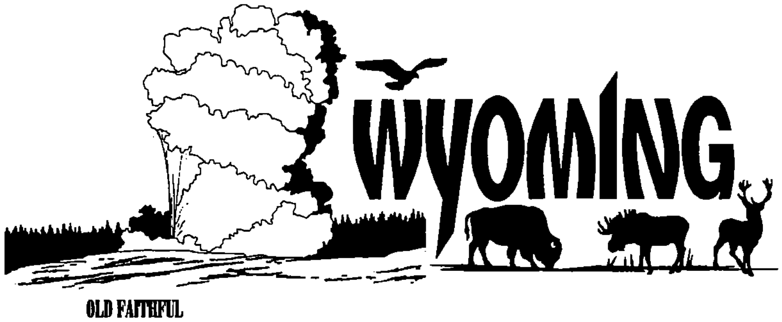
47th Highway Geology Symposium

Cody, Wyoming
September 6-9, 1996

PROCEEDINGS

Ted Vredenburg and Mike Schulte, Proceedings Coordinators

Sponsored by:
Wyoming Department of Transportation
Wyoming Geological Survey



47th Highway Geology Symposium

Cody, Wyoming
September 6-9, 1996
AGENDA

Friday, September 6

7:00 p.m. to 10:00 p.m. Welcoming Ice Breaker, Bandana Room
Compliments of Geo-Recovery Systems, Inc.

9:00 p.m. to 10:00 p.m. Registration, Breezeway

Saturday, September 7

7:00 a.m. to 5:00 p.m. Registration, Breezeway

8:00 a.m. to 6:00 p.m. Exhibitor Display, Taggart's

8:00 a.m. to 8:35 a.m. **All Technical Sessions will be held in the Holiday Inn's Meeting Room**

Welcoming Remarks: HGS Steering Committee, WYDOT, Wyoming Geological Survey

Keynote Address: *"Geology and Minerals of Wyoming"*, Gary B. Glass, Wyoming State Geologist

Technical Session I

Landslides/Stability
Session Moderator: James Dahill, WYDOT

8:35 a.m. to 9:00 am *"Earthflows, Debris Flows and Roads: The Aftermath of the Winter Storm of 1996, Portland Oregon"*, Scott Burns, Portland State University

9:00 a.m. to 9:25 a.m. *"Landslide Hazards in New York State: A Geological Overview"*, Robert Fickies, New York State Geological Survey

9:25 a.m. to 9:50 a.m. *"Peters Road Landslide"*, Wayne Adams and Dave Findley, Golder Associates; Steve Lowell, WASHDOT

9:50 a.m. to 10:20 a.m. **Coffee Break, Taggart's,** *Compliments of Central Mine Equipment*

10:20 a.m. to 10:45 a.m. *"Rock Slope Stabilization at Dunderberg Mt., Rockland County, New York"*, Henry Willems and Matthew Podniesinski, NY State DOT

10:45 a.m. to 11:10 a.m. *"Geologic Investigation of the Gordon Canyon Slide"*, Nick Priznar, ADOT; Kenneth Euge, Geological Consultants

11:10 a.m. to 11:35 a.m. *"Reinforced Earth Retaining Wall for Landslide Control, Snake River Canyon, Wyoming"*, J. P. Turner, R. N. Hasenkamp, and T. V. Edgar, Univeristy of Wyoming; James Dahill, WYDOT

11:35 a.m. to 12:00 p.m. *"Wedge Stability Analysis - Geometric and Groundwater Enhancements"*, Duncan Wyllie and W. Kielhorn, Golder Associates Ltd.

12:00 p.m. to 1:10 p.m. **Lunch, on your own**

Technical Session II

Geophysical & Geotechnical
Session Moderator: Harry Moore, Tennessee DOT

1:10 p.m. to 1:35 p.m. *"Seismic Refraction as a Method for Determining Thickness of Organic Sediments"*, Priscilla Duskin, New York State DOT

1:35 p.m. to 2:00 p.m. *"Current and Potential Uses of Time Domain Reflectometry for Geotechnical Monitoring"*, Timothy J. Beck, California DOT; William F. Kane, University of the Pacific/Neil O. Anderson and Associates, Stockton, California

2:00 p.m. to 2:25 p.m. *"Instrumentation of a Shredded Tire Fill Used for a Landslide Repair, The Burning Issue"*, Bret Boundy and James Dahill, WYDOT

2:25 p.m. to 2:50 p.m. *"Integrated Geohazard Management- A Systemwide Approach"*, Donald V. Gaffney, Michael Baker Jr., Inc.; Matthew L. McCahan, Pennsylvania Turnpike Commission

2:50 p.m. to 3:15 p.m. *"SGH Form: Format for Early Collection of Essential Geotechnical Data"*, William R. Adams, Jr., Pennsylvania DOT; Christopher Ruppen, Michael Baker Jr., Inc.

3:15 p.m. to 3:45 p.m. **Refreshment Break, Breezeway,** *Compliments of all Exhibitors*

Technical Session III

Retaining and/or Soil Anchoring Systems
Session Moderator: Mark Falk, WYDOT

3:45 p.m. to 4:10 p.m.	"*Permanent and Temporary Earth Anchoring for Highway Applications*", Kevin Heinert, Williams Form Engineering Corp.
4:10 p.m. to 4:35 p.m.	*"Making a Soil Nail Wall Look Like Rock"*, Claus Ludwig, Schnabel Foundation Company
4:35 p.m.	*"Things to Come",* Slide Show Presentation, WYDOT

Evening Events

5:00 p.m.	**Adjournment-Dinner, on your own**
7:00 p.m. to 10:00 p.m.	**Mid-Meeting Mingler, Bandana Room** *Compliments of Michael Baker Jr., Inc.*

Sunday, September 8

Field Trip to Sunlight Basin

7:30 a.m.	**Leave from the Holiday Inn**
	Morning Break Compliments of Tensar Earth Technologies and Contech Construction Products *Lunch Compliments of Brugg Cable Products, Inc.* *Afternoon Break Compliments of Golder Associates*
5:00 p.m.	**Return to Holiday Inn**

Evening Events

6:30 p.m.	**Banquet Social Hour Begins, Meeting Room** *Compliments of Nicholson Construction Company*
7:30 p.m.	**Banquet, Meeting Room**

8:30 p.m.	**Dinner Speaker:** *"A Dam is Built"*, A History of the Buffalo Bill Dam and Shoshone Reclamation Project; Beryl Churchill, Park County Commissioner

Monday, September 9

Technical Session IV

Construction
Session Moderator: Vernon Bump, South Dakota DOT

8:00 a.m. to 8:25 a.m.	*"The Use of Underbenching in Embankment Construction Through Mountainous Terrain, I-26, Unicoi County, Tennessee"*, Harry Moore, Tennessee DOT
8:25 a.m. to 8:50 a.m.	*"Rock Durability of Argillaceous Carbonate Rocks in Cut Slopes for Indiana Highways"*, Terry West and Hyuck Jin Park, Purdue University
8:50 a.m. to 9:15 a.m.	"*Flowable Fill Using Waste Products*", C. W. Lovell, Purdue University
9:15 a.m. to 9:45 a.m.	**Coffee Break, Taggart's,** *Compliments of Michael Baker Jr., Inc.*
9:45 a.m. to 10:10 a.m.	*"Subsidence Definition and Effects on Surface Construction"*, Irving Studebaker and Susan Patton, Montana Tech Mining Dept.; Raymond Studebaker, Montana DOT
10:10 a.m. to 10:35 a.m.	*"Roadway Settlement Over Culverts: Causes and Cures"*, J. F. Lundvall, J. P. Turner, J. M. Stewart and T.V. Edgar, University of Wyoming
10:35 a.m. to 11:00 a.m.	*"Debris Flow Mitigation Using Flexible Barriers"*, John Duffy, California DOT; Jay DeNatale, California Polytechnic State University
11:00 a.m. to 11:25 a.m.	"*Preliminary Investigations Using ALROC Potliner Sand in Asphalt Concrete Mixtures*", Kevin Hall, University of Arkansas
Noon:	**Official Closure of the 47th Annual HGS Conference**

TABLE OF CONTENTS

PROCEEDINGS PAPERS

APPENDICES

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 relatively relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two to 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 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 principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and Glenwood Canyon; while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area, which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures in the greater Louisville area. The 1986 field trip was through the Rockies on recent Interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

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

HGS STEERING COMMITTEE OFFICERS

Mr. Earl Wright, Chair — 1997
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Geotechnical Branch, Division of Materials
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PH: 813/974-5845

Mr. Russell Glass, Treasurer — Appointed by Chairman
Area Geologist, Geotechnical Unit
North Carolina Department of Transportation
P. O. Box 3279
Asheville, NC 28802
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NOTE: Officers' terms expire at conclusion of 1997 Symposium.

