geologist."
Kersch, George A., "Regional - Aerial Geological Investigations in Highway Geology."

<u>Fourteenth Annual Highway Geology Symposium</u> - March 22, 1963 - A. & M. College of Texas - College Station, Texas, Texas Highway Department, and the Texas Bureau of Economic Geology.

Pelzner, Adrian, "Applications of Agricultural Soil Surveys to Highway Geology."
Laughter, C. N., "Correlation of Culvert Performance and Soil Conditions."
Henry, H. A., "Engineering Geology Operations in the Texas Highway Department."
Benson, Gordon R., "Geology -- A Vital Part of Subsurface Engineering in Illinois."
Welp, Theodore L., "Materials Geology, Coordinator of the Aggregate Inventory."
Lemish, John, "Carbonate Aggregate Research."
Moore, Richard T., "The Effect of Fractured Ground on Highway Structure Design."
Leith, C. J., and Gupton, C. P., "Some Geologic Factors in Highway Slope Failures in North Carolina."
Drew, E. D., "The Development and Utilization of Engineering Geology in the California Division of Highways."

<u>Fifteenth Annual Highway Geology Symposium</u> - March 19, 1964 - Missouri Geological Survey and Water Resources - Rolla, Missouri.

Helmer, R. A., "The Duties and Training of a Geologist in the Oklahoma State Highway Department."
Grimes, Walter W., "Geology and Foundation Problems of Glacial Drift in Eastern South Dakota."
Migliaccio, Ralph R., "Engineering and Construction Problems in the Valdez District, Alaska."

Landrum, J. D., "A Foundation Investigation of Cherokee Cave Under Route I-55, City of St. Louis."
Lounsbury, W. R., and Schuster, R. L., "Petrology Applied to the Detection of Deleterious Materials in Aggregates."
West, T. R., and Aughenbaugh, N. B., "The Role of Aggregate Degradation in Highway Construction."
Heagler, John B., Jr., "Mineralogy and Soil Stabilization."

Sixteenth Annual Highway Geology Symposium - March 25-26, 1965 - University of Kentucky - Lexington, Kentucky.

Goodwin, W. A., "The Application of Geology in the Benefication of Aggregates."
Laughlin, G. R.; Scott, J. W.; and Havens, J. H., "Freeze-Thaw Characteristics of Aggregates."
Lounsbury, R. W., and West, T. R., "Petrography of Some Indiana Aggregates in Relation to Their Engineering Properties."
Sherwood, W. Cullen, Dr., "The Role of Aggregate Type in Pavement Slipperiness."
Smith, Preston C., "Landslide Research."
Struble, Richard A., "Shallow Subsurface Exploration Utilizing Airphoto Interpretation and Geophysical Techniques."
Dobrovolny, Ernest, "The Highway Research Board and its Committee on Engineering Geology."

Seventeenth Annual Highway Geology Symposium - April 21-23, 1966 - Iowa State University and Iowa State Highway Commission - Ames, Iowa.

Burgat, Virgil A., "Engineering Geology in Kansas Highway Construction."
Mitchell, Robert E., "Seismic and Resistivity Matured."
Michael, Robert D., "Techniques of Engineering Geology in Evaluation of Rock Cores for Construction Materials Sources."
Blattert, Robert E., "The Effects of Peat Deposits on Highway Design in Iowa."
Eversoll, Duane A., and Burchett, Ray, "Nebraska Geology and Highway Engineering Procedures."
Bruce, Richard L., "Landslides in the Pierre Shale of South Dakota."
Kneller, William A., "A Study of Chert Aggregate Reactivity Based on Observations of Chert Morphologies Using Electron Optical Techniques."
Fritz, Axel M., "Engineering Geophysics: Its Use and Abuse."
Ledbetter, John F., "Aspects of Using a Borehole Deflectometer to Diagnose an Unstable Rock Slope."
Welp, T. L. and Others, "Panel Discussion on Chemical and Physical Reactions of Carbonate Aggregates in Concrete."
McElherne, T. E., "Introduction."
Lemish, John, "Background on Carbonate Aggregate Behavior in Concrete."
Mather, Katharine, "Test Methods Used in Investigating Alkali - Carbonate Reaction."
Gillott, J. E., "Concrete Performance as Related to the Behavior of Carbonate Aggregates."
Newlon, Howard, "Chemical and Physical Reactions of Carbonate Aggregates in Concrete."
Axon, Ely O., "Concrete Performance as Related to the Behavior of Carbonate Aggregates."
Mather, Katharine, "Waterways Experiment Station Experience with Alkali - Carbonate Reaction."
Newlon, Howard, "Physical Test Methods Used in Investigating Aggregates."
Dunn, James, "Distress of Aggregate by Absorbed Water."