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Mr. Vernon Bump Division of Engineering Department of Transportation Pierre, SD 57501 PH: 605/773-3401	1996
Mr. Richard Cross New York State Thruway Authority 200 Southern Boulevard P. O. Box 189 Albany, NY 12201-0189 PH: 518/471-4277	1997
Mr. Jeff Dean Oklahoma Department of Transportation	1998
Mr. John B. Gilmore Colorado Highway Department 4340 East Louisiana Denver, CO 80222 PH: 303/757-9275	1997
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Mr. G. Michael Hager Wyoming Department of Transportation P. O. Box 1708 Cheyenne, WY 82003-1708 PH: 307/777-4475	1996
Mr. Robert W. Henthorne Kansas Department of Transportation Box 498 411 W. 14th Street Chanute, KS 66720-0498 PH: 316/431-1000 Ext. 47	1996
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Mr. Charles T. Janik Pennsylvania Department of Transportation 1118 State Street Harrisburg, PA 17120 PH: 717/787-5405	1997
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Mr. Harry Ludowise 5308 NE 12th Avenue Vancouver, WA 98665 PH: 206/693-1617	1998

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Mr. Earl Wright Geotechnical Branch Kentucky Department of Highways Frankfort, KY 40622 PH: 502/5674-2374	1996

EMERITUS MEMBERS OF THE STEERING COMMITTEE FOR THE HIGHWAY GEOLOGY SYMPOSIUM

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David Bingham

Virgil E. Burgat

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David L Royster *

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Berke Thompson

Burrell Whitlow *

Ed J. Zeigler

Status granted by Steering Committee

* deceased

MEDALLION AWARD WINNERS

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R. J. Edmonson	1972
C. S. Mullin	1974
A. C. Dodson *	1975
Burrell Whitlow *	1978
Bill Sherman	1980
Virgil Burgat	1981
Henry Mathis	1982
David Royster *	1982
Terry West	1983
Dave Bingham	1984
Vernon Bump	1986
C. W. "Bill" Lovell	1989
Joseph A. Gutierrez	1990
Willard McCasland	1990
W. A. "Bill" Wisner	1991
David Mitchell	1993

In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana, designed a plaque to be presented to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallian mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

* deceased

FUTURE HIGHWAY GEOLOGY SYMPOSIUM SCHEDULE

TIME & LOCATION	COORDINATORS
1. 1997 Meeting: 48th May, 1997 Tennessee	Harry Moore Tennessee Dept. of Transportation
2. 1998 Meeting: 49th Arizona	Mr. G. Bruce Kay Geotechnical Services Engineer 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
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	West (Open)
9. 2005 Meeting: 56th Vermont	Alan McBean Vermont DOT

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Santa Fe, NM 87505

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Michael Baker Jr., Inc.
4301 Dutch Ridge Road
Beaver, PA 15009

Saturday Social Hour and
Monday Morning Break

Central Mine Equipment
4215 Rider Trail North
Earth City, MO 63045

Saturday Morning Break

Geo-Recovery Systems, Inc.
400 Corporate Circle, Suite B
Golden, CO 80401

Friday Night Ice-Breaker

Golder Associates
3730 Chamblee Tucker Road
Atlanta, GA 30341

Field Trip Beer Break

Nicholson Construction Company
P.O. Box 98
Bridgeville, PA 15017

Sunday Social Hour

Tensar Earth Technologies
8703 Yates Drive, Suite 110
Westminster CO 80030

Contech Construction Products
4891 Independence St. #195
Wheat Ridge, CO 80033

Field Trip Morning Break

Thanks to All Participating Exhibitors!

A. B. Chance Company	Centralia, Missouri
Brugg Cable Products, Inc.	Santa Fe, New Mexico
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Elastizell Corporation of America	Ann Arbor, Michigan
Geokon, Inc.	Lebanon, New Hampshire
Geo-Recovery Systems, Inc.	Golden, Colorado
Golder Associates, Inc.	Lakewood, Colorado
Hilfiker Retaining Walls	Eureka, California
Maccaferri Gabions, Inc.	Puyallup, Washington
Michael Baker Jr., Inc.	Beaver, Pennsylvania
Monotube Pile Corporation	Canton, Ohio
Neil O. Anderson & Associates	Lodi, California
Nicholson Construction Company	Bridgeville, Pennsylvania
Penndrill Manufacturing	McKees Rocks, Pennsylvania
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RST Instruments	Coquitlam, B.C., Canada
Schnabel Foundation Company	Denver, Colorado
Slope Indicator Company	Bothell, Washington
Williams Form Engineering Corporation	Grand Rapids, Michigan
Wyoming Geological Survey	Laramie, Wyoming

In Recognition

With deep appreciation for their tireless and unselfish efforts in arranging, organizing, compiling, disseminating, typing, calling, faxing, errand-running, driving, traffic-controlling, and generally working long hours to ensure that the 47th Annual Highway Geology Symposium would be a success, the following members of the WYDOT Geology Program are hereby recognized:

Kathy Ahlenius
Bret Boundy
Jim Coffin
Dave Collins
Jerry Cooke
Doug Crabtree
James Dahill
Den Dudrey
Shawn Enright
Mark Falk
Marilyn Foster
Connie Fournier
Ron Kaiser
Mike Kent
Ed Miller
Greg Miller
Mike Schulte
Blaine Smith
Ted Vredenburg
Craig Walker
Alan Wood

I would also like to thank Gary Glass of the Wyoming Geological Survey and his staff for their geological contributions on the field trip and the production of the road log.

Thank You!!!

G. Michael Hager, Chairman
47th Annual Highway Geology Symposium

EARTHFLOWS, DEBRIS FLOWS, AND ROADS: THE AFTERMATH OF THE WINTER STORM OF 1996, PORTLAND, OREGON

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ABSTRACT

In the middle of a winter with high rainfall, a rain on snow climatic event caused major flooding in the Pacific Northwest with the Portland area getting particularly hit hard during the second week of February, 1996. Associated with the flooding was abundant landslide activity which crippled the region. In the Portland metropolitan region 40% of the total $10 million of damage resulted from landslides. The majority of the failures were earthflows and slump/earthflows in the loess deposits of the West Hills of Portland. Debris flows were abundant in the steep drainages with bedrock streams such as along the Columbia River and in the Coast Range. Major highways were closed for over a week. Interstate 84 east of Portland at Dodson was closed by the largest debris flow in the region that had over six major pulses. Highway 6 west of Portland in the Coast Range was closed for two months from over 20 slumps in the Yamhill Formation and debris flows in the Tillamook basalts. Repair of most of the failures on Highway 6 was accomplished using light weight fill of wood chips. Repair of the loess failures along roadways is mainly by rockfill and geotextiles.

INTRODUCTION

During any winter in the area around Portland, Oregon maybe one major road might be closed for a few hours by a landslide. This past February in 1996 a rare climatic event brought abundant water to the Oregon slopes causing major flooding and many landslides. At one point in the second week of February, every major highway in and out of Portland was closed by a landslide. This fluke storm created so much landslide activity that many people mentioned that this may have been a 100 year event for landslides.

An inventory of the landslide failures is being undertaken at this time so no solid numbers have been accumulated. This paper will focus more qualitatively on the processes that created the slides, how they were and are being mitigated, and how some might have been prevented. I will also show a few photos of some of the examples.

CLIMATIC EVENT

The climatic event that happened the second week of February is called a rain on snow event. Four factors led up to this incredible event (George Taylor, State Climatologist, personal communication). First, the winter had been wet and the soils were saturated, i.e. rainfall was twice the normal. Secondly, two weeks before had had a very large snowfall that changed the winter snowpack from 27% of normal to 120% of normal. Then, after a week of freezing temperature, the "Pineapple Express" hit Oregon. In the four days from February 5 to the 8th, a warm tropical storm brought abundant rainfall to the area. Up to 15 cm fell in Portland in those four days. In the mountains, the rain melted this abundant snowfall and all of that water went into the streams. The streams could not hold all of its water and floods happened. The landslides were mostly caused by the increased rainfall. At the Saddle Mountain weather station in the Coast Range, 55 cm of rainfall combined with 35 cm of melted snow to produce 90 cm of water going into the already saturated soils.

LANDSLIDES: EXAMPLES, PROCESSES, AND MITIGATION

A) Earthflows in the loess:

On the hills surrounding Portland, there is abundant loess, up to 25 meters thick. This region of Portland is notorious for earthflows and slump-earthflows during the winter. They usually form in roadcuts where water has been concentrated on the slope by drainage problems from above like broken drainpipes. Our students mapped in excess of 200 of such failures alone in Portland. Figure 1 shows an earthflow that resulted when water was concentrated on the slope from runoff from a driveway of a home. It closed the street below for two days. Portland became the city of plastic in two days with everyone covering their new slides. That earthflow was repaired by removing the failure debris, covering the hole with a geotextile, and then filling the hole with interlocking basalt boulders in a matrix of crushed gravels.

Figure 2 depicts another earthflow that happened between two homes. The earthflow was greatly enhanced by runoff from the unimproved road above the slide. The only place for water to leave that small street was over the slope face, thereby saturating the loess. The earthflow covered the street below for four days. The slide has not been repaired yet.

Figure 3 shows another earthflow in another neighborhood. This failure occurred on a paleosol within the loess deposit of another older loess unit. Many such failures were encountered in the West Hills. Highway officials have noticed that if they see a red zone daylighting in the side of the roadcut that they are about to have problems in the rainy weather. This slide was repaired by removing all of the colluvium, putting in a geotextile base, filling with rockfill, and then covering the site with loess soil and barkdust. They also put a retaining wall in at the bottom of the slope.

Figure 1: Earthflow in cutslope in loess. Uncontrolled runoff from driveway most likely caused this failure.

Figure 2: Earthflow in loess. Runoff from street above concentrated water on the slope and caused the failure.

Figure 3: Earthflow in loess where failure surface is on a paleosol in the loess.

Figure 4 also shows a small earthflow that occurred on Canyon Road, a major road from the west side of Portland. The actual width of the failure was only 30meters, but the repair was about 180 meters wide. In the middle of the slide was a very important power pole which needed to be stable. Figure 5 shows the repair of the slope again by rockfill on top of a geotextile with good drains. They will have problems in the future because the dark areas are springs.

The failures in the loess were common in this event, but they were not the big failures that were found other places. They closed many small streets and a couple of large ones.

B) Debris flows:

Debris flows were common where there were bedrock streams on a steep gradient. The areas of the Coast Range, just west of Portland and the areas next to the Columbia Gorge were prime examples. Heavy rains would cause landslides to occur in the upper drainages of a watershed, and the landslide debris would eventually become mobilized and flow in the stream system. Stream bottoms were not places to have roads for many of the roads were wiped out by flooding waters and debris flows.

The most famous of the debris flows happened just east of Portland at the town of Dodson. The town sits on an old alluvial fan which tells us that these debris flows have been active there in the late Quaternary (Figure 6). In Dodson, the first debris flow moved at about 10 km/hr. The inhabitants of the farmhouse that is shown in Figure 7 watched the debris flow come out of the canyon uphill on the property. They were able to run away from it so the velocity was probably about 8 km/hr. The house was filled up with debris, but the owners are trying to move back in. On the eastern side of the fan the debris flows moved at over 30 km./hr. The inhabitants said that there were over 6 pulses of sediment. This debris flow shut down all of the train and motor vehicle traffic along I-84 which was at its base. Repair has been putting in bigger culverts and diversion canals at the top of the fan.

Another set of debris flows occurred to the west of Portland on Highway 6 to the beach. Just to the west of the summit, there were over 10 debris flows that had come down the slope and had washed out the fill under the road. Most of these failures were fixed by putting in lots of light-weight fill , wood chips (Figure 8). The highway was closed for over three months.

One debris flow occurred just outside of Portland on Germantown Road. A landslide in the upper drainage came to rest in the bottom of the drainage. The heavy rainfall mobilized the colluvium and turned it into a debris flow which wiped out a house built in the valley bottom (Figure 9). The house was knocked off of its foundation and crushed the owner's car which was parked in front of the house. As most homeowners in

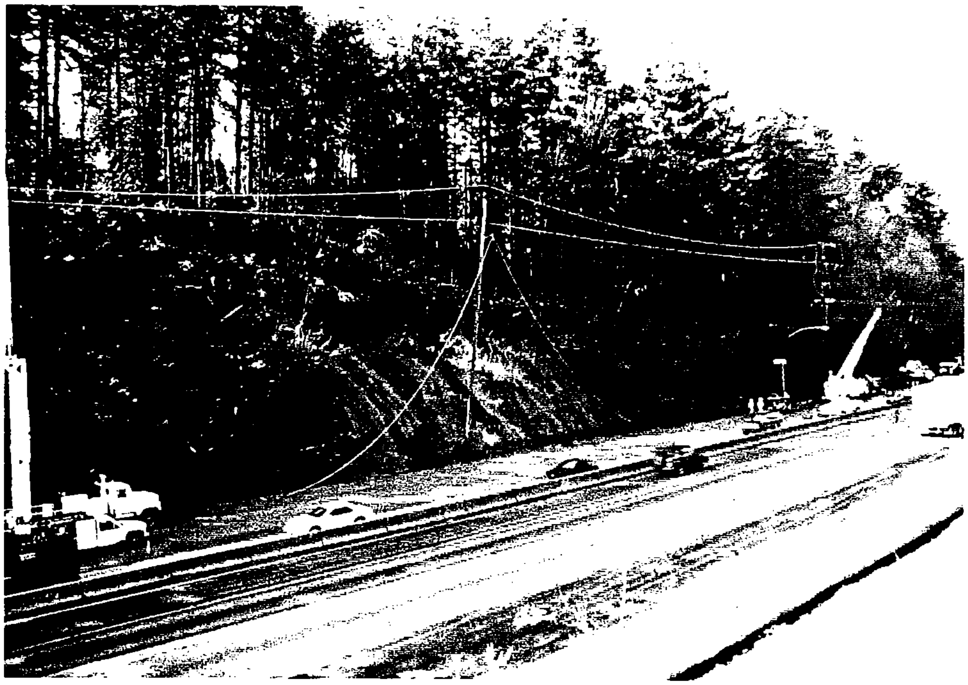

Figure 4: Earthflow in loess on major highway, Canyon Road. Note the size of the failure around the telephone pole.

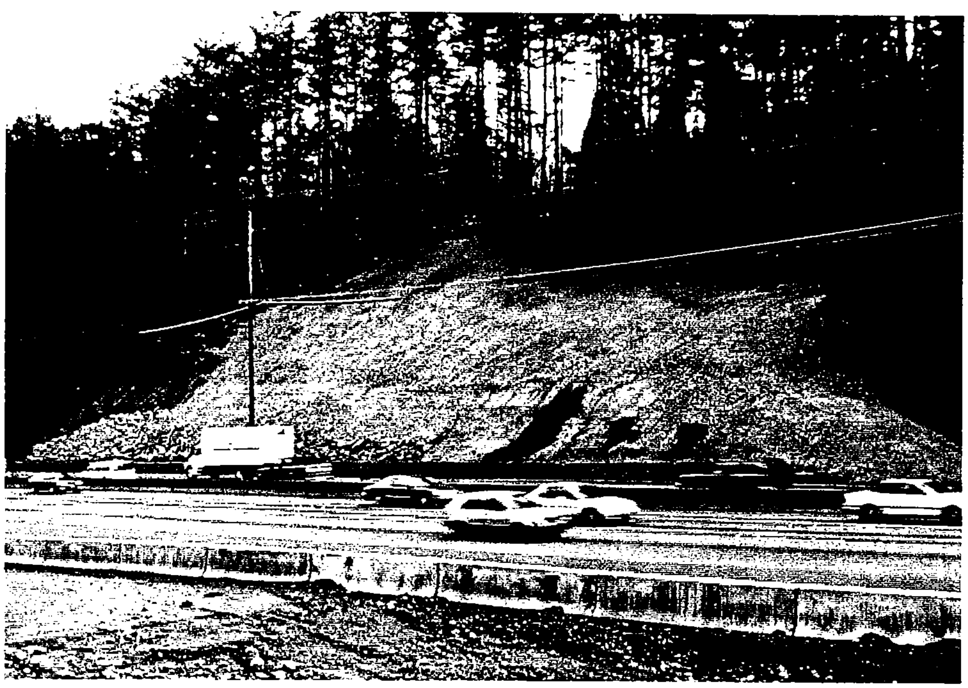

Figure 5: Repair of Earthflow in Figure 4. Note the same telephone pole. The size of the width of the repair is many times the size of the original failure.

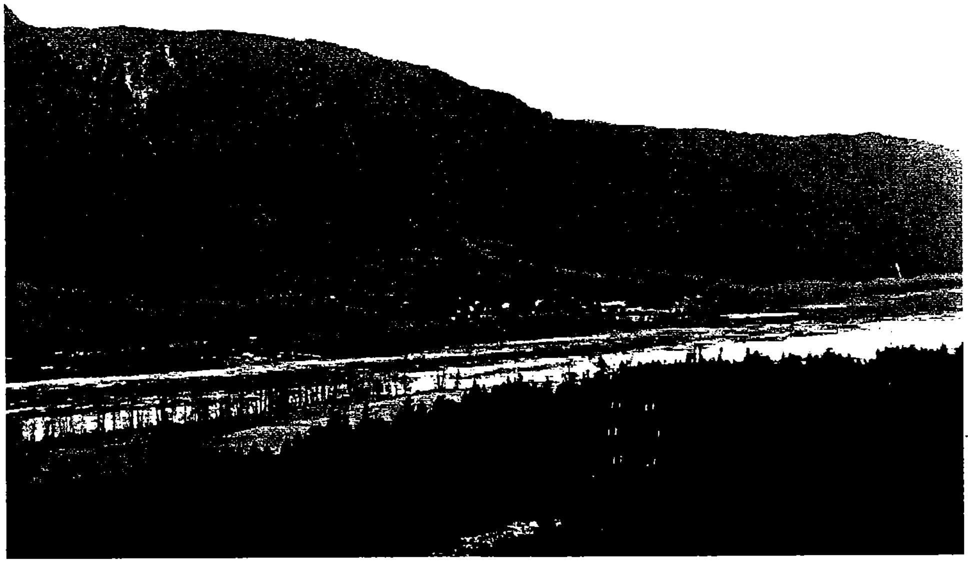

Figure 6: Alluvial fan of the Dodson debris flow. Age of the fan is 12,800 years old.