Lemish, John, "Concrete Weathering Studies."
Panelists and Audience, "Questions and Answers."

Eighteenth Annual Highway Geology Symposium - April 20-21, 1967 - Purdue University and Indiana Highway Commission - Lafayette, Indiana.

Woods, K. B., "Some Highway Problems of the United States Correlated with Physiographic Provinces."
Miles, R. D., "Anyone Can Interpret Soil Groups from Aerial Photographs."
Struble, R. A., and Mintzer, O. W., "Combined Investigation Techniques for Procuring Highway Design Data."
Berliant, R. F., and Sanborn, A. F., "Comprehensive Investigations Facilitate Design of Interstate Highways over Bottomland Soils."
Johnson, Robert B., "The Use and Abuse of Geophysics in Highway Engineering."
Vineyard, Jerry D., and Williams, James H., "A Foundation Problem in Cavernous Dolomite Terrain."
Fredericksen, Walter, "Stabilization of Abandoned Mine Under an Interstate Highway."
Hotler, C. F., "Soil Survey Practices in Indiana."
French, Robert R., and Carr, Donald C., "Geologic Factors Affecting the Exploration for Mineral Aggregates in the Indianapolis Area."
Malott, D. F., "Shallow Geophysical Exploration by the Michigan Department of State Highways."
Panel Members, Panel Discussion: "Preliminary Exploration for Highways with Emphasis on Local Problems."

Nineteenth Annual Highway Geology Symposium - May 16-17, 1968 - West Virginia Geological and Economic Survey, State Road Commission of West Virginia, West Virginia University Department of Geology, and West Virginia University College of Engineering.

Donaldson, Alan C., "Geology of West Virginia with Special Reference to the Field Trip Area."
Hayes, Russel R., and Worrell, Donald T., "Geologic and Engineering Implications of Poor Quality Sandstone from Simulated Highway Tests."
Leach, Richard C., " The Problem and Correction of Landslides in West Virginia."
Thompson, Berke L., and Long, Donald C., "A Controlled Fill over Sediments of Ancient Lake Monongahela near Fairmont, West Virginia."
Seeger, Ralph W., "Geological Investigations for a Trans - Andean Highway."
Smith, James D., "Geology -- Its Relation to the Design of the East River Mountain Tunnels."
Cooper, Byron N., "Geology of Big Walker Mountain Tunnel on Interstate Route 77, Wythe and Bland Counties, Virginia."
McGrain, Preston, and Dever, Garland R., Jr., "The Geometry of Limestone Aggregate Sources in Kentucky's Appalachian Region."
Arkle, Thomas, Jr., and Brock, Samuel M., Jr., "The Geology of Construction Sand and Gravel Resources in West Virginia."

Twentieth Annual Highway Geology Symposium - April 16-19, 1969 - University of Illinois - Champaign-Urbana, Illinois.

Thornburn, T. H., and Liu, T. K., "Soils of Illinois and Their Engineering Characteristics."
Brownfield, Robert L., "A Geotechnology Profile in Jo Davies County, Illinois."
Keene, Kenneth R., "Problems with Highway Cuts in Loess near East St. Louis, Illinois."
Harvey, Richard D., "Fracture Surfaces of Carbonate Aggregates: A Scanning Electron Microscope Study."
Gamble, James C., Hendron, A. J., Jr., and Way, Grover C., "Foundation Exploration for Interstate 280 Bridge over Mississippi River near Rock Island, Illinois."
Ferland, Jack A., "Seismic Mapping of Cavities and Voids."
Thompson, M. R., "Properties of Lime - Treated Soils."