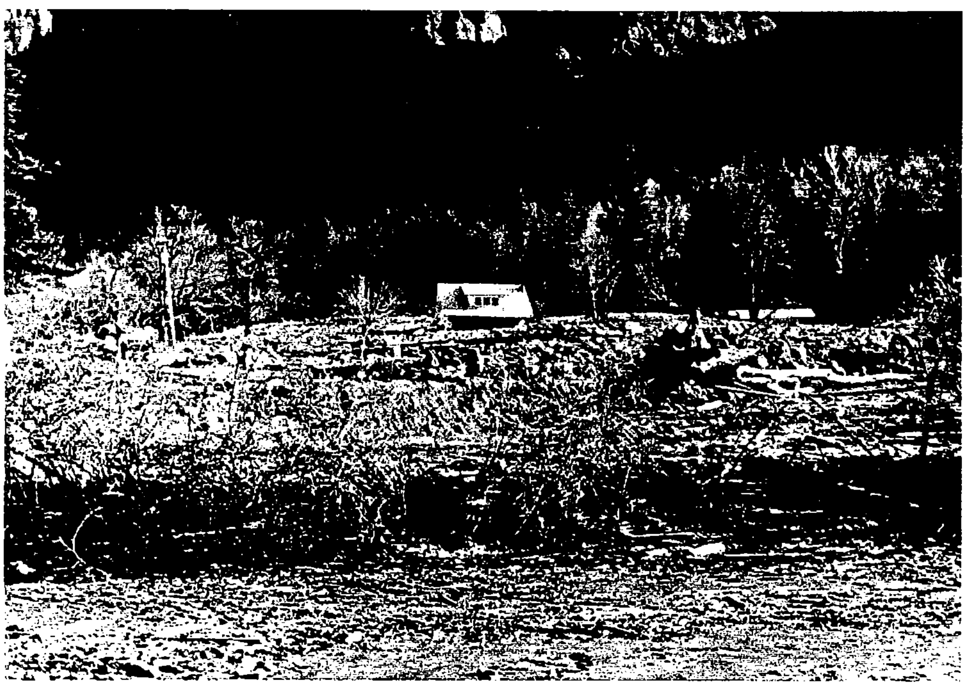

Figure 7: Debris flow at Dodson. It buried this farmhouse to 3 meters deep.

Figure 8: Use of wood chips for light-weight fill on Highway 6 failure.

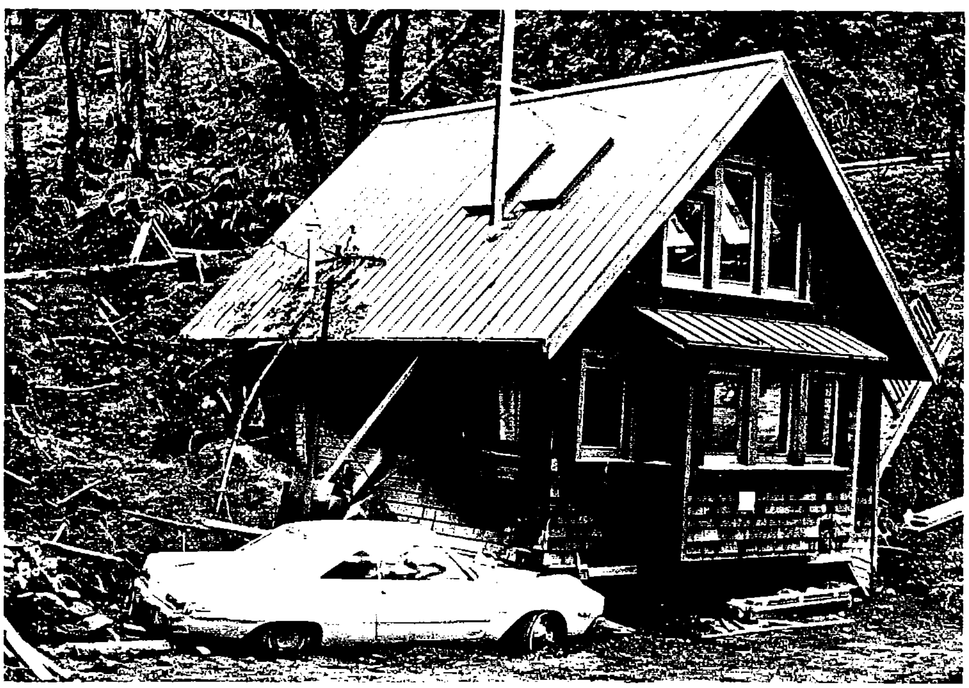

Figure 9: Debris flow knocks house off of its foundation.

the area with landslide damage, they found that homeowner's insurance does not cover any landslide damage. FEMA won't even give you a low interest low.

c) Landslides in weak rock

Just to the west of Portland there are large deposits of clay rich rocks. One famous formation is the marine clay called the Keasy Formation. It had so many failures on it, that the most of the roads along the river which follows the outcrop of this north-plunging anticline closed. Most of these failures were repaired by putting in light weight aggregate of wood chips.

d) Translational Slides

In the Cascades where interflow zones are found in the bedrock , one finds abundant paleosols and clays. The failures of this type are mainly translational slides. There were some of these in Portland, but most were found outside of the territory.

HOW NOT TO REPAIR OF FAILURE

I got a chance to visit the small airport in the town of Estacada. There had been a large fill failure on the road going to the airport. They needed to have it repaired, but did not want to spend money. I arrived on the site and dump truck after dump truck was arriving and dumping their wet soil over the bank and into the upper scarp of the landslide. A bulldozer was then moving it around. The owner had asked all dump trucks to bring all landslide debris from the other landslides of the region to fill in his "hole" in the slope, not realizing that most of the soils were above the liquid limit. He was essentially loading the upper part of the slide and it took off! The owner did not want to pay a professional and eventually had to pay the consequences.

CONCLUSIONS

An overall conclusion drawn so far from the data is that we missed the large, gigantic landslides because most were of the small to medium size. When do the large landslides occur? Maybe they form mainly after we have had a large subduction zone earthquake.

The major type of failures that occurred were earthflows and slumps in the fine-grained soils, debris flows in the steep bedrock drainages, and failures in weak rocks when they get wet.

Landslide Hazards in New York State: a Geological Overview

by

Robert H. Fickies, C.G.
New York State Geological Survey

Abstract

The present day landscape of New York State is the product of three major geologic processes:

- long term; bedrock development, modification, and erosion, measured in hundreds of millions of years,
- short term; complex terrain deposition and modification by glacial action, measured in hundreds of thousands of years,
- post-glacial modification and erosion of geologic materials; measured in tens of thousands of years.

As a result of the combined effect of these processes, a significant portion of New York's landscape has been left in a marginally stable to unstable state. In these areas, landslides and other slope stability problems pose a threat to roadways, buildings and people.

The various types of landslides that occur in New York can be categorized based on type of material, method of failure, and rate of failure. This method of classifying slope failures was first described by Varnes[1] in 1978. A modification of this classification was used by Fickies and Brabb[2] in 1987 to inventory landslides statewide in New York. This latter study determined that there had been approximately 80 landslide related deaths in New York since 1836, and that present day, economic losses related to slope failures amounting to at least ten million dollars annually.

For purpose of the present study, New York State is best defined as being made up of three major upland regions (The Adirondack Mountains, the Allegheny-Catskill Plateau, and the smaller New England uplands), surrounded by several, distinct lowlands (Atlantic Coastal Plain, Hudson-Mohawk, St. Lawrence-Champlain, Ontario, and Erie Lowlands). Different types of landslides predominate in upland and lowland regions.

Surficial deposits in upland areas are, for the most part, shallow to thick till over bedrock, with alluvial and ice-contact deposits occupying numerous incised valleys. Upland landslide types include rock falls, rock topples, debris slides, debris avalanches, and earth slumps.

Lowland regions, occupied by fresh water lakes or marine waters at the end of the Pleistocene era, are the loci of slump-earth flow, earth slump, and earth block slides.

While certain types of landslides predominate within the upland or lowland geomorphic regions, occurrence of a particular failure mode is not restricted to one region. This is particularly true within the uplands, where numerous incised valleys were occupied by glacier-dammed lakes. The resultant deposits may be considered as a localized lowland setting within the upland. The Tulley Valley of central New York is such an example.

1. Varnes, D. J., 1978, *Slope Movement Types and Processes*, in Schuster, R. L. and R. J. Krizek, eds. **Landslides, Analysis and Control.** Trans. Res. Bd. Spec. Rpt., Nat. Acad., of Sci. #176, pgs. 11-35.

2. Fickies, R. H., and E. E. Brabb, 1989, *Landslide Inventory Map of New York,* NYS Mus. Circ. #52, 1 map, 1:500,000 scale.

Peters Road Landslide

By

Wayne Adams[1], Dave Findley[2], and Steve Lowell[3]

Abstract

The purpose of this paper is to describe the occurrence, investigation and mitigative measures at the site of an ancient landslide (Peters Road Landslide). This landslide reactivated during heavy rains in November 1994 causing a debris flow that covered State Route 12 (SR 12), about one mile west of Randle, Washington, with a thickness of up to 40 feet of rock and mud. The initial 1994 slope failure occurred as a debris/colluvium slide along a NNE striking fault structure that resulted in a 1800 feet long, 200 feet wide debris slide track. SR 12 is located along the southern (downslope) end of this debris slide track. Several conditions ultimately lead to a massive bearing-type failure of a rock mass contained by two undetermined structural features on January 1996. The western bedrock escarpment delivered at least 200,000 yd^3 to the slide. The conditions causing mobilization included debris/colluvium movement, intense rainfall and rock degradation. It was determined by the project team that the most effective mitigative measures were to move SR 12 farther away from the toe of the slope and include a catchment area in the highway design.

Introduction

In January 1995, Golder Associates (Golder), under a General Services Agreement, was requested by the Washington State Department of Transportation (WSDOT) to conduct a geotechnical investigation of the colluvium/debris slide that had covered State Route 12 with an approximately 40 feet thick section of debris on November 22, 1994. The purpose of the investigation was to gain an understanding of the landslide geometry and mechanism and formulate potential remedial options to mitigate the debris slide. A geologic model was developed based on subsurface drill information and geophysical surveys, supplemented with aerial and surficial reconnaissance. Stability analyses were based on this model and various remedial options were presented to WSDOT. These remedial options centered on maintaining the existing alignment of SR 12 and construction of an earthen berm structure for rockfall catchment located uplsope of the highway in the toe area of the 1994 landslide.

During construction of the catchment berm in the fall of 1995, tension cracks began to develop above and behind the 200 feet high western bedrock lateral scarp of the 1994 landslide track. The tension cracks continued to grow and widen during the fall rainy season of 1995 with orientations parallel to the high angle joints and faults exposed in the western escarpment. It was apparent that a structurally confined bedrock failure had developed in contrast to the 1994 failure which was primarily a debris slide/flow of accumulated colluvium and older landslide debris. Survey data indicated a general direction of movement along an azimuth of 90 degrees and dipping at about 55 degrees.

WSDOT completed two coreholes above and behind the western escarpment to identify possible shallow east dipping structures, that were not visible at the surface and could be responsible for the continued slow creep of the bedrock blocks toward the 1994 landslide track. Golder completed additional geophysical surveys to evaluate the persistence of a high angle structure (joints and faults) that appeared

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[2] Associate Engineering Geologist, Golder Associates - Redmond, Washington
[3] Chief Engineering Geologist, Washington State Department of Transportation - Olympia, Washington

to be forming the boundaries of the moving bedrock blocks. The geophysical survey results suggested that a continuous northwest trending, subparallel structure extended to the west. It was concluded that the blocks of the western rock slope were moving along a structure with an anomalous orientation not apparent in outcrop, and that the slow gradual, downward angled nature of the deformation was consistent with a bearing-type failure along this structure.

The rock mass eventually moved out into the debris slide track in January, 1996. The post-failure geotechnical field review of the rock slope revealed the probable mechanism for movement was due to a bearing capacity failure along a weak bed of volcaniclastic siltstone (of an unknown thickness) that was underlying the failure block. The initial direction of movement was east parallel to the bedding strike, until the rock material reached the south facing debris slide track. The rock failure was bounded by steeply dipping discontinuities west and north of the failure block. A Digital Terrain Model (DTM) comparison of pre and post failure topography of the 1994 landslide track indicated that there was no net gain in elevation, with the exception of the extreme western lateral edge of the landslide track, and near the toe of the bedrock failure. This additional material should not affect the short term stability of the debris slide track.

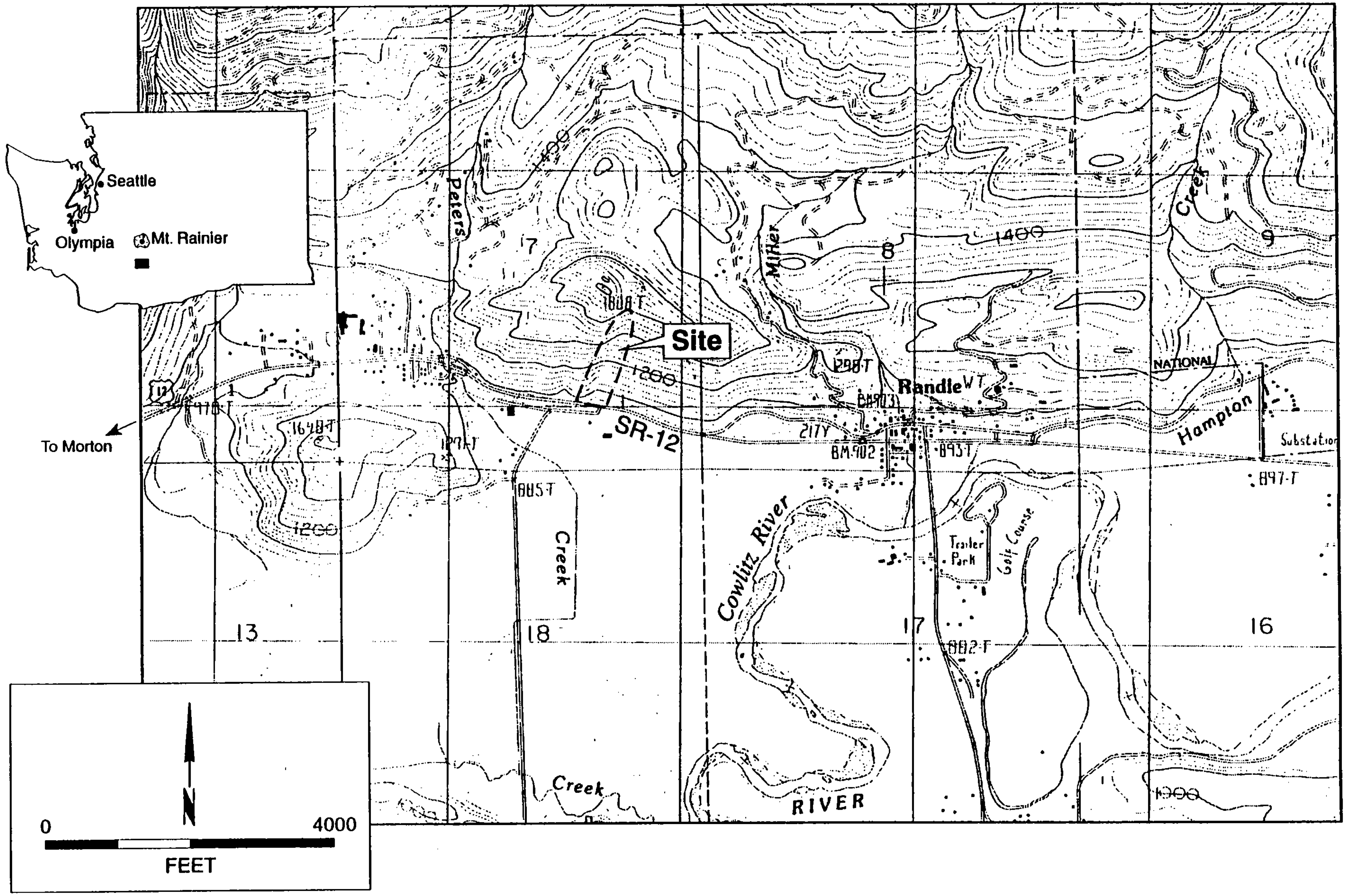
It is expected that these mass wasting processes will continue at the same scale and with the same general mechanisms. In the short term, it is expected that the unstable portion of the rock slope will continue to form structurally bounded failures and/or bearing capacity failures similar to the January 12, 1996 event. In addition, minerologic degradation of the rock and added debris will contribute to sliding. These ongoing mass wasting processes will result in decreased stability within the slide track as colluvial deposits thicken, resulting in higher landslide risk to the highway, at its current location. It was decided that further attempts to stabilize the western rock slope should not be attempted because the risk was too high for the added benefit. Mitigative measures currently being implemented include realignment of the highway around the toe area of the slide and construction of a debris catchment berm located between the new highway location and that same toe area.

November 22, 1994 Landslide Event

The site is located on the south facing slope of a generally east-west trending ridge that slopes to the south toward the flood plain of the Cowlitz River within the western foothills of the Cascade Range of western Washington State (see Figure 2). The approximate elevation of the crest of the slope is around 2,000 feet. The elevation along the toe of the slope along the SR 12 alignment is around 920 feet. Slope angles in the areas outside the 1994 slide track are around 30 degrees.

A large debris slide occurred on Tuesday November 22, 1994 (at approximately 6:00 PM), near MP 114 of SR-12, approximately one mile west of Randle, Washington as shown on Figure 1. The debris slide occurred during a period of heavy prolonged rainfall. Rainfall data from the U.S. Forest Service ranger station located in Randle, indicate that at least 16.82-inches of rain fell in the 42 days prior to the landslide. In just 22 days before the event 8.68-inches fell and 1.96-inches occurred from the intense 24-hour storm event 2 days prior to the 1994 slide.

The debris slide initiated as a translational failure of a deep talus/colluvium deposit, with the toe of the debris slide located north approximately 600 feet upslope and north of the highway. The slide crossed the highway and deposited a large amount of debris on the highway to depths of 30 to 40 feet. This landslide debris consisted of talus, colluvium, and forest debris. The eastern portion of the debris slide remobilized between November 22 and November 26, 1994 as a saturated/viscous flow that over -topped



Base map from USGS 7.5 minute topographic quadrangle "Randle", 1989.

Figure 1. Vicinity Map

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the landslide debris already present on the highway. The movement velocity, as measured from a series of survey lath lines set perpendicular to the slide track, was on the order of 1.5 to 2.0 feet per hour.

Flow type movement continued for about one week at the same approximate rate. Movement velocity decreased beginning on December 4, 1994 and by December 8, 1994 the rate had slowed to less than a foot per day. Movement of landslide debris within the slide track ceased by mid-December 1994. In addition to the movement of colluvial material in the slide track during this period, a portion of a high bedrock escarpment located on the western lateral edge of the slide track broke away from the main outcrop on the afternoon of (Monday) November 28, 1994. This bedrock failure occurred a high angle wedge formed by two repeating joint sets present within the western bedrock rock slope. This bedrock failure probably occurred as a result of the loss of lateral support that was provided by the colluvial/talus deposits that moved the previous week.