Twenty-First Annual Highway Geology Symposium - April 23-24, 1970 - University of Kansas - Lawrence, Kansas, State Highway Commission of Kansas, State Geological Survey of Kansas.

Wilson, Frank W., "Highway Problems and the Geology of Kansas."
Goodfield, Alan G., "Rock Falls and Landslides along State Highway 79 at Clarksville, Missouri."
Hammerquist, D. W., and Hoskins, Earl, "Correlation of Expansive Soil Properties and Soil Moisture with Pavement Distress in Roadways in Western South Dakota."
Taylor, Charles L., "Geometric Analysis of Rock Slopes."
Sennett, Robert B., "Geologic Factors in Design of Excavated Rock Slopes."
Jenkins, Gomer, Jr., "Hydraulic Borrow Materials in Urban Areas."
Eversoll, Duane A., "Amphibious Drilling Rig."
Whitfield, John W., and Williams, James H., "Underground Transportation Routes and Quarry Practices in the Kansas City, Missouri - Kansas Area."
West, Terry R., "Application of Remote Sensing to Highway Locations."
Stallard, Alvis H., "Remarks on Kansas Highway Research Project in Remote Sensing for Soils and Geologic Mapping."
Hartley, Alan, "The Influence of Geological Factors upon the Mechanical Properties of Road Surfacing Aggregates."

Twenty-Second Annual Highway Geology Symposium - April 22-23, 1971 - University of Oklahoma - Norman, Oklahoma, Oklahoma Geological Survey, Oklahoma Department of Highways.

Mankin, Charles J., and Johnson, Kenneth S., "Geology of Oklahoma – A Summary."
Hayes, Curtis J., "Engineering Classification of Highway – Geology Problems in Oklahoma."
Laguros, Joakim G., and Kumar, Subodh, "Predictability of Shale Behavior."
Deere, Don U., and Gamble, James C., "Durability - Plasticity Classification of Shales and Indurated Clay."
Bayless, Glen, "Post-Construction Performance of P-18 Bridge Abutments and Approach Fills, Eastern Oklahoma."
Parrott, William T., "Control of a Slide by Vertical Sand Drains, U. S. Route 220, Alleghany County, Virginia."
Merten, Fred K., "Straight Creek Tunnel Construction, Route I-70, Colorado."

Ivey, John B., "Highway Geology Feasibility Study, Luluabourg to Mbuji-Mayi Republic Of Congo (Kinshasa)."
Cleaves, Arthur B., "The Camino Marginal, Peru."
Gedney, David S., "A Design Approach to Rock Slope Stability."
Thompson, Berke L., "The Use of Air as a Drilling Medium for Subsurface Investigation."
Johnson, B. J., "Penetrohammer, the Penetrometer Machine and its Application."

Twenty-Third Annual Highway Geology Symposium - April 27-28, 1972 -Virginia Highways Research Council, Virginia Department of Highways, Virginia Division of Mineral Resources, Old Dominion University - Hampton, Virginia.

Jimenez, John, "Embankment Failures on the Tijuana -- Ensenada Tumpike in the Lower California Peninsula, Mexico."
Branthoover, Gerald L., "Geological Engineering Investigation of Talus Slopes in Lewiston Narrows, Pennsylvania."
Herbold, Keith, "Cut Slope Failure in Residual Soil."
Vogelsand, W. H., and Munyan, A. C., "Engineering Geology of the Semi-Indurated Strata of the Virginia Coastal Plain."
Thomas, Carl O., "Remote Sensing Applications to Near Surface Geology."
Mitchell, D. A., and Brack, C., "Ecological Impact of Hydraulic Construction Methods in Georgia."
Whitlow, B. S., "Investigation of Deterioration in Concrete Roadway Slab of the Robert E. Lee Bridge, Richmond, Virginia."
Baldwin, J. S., and Dawson, J. W., "Effects of Angular Sands on Portland Cement Concrete."
Sennett, R. B., "Engineering Geology, Environmental Geology and Land Use."