Due to the risks involved from the slope instabilities that were occurring upslope of the highway, a 1000 feet long temporary detour alignment was constructed to the south of the existing SR-12 alignment. This detour alignment was opened to traffic on December 4, 1994. In addition, an earthen berm was constructed between the detour and the existing highway alignment to provide some protection to the traveling public in the event of a rapid, debris flow and/or rockfall incident.

Geotechnical Investigation

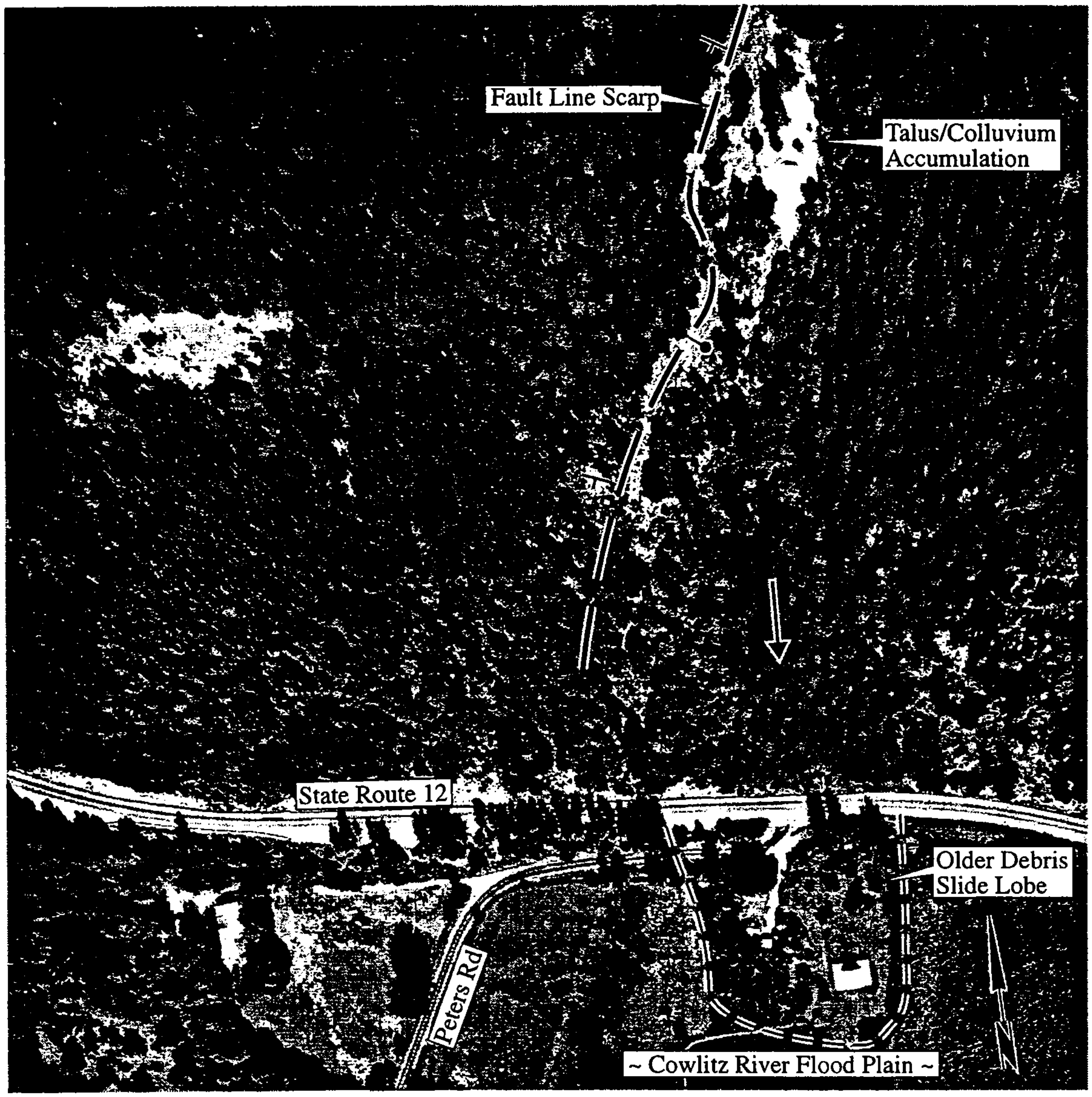
A field investigation was initiated to develop an understanding of the debris slide and develop potential remedial options. The field investigation was also directed at determining if highway improvements made the preceding year (that included widening of the shoulder and resultant small slope cuts) contributed to the debris slide. The field investigation consisted of aerial photographic analysis, general field reconnaissance, test pit excavations, geophysics, and core drilling.

Aerial Photogeologic Analysis Vertical and oblique aerial photographs (including pre- and post-1994 slide photographs) were used during the geotechnical investigation. Review of pre-1994 debris slide aerial photographs clearly showed geomorphic evidence that failures larger than the November 1994 debris slide had occurred in the past in the same approximate location. A distinct lobate toe of a prehistoric debris slide is evident in the same general area of the 1994 debris slide. This feature is approximately 500 feet to the south side of the existing SR 12 alignment as shown on Figure 2.

The pre-1994 aerial photographs also showed an active talus slope, devoid of vegetation, developing along the base of an east-facing rock scarp that forms the west lateral scarp of the November 1994 debris slide, see Figures 2, 3 and 4.

The aerial photographic analysis illustrated the persistence of some of the steeply dipping joint and fault structures that were observed in the western bedrock lateral scarp of the 1994 debris slide track. The structures extend into the forested second growth timber west of the debris slide track. Because of the persistence of the joint sets, a concern was that continued structurally controlled rock failure of the western lateral scarp would occur because of the loss of any buttressing effect the talus/colluvial apron, provided by the debris slide in 1994.

Site Reconnaissance An aerial and field reconnaissance was conducted on December 15 and 16, 1994 to collect information to evaluate the existing stability of the debris slide and the stability of the areas to the west and east of the 1994 debris slide track. The field reconnaissance concluded that:



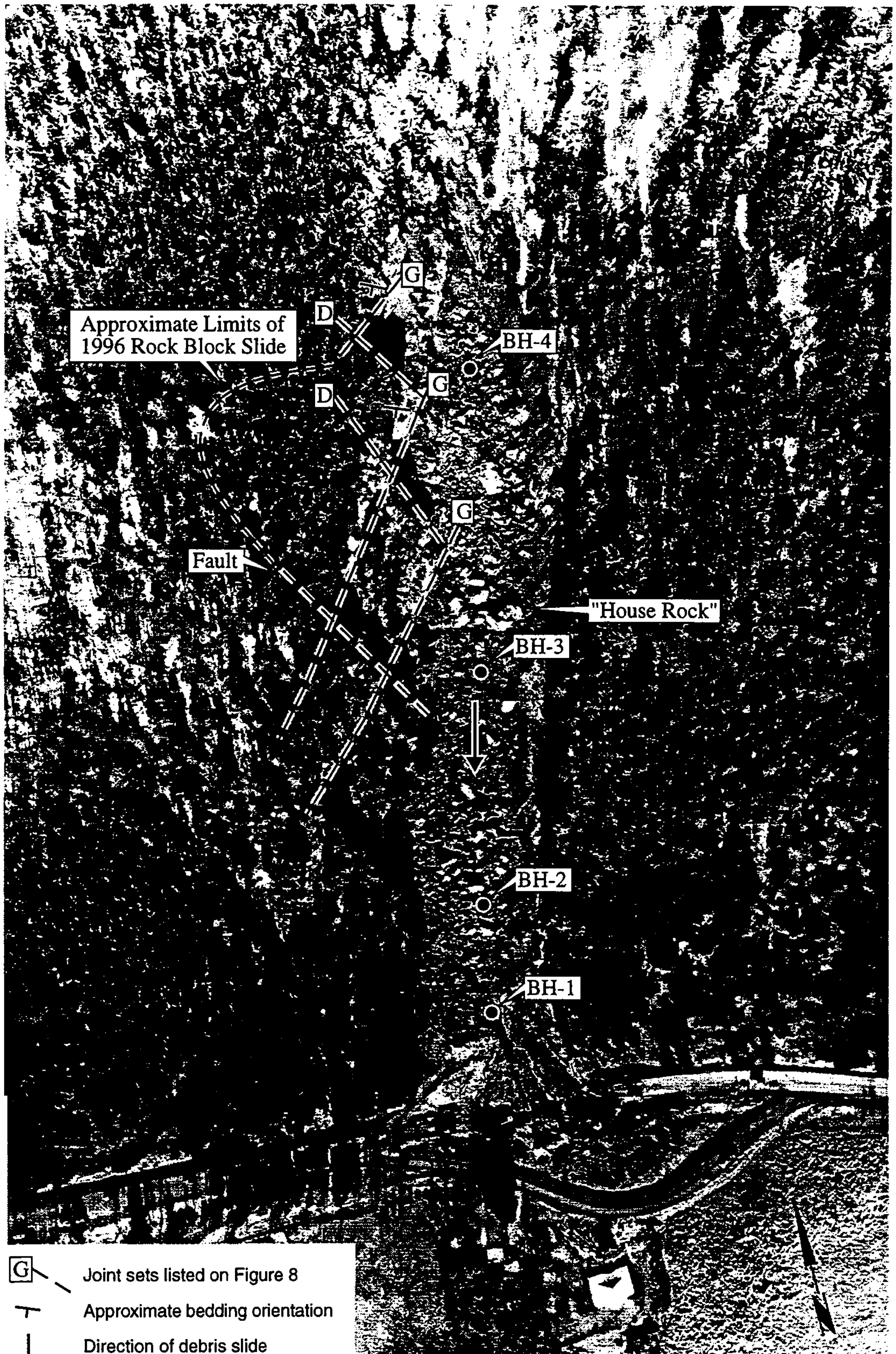
Note active talus development along east-facing fault line scarp. Barn on south side of SR 12 is located on distal toe of prehistoric debris slide.

0 800
FEET
Approximate Scale

Fault Line Scarp, ball on down-thrown side

General strike and dip of bedding

Figure 2. Pre-slide aerial photograph taken 21 Sept. 91.



Note same barn on south side of SR 12 as shown on Figure 2 and temporary detour road around toe of debris slide. Note pronounced NNE striking parallel joints on west side of debris slide track.

0 600
FEET
Approximate Scale

Figure 3. Aerial photograph taken 4 Dec. 95 of November 1994 debris slide.

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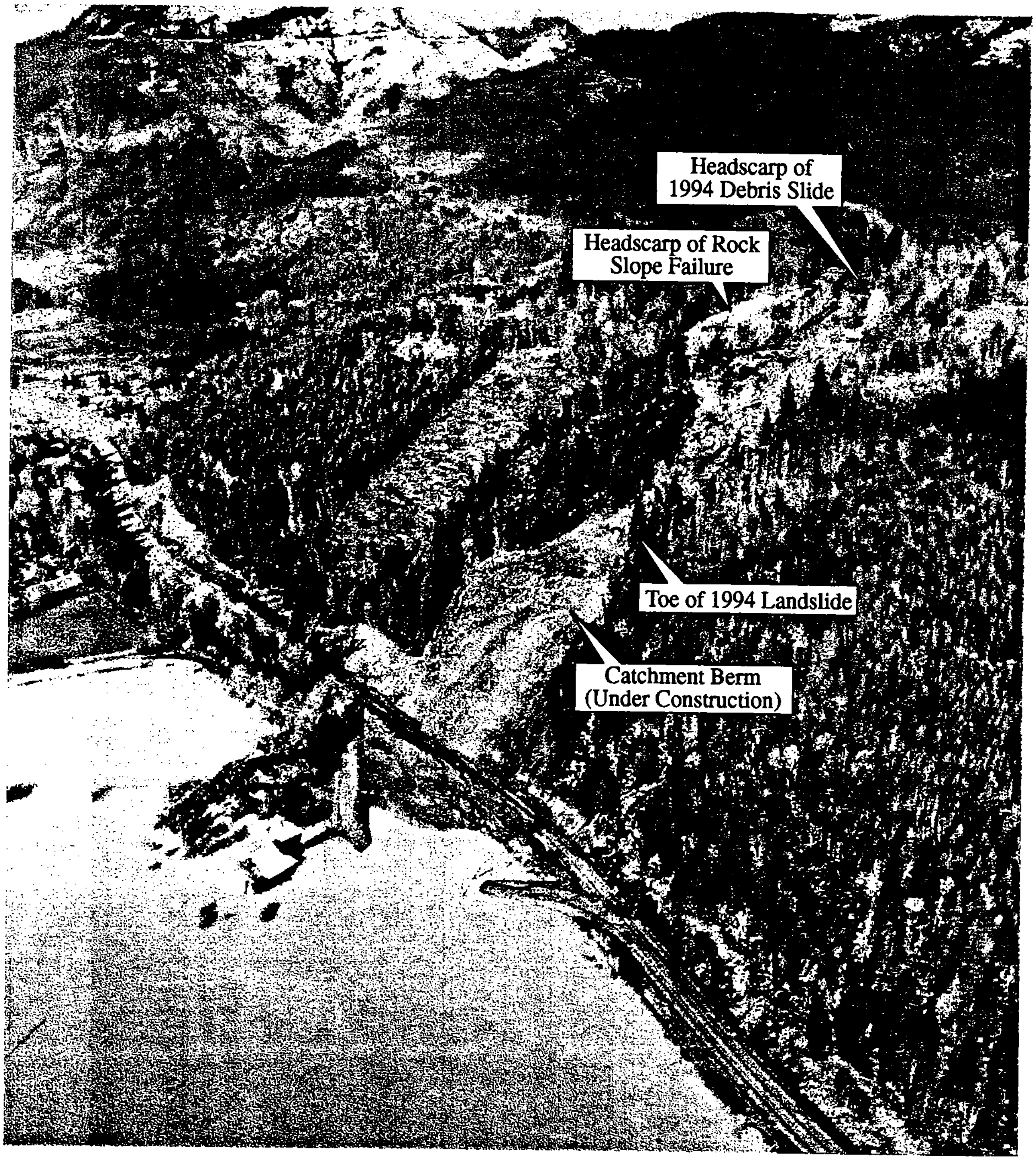

Figure 4. Aerial Oblique of Peters Road Landslide.

- the debris slide mass appeared stable under the current conditions;

- the debris slide appeared to be located along a roughly north-south striking, 80 to 120 feet high, east-facing fault scarp that formed the western limits of the November 1994 debris slide. The scarp formed a distinct topographic step along the east-west ridgeline at the top of the slope:

- several unstable blocks along the west lateral bedrock escarpment were identified;

- no tension cracks were identified behind the lateral scarps,

- several areas were identified as possibly being underlain by older landslide debris outside of the November 1994 debris slide track ;

- the landslide may have been the result of failure within highly weathered or altered, weak colluvium. or older landslide debris that occurs along the base of the western bedrock lateral escarpment;

Geophysical Exploration Geophysical exploration methods included seismic refraction, and electro-magnetic techniques(EM34 and VLF). The purpose of the geophysical program was: to delineate the limits of the suspected fault zone, estimate the thickness of the remaining debris slide deposits, and complement the increased subsurface data acquired through the drilling program. The seismic refraction program was used primarily for determining the depth to competent bedrock (thickness of colluvium and landslide debris). The seismic model was calibrated with the subsurface data collected from the boreholes.

The EM 34 was most useful in delineating the approximate horizontal limits of the steeply dipping fault zone that underlies the debris slide track. The EM34 can measure changes in the subsurface materials electrical conductivity due to metallic conductors, lithologic and water content changes. Fault zones and/or fractures that are clay-filled often have a higher conductivity than the surrounding rocks and can produce anomalous conductivity readings. A typical response of the EM34 over a relatively thin linear conductive zone has an increase in conductivity values on either side of the zone and apparent low (possibly negative) values centered over the body. An example of the EM-34 response across the fault zone is shown on Figure 5.

Drilling Program The subsurface drilling program consisted of four boreholes in the main body of the 1994 landslide and three boreholes east of the November 1994 landslide. The locations of the boreholes are shown on Figures 6 and in the sections of Figures 5 and 7. Prior to the start of the drilling program, an unstable block of bedrock, 30 feet in diameter, located along the western lateral scarp adjacent to the location of BH-4 was removed by Pacific Blasters under contract to WSDOT. In addition to the removal of this block, Pacific Blasters performed selected rock scaling along the western bedrock lateral scarp prior to the drill rig entering the debris slide track.

Borings were drilled with a track mounted, modified Mobile B-30 drill with a 4-inch ID casing advancer until intact rock was encountered, and then fully-cored to final depth using HQ-sized wireline equipment. A triple-tube core barrel was utilized for the rock core drilling operations. Standard penetration tests were performed in the unconsolidated materials. An inclined borehole was drilled at an angle of 50 degrees from the horizontal, and oriented to the east in an attempt to drill across the steeply dipping fault zone. Sealed standpipe piezometers were installed in selected holes to enable monitoring of groundwater levels, with screened intervals of 5 to 10 feet long. Slope inclinometers were installed in several boreholes to monitor for slope movement.

Site Geology The information obtained from the field exploration program indicated that the area located to the east of the debris slide track is underlain by a variable thickness of colluvium and older landslide debris overlying bedrock. The bedrock encountered consisted of interbedded andesite and volcaniclastic rocks striking west-northwest and dips 25-30 degrees to the south. Within the landslide track, landslide debris overlies clay fault gouge and breccia. The primary geologic units are described in more detail below.

Colluvium (Qc) Colluvium was encountered in varying thickness outside the landslide track. It varies in composition ranging from loose to compact, nonstratified, angular to subrounded sandy gravel, with some silty clay. The colluvium is commonly iron oxide stained.

November 1994 Landslide Debris (Qls) The landslide debris is generally loose, mottled, nonstratified, fine to coarse angular gravel with some sand in a silty clay to clayey silt matrix, with some boulders. Many of the larger boulders are truck and house size.

Mt. St. Helens Y Tephra (Qt) The tephra is exposed in the head and lateral scarps of the 1994 land slide track, the test pits, and borings. The tephra, of variable thickness underlies locally underlies the modern soil and ground surface. The tephra has been tentatively identified as Mt. St. Helens Y Ash based on the presence of comingtonite, a diagnostic mineral of the Mt. St. Helens Y ash. The Mt. St. Helens Y tephra has been dated at approximately 3,300 to 4,000 years old (Mullineaux and Crandell, 1981). The tephra is typically loose to compact, nonstratified silty lapilli size tephra. Iron oxide staining is common.