Twenty-Fourth Annual Highway Geology Symposium - August 9-10, 1973 - Wyoming State Highway Department - Sheridan, Wyoming.

Miller, Daniel, "Wyoming Geology" (Abstract only).
Royster, David L., "Highway Landslide Problems Associated with the Escarpment Areas of the Cumberland Plateau in Tennessee."
Wisehart, Richard M., and Wagner, J. Ross, "Slope Stability and Engineering Consequences of a Stream Capture, Bluff Creek, Northern California."
Edwards, Larry J., "Engineering Geologic Map Units for Highway Planning."
Patty, Tom S., "Accelerated Polish Test for Coarse Aggregate."
McKittrick, David P., and Gedney, David S., "Reinforced Earth for Highway Applications."
Bukovansky, Michael, "Engineering Geology and Rock Mechanics Help to Design a Freeway in Northern Spain."
Gilmore, John B., "Landslide Problems on the West Approach to Eisenhower (Straight Creek) Tunnel, Colorado."
Everitt, Martin C., and Holland, T. W., "The Shell Canyon, Wyoming Landslides."
Bauer, Edward, "Application of Geology to Highway Construction in Mountain Terrain, Lovell-Burgess Junction, Wyoming."
Patty, Tom S., "Investigation of Failing Concrete in Houston, Texas, Caused by Unsound Cement."

<u>Twenty-Fifth Annual Highway Geology Symposium</u> - May 23-24, 1974 - N. C. Department of Transportation and Highway Safety, N. C. Department of Natural and Economic Resources, and N. C. State University - Raleigh, North Carolina.

Keene, Kenneth R., "An Evaluation of Sand Drain Installation Methods in Recent Alluvium."
Conrad, Stephen G., "Introduction of the Geology and Mineral Resources of North Carolina."
Royster, David L., "Construction of a Reinforced Earth Fill along I-40 in Tennessee."
Brown, L. F., Jr., and Fisher, W. L., "Environmental Atlas of the Texas Coastal Zone and its Role in Land Use Planning."
Sowers, George F., "Geological and Seismic Engineering for Nuclear Power Plants in the Southeast."
Welby, Charles W., "ERTS and Multispectral Photography."
Goughnour, Roger D., and Mattox, Robert M., "Subsurface Exploration State of the Art."
Sams, Clay E., and Gardner, Charles H., "Engineering Geology of I-26 Landslides, Polk County, North Carolina."
Eades, James L., Dr., "Lime Stabilization." (Paper not published)

<u>Twenty-Sixth Annual Highway Geology Symposium</u> - August 13-15, 1975 - Idaho Transportation Department, Division of Highways - Boise, Idaho.

Bond, John G., "General Geology of Northern Idaho and Western Montana and Its Implication on Highway Construction." (Paper not published)
Mathis, Don H., "Application of Geotechnique to Design and Construction in North Idaho."
Larsen, Ronald E., "Seismic Designed Backslopes and Evaluation in a Structurally Disturbed Basalt Section."
Welch, J. David, "Environmental Geology Survey, Regional Land Use and Intermodal Transportation Planning, Northeastern Pennsylvania."
Steward, John E., "Use of Woven Plastic Filter Cloth as a Replacement for Graded Rock Filters."
Garner, L. E., "Aggregate Resource Conservation in Urban Areas."
Patty, Tom S., "Petrography as Related to Potential Skid Resistance of Paving Aggregates Used on Texas Highway Projects."
Bailey, Allen D., "Rock Types and Seismic Velocities Versus Rippability."
Johnson, Edgar L., "Traffic Safety Instrumentation, Foundation Landslide, Oregon."
Farnham, Paul R., "In-Situ Measurement of Shear - Wave Velocities for Engineering Applications."
Bartholomew, Charles L., and Ireland, Herbert O., "The Use of Organic Topsoils as Construction Materials."
Kumar, S., Annamalai, M., and Laguros, J. G., "A New Analytical Approach to Split Tensile Strength of Pavement Materials."
Thornton, Sam I., and Arulanandan, Kandiah, "Collapsible Soils, State-Of-The-Art."
Weatherby, David E., and Chapman, Ronald K., "Design and Installation of an Earth Tieback Support System."
Denness, Bruce, and Smith, Robert M., "A Landslide in Pleistocene Deposits: Columbia (S. Am.)."
Wilmarth, Verle R., Kaltenbach, John, and Lenoir, William B., "Skylab Explores the Earth."