Older Landslide Debris (Qols) Field evidence suggest at least two and possibly three landslide events have occurred in the past at the site. Chaotic deposits of soil-like material were encountered both above and below the Mt. St. Helens Y tephra layer. These deposits typically consist of dense, mottled (commonly iron oxide stained), nonstratified, fine to coarse angular gravel size clast in a silty clay to clayey silt.

Fault Gouge (fg) The fault gouge was encountered in the boreholes and was observed in selected exposures near the existing highway cut and along the eastern lateral scarp. The fault gouge is generally stiff to very stiff, mottled, commonly iron oxide stained, silty clay with some to a trace of subangular gravel.

Volcanic Conglomerate (Tvc) The volcanic conglomerate is fresh to moderately weathered coarse grained, crudely stratified, medium strong (ISRM R3) to strong (ISRM R4) with thin (1 to 2- feet thick) interbeds of fine to medium grain sandstone. The conglomerate is typically composed of well rounded andesite clasts in a fine sand to siltstone matrix. The conglomerate matrix locally weathers rapidly, probably the result of montmorillonite clay mineralogy. Within the span of a few months blocks of conglomerate slide debris had degraded to a dense soil-like consistency. This rapid weathering process is largely responsible for the accumulation of colluvium at the base of the east-facing fault scarp.

Within the mass of slide debris, numerous large blocks of conglomerate were observed to exhibit slickensided and polished surfaces. Slickensides were located across both the matrix and the andesite clasts.

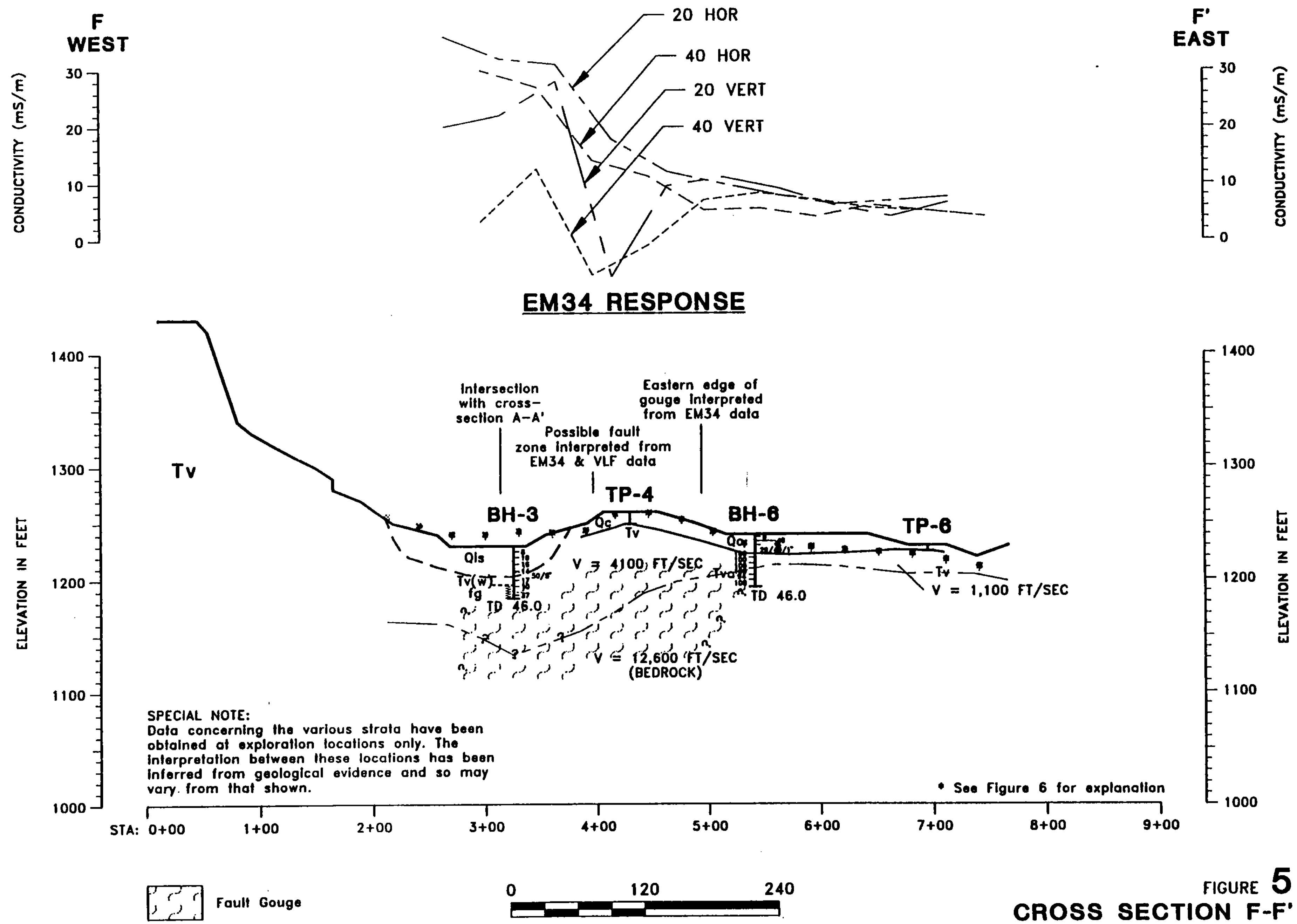

FIGURE 5
CROSS SECTION F-F'
WSDOT/PETERS SLIDE/WA

Golder Associates

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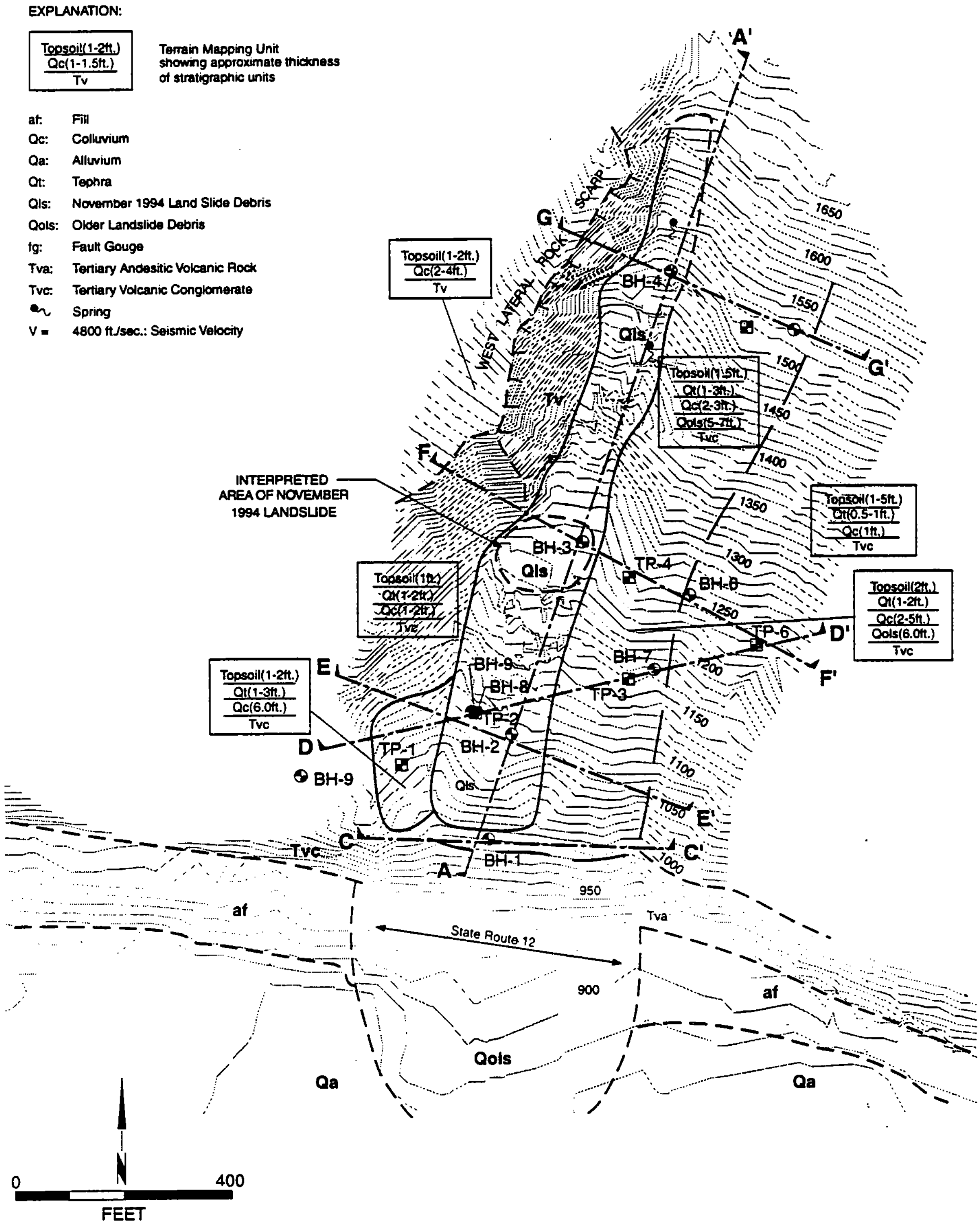