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."
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 Soft - 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., and 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, Austin, Texas.

Moore, H. L., "Karst Problems along Tennessee Highways: An Overview."
Simpkins, W. W., Gustavson, T. C., Alhades, A. B., and Hoadley, A. D., "Impact of Evaporite Dissolution and Collapse on Cultural Features in the Texas Panhandle and Eastern New Mexico."
Sansom, J. W., Jr., and 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 H., and Sherman, William F., "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."

<u>Thirty-Fifth Annual Highway Geology Symposium</u> - 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."

<u>Thirty-Sixth Annual Highway Geology Symposium</u> - 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., and 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."

<u>Thirty-Eighth Annual Highway Geology Symposium</u> 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., and 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 Geomorphological 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., and 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., and 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., and 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 and 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 and 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 Tieback 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 and Debris Flow and All Other Purposes of Protection."

Mitchell, David A., "4000 Bridge Foundation Investigations."

Capaul, W., Wylie, 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, Case History."

West, Terry R., "Construction of a New Interchange for the Indiana Toll Road, Complicated by Poor Soil Conditions and Presence of Sanitary Land Fill, Gary, Indiana."

Pihl, Roger, and Bowen, Tim, "Design and Construction Methodology for Rock Cuts in Glenwood Canyon."

Miller, Stanley M., "Modeling Shear Strength at Low Normal Stresses for Enhanced Rock Slope Engineering."

Burk, Robert L., and Moser, Kenneth R., "Spirit Lake Memorial Highway -- Geologic Investigations in a Zone of Natural Aesthetic Change."

Levine, Edward N., Genson, Gordon, R., Dye, Ronald R., and Slifer, James C., "From Geophysics to Design in an Environmentally Sensitive Area."

Scott, James H., Burdick, Richard G., and Ludowise, Harry, "Interpretation of Seismic Refraction Data on a Microcomputer."

Thornton, Sam I., and Elliott, Robert P., "Rapid Shear as an Evaluation for Base Course Material."

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-Swell Clays: Prevention and Repair."

Nelson, K. Jeff, and Selvage, John R., "Design Applications of the Welded Wire Wall."

Sharma, Sunil, "Integrated Slope Stability Analysis Using Microcomputers."

Morales, Carlo Hugo Rivera, "Honduras Highway Geology."

Wolosick, John R., "Contract Specification Options for Retaining Wall Design and Construction: Discussion of Construction Alternates and Case Histories."

Sharma, Sunil, and Hardcastle, James H., "Finite Element Analysis of a Rib-Reinforced Steel Culvert."

Thommen, Robert A., Jr., "First Wire Rope Net Rockfall Protective Barrier Installed 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."

<u>Forty-Second Annual Highway Geology Symposium</u> 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" (Abstract only).
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" (Abstract only).
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?"

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. and Wisner, William A., Jr., "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., "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., and Law, Donald S., "Technical Related Analysis, Design, and Construction Four-Lane Highway over 8 to 20 Feet of Peat."

Marienfeld, Mark, "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., and McCahan, Matthew L., "Pennsylvania Turnpike Expansion: a Retrospect."

Ahmed, Imtiaz, and Lovell, C. W., "Laboratory Study on Properties of Rubber - Soils." (Abstract only)

Schneider, Nicholas P., and Bauer, Robert A., "Environmental Property Assessments for Highway Projects: Key Elements for Successful Program Implementation."

Madrid, Larry D., and Smith, Ted J., "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., "Cement Amended Fly Ash as a Structural Fill."

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, North Carolina 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 of 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, Barry F., Dr., Smoot, James L., Dr., 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., and 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., and 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 Permeability."

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."

<u>Forty-Seventh Annual Highway Geology Symposium</u> - September 6-9, 1996 - Wyoming Department of Transportation Geology Program and Wyoming Geological Survey - Cody, Wyoming.

Adams, Wayne, Findley, Dave, and Lowell, Steve, "Peter's Road Landslide."

Adams, William R., Jr., and Ruppen, Christopher A., "Format for Early Collection of Essential Geotechnical Data."

Beck, Timothy J., and Kane, William F., "Current and Potential Uses of Time Domain Reflectometry for Geotechnical Monitoring."

Bhat, S., and Lovell, C.W., "Flowable Fill Using Waste Products."

Boundy, Bret, and Dahill, James, "Instrumentation of a Shredded Tire Fill Used for Landslide Repair, The Burning Issue."

DeNatale, Jay S., and Duffy, John D., "Debris Flow Mitigation."

Duskin, Priscilla, "Seismic Refraction as a Method for Determining Thickness of Organic Sediments."

Fickies, Robert H., "Landslide Hazards in New York State: A Geological Overview."

Gaffney, Donald V., and McCahan, Matthew L., "Integrated Geohazard Management – A Systemwide Approach."

Hall, Kevin D., "Preliminary Investigation Using ALROC Potliner Sand in Asphalt Concrete Mixtures."

Heinert, Kevin, "Permanent and Temporary Earth Anchor Systems."

Lundvall, J.F., Turner, J.P., Stewart, J.M., and Edgar, T.V., "Mitigation of Roadway Settlement Above Buried Culverts and Pipes."

Moore, Harry, "The Use of Underbenching in Embankment Construction through Mountainous Terrain – I-26, Unicoi County, Tennessee."

Priznar, Nick M., and Euge, Kenneth M., "Engineering Geologic Investigation of the Gordon Canyon Slide."

Studebaker, Irving G., Patton, Susan B., and Studebaker, Raymond G., "Subsidence Definition and Effects on Surface Construction."

Turner, John P., Hasenkamp, R.N., and Dahill, James, "Reinforced Earth Retaining Wall for Landslide Control, Snake River Canyon, Wyoming."

West, Terry R., and Park, Hyuck Jin, "Rock Durability of Argillaceous Carbonate Rocks in Cut Slopes for Indiana Highways."

Willems, Henry T., and Podniesinski, Matthew J., "Rock Slope Stabilization at Dunderberg Mountain, Rockland County, New York"

Wyllie, Duncan C. and Kielhorn, W., "Wedge Stability Analysis – Geometric and Groundwater Enhancements."

Ludwig, Claus, "Making a Soil Nail Wall Look Like Rock."

Burns, Scott, "Earthflows, Debris Flows and Roads: The Aftermath of the Winter Storm of 1996, Portland, Oregon"

APPENDIX B

HGS Proceedings Availability List

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS
- Availability List -

Symposium/Date	Sponsor	Availability	Price
First (1950)			
Second (1951)			
Third (1952)		The First 10 Symposia "Proceedings" are contained in a 3 volume set.	
Fourth (1953)			
Fifth (1954)			
Sixth (1955)		A limited number of these volumes are available at $20.00 a set.	
Seventh (1956)			
Eighth (1957)			
Ninth (1958)			
Tenth (1959)			
Eleventh (1960)	Florida State Univ.	Out of Print	
Twelfth (1961)	Univ. of Tennessee	Out of Print	
Thirteenth (1962)	Phoenix, Arizona	Out of Print	
Fourteenth (1963)	Texas Highway Dept.	Out of Print	
Fifteenth (1964)	Missouri Geol. Survey	Out of Print	
Sixteenth (1965)	Univ. of Kentucky	Out of Print	
Seventeenth (1966)	Iowa State Univ. and Iowa State Highway Commission	F. R. Glass*	$6.00
Eighteenth (1967)	Purdue University and Indiana Highway Commission	Professor Terry West Civil Engineering Dept. West LaFayette, Indiana	

Symposium/Date	Sponsor	Availability	Price
Nineteenth (1968)	West Virginia Geol. & Economic Survey	W. Virginia Geological and Economic Survey P. O. Box 879 Morganton, W. Va. 26505 (Circular 10)	$20.00
Twentieth (1969)	Univ. of Illinois	Out of Print	
Twenty-First (1970)	State Highway Commission of Kansas	Out of Print	
Twenty-Second (1971)	Oklahoma Geol. Survey and Oklahoma Dept. of Highways	Oklahoma Geological Survey University of Oklahoma Norman, Oklahoma	
Twenty-Third (1972)	Virginia Hwy. Research Council and Virginia Dept. of Highways	Highway Geologist Virginia Dept. of Highways Richmond, Virginia	
Twenty-Fourth (1973)	Wyoming State Highway Department	Out of Print	
Twenty-Fifth (1974)	North Carolina Dept. of Transportation	Out of Print	
Twenty-Sixth (1975)	Idaho Transportation Department	R. G. Charboneau Chief Geologist Idaho Dept. of Highways P. O. Box 7129 Boise, Idaho 83707	
Twenty-Seventh (1976)	Florida Dept. of Transportation and Univ. of Florida	F. R. Glass*	$20.00
Twenty-Eighth (1977)	South Dakota School of Mines	Out of Print	
Twenty-Ninth (1978)	Maryland State Hwy. Administration and Maryland Geological Survey	Publications Dept. Maryland Geol. Survey 711 W. 40th Street Suite 440 Baltimore, Maryland 21314	$20.00
Thirtieth (1979)	FHWA at Portland, Oregon	F. R. Glass*	$20.00
Thirty-First (1980)	Bureau of Economic Geology and Texas Dept. of Highways	F. R. Glass*	$20.00

Symposium/Date	Sponsor	Availability	Price
Thirty-Second (1981)	Tennessee Dept. of Transportation	F. R. Glass*	$20.00
Thirty-Third (1982)	Colorado Dept. of Highways, Colorado Geol. Survey and USDA Forest Service	Colorado Geol. Surv. Dept. of Nat. Resources 1313 Sherman Street Denver, Colorado 80203	$20.00
Thirty-Fourth (1983)	Georgia Dept. of Transportation	F. R. Glass*	$20.00
Thirty-Fifth (1984)	California Dept. of Transportation and San Jose State Univ.	F. R. Glass*	$20.00
Thirty-Sixth (1985)	Kentucky Dept. of Transportation and School of Civil Eng., Purdue University	F. R. Glass*	$20.00
Thirty-Seventh (1986)	Montana Dept. of Highways and Montana Div. - FHWA	F. R. Glass*	$20.00
Thirty-Eighth (1987)	Pennsylvania Dept. of Transportation and Engineer's Society of Western Pennsylvania	F. R. Glass*	$20.00
Thirty-Ninth (1988)	Utah Dept. of Trans., Utah Geological & Mineral Survey and Brigham Young University	F. R. Glass*	$20.00
Fortieth (1989)	Alabama Highway Department	F. R. Glass*	$20.00
Forty-First (1990)	New Mexico State Hwy. & Transportation Dept. and New Mexico State Univ. Department of Civil, Agricultural & Geological Engineering	F. R. Glass*	$20.00
Forty-Second (1991)	American Society of Civil Engineers, American Inst. of Professional Geologists, NYSDOT, and New York State Geol. Survey	F. R. Glass*	$20.00

Symposium/Date	Sponsor	Availability	Price
Forty-Third (1992)	Univ. of Arkansas Dept. of Civil Engineering, Arkansas State Highway & Transportation Dept., and Arkansas Geol. Commission	F. R. Glass*	$20.00
Forty-Fourth (1993)	Univ. of South Florida Dept. of Civil Engineering and Florida Dept. of Transportation	F. R. Glass*	$20.00
Forty-Fifth (1994)	Portland State Univ., Oregon Dept. of Transportation, Washington Dept. of Transportation and Transportation Research Board	F. R. Glass*	$20.00
Forty-Sixth (1995)	West Virginia Geological and Economic Survey, West Virginia Div. of Highways, West Virginia University and Transportation Research Board	F. R. Glass*	$20.00
Forty-Seventh (1996)	Wyoming Dept. of Transportation-Geology Program and Wyoming Geological Survey	F. R. Glass*	$20.00

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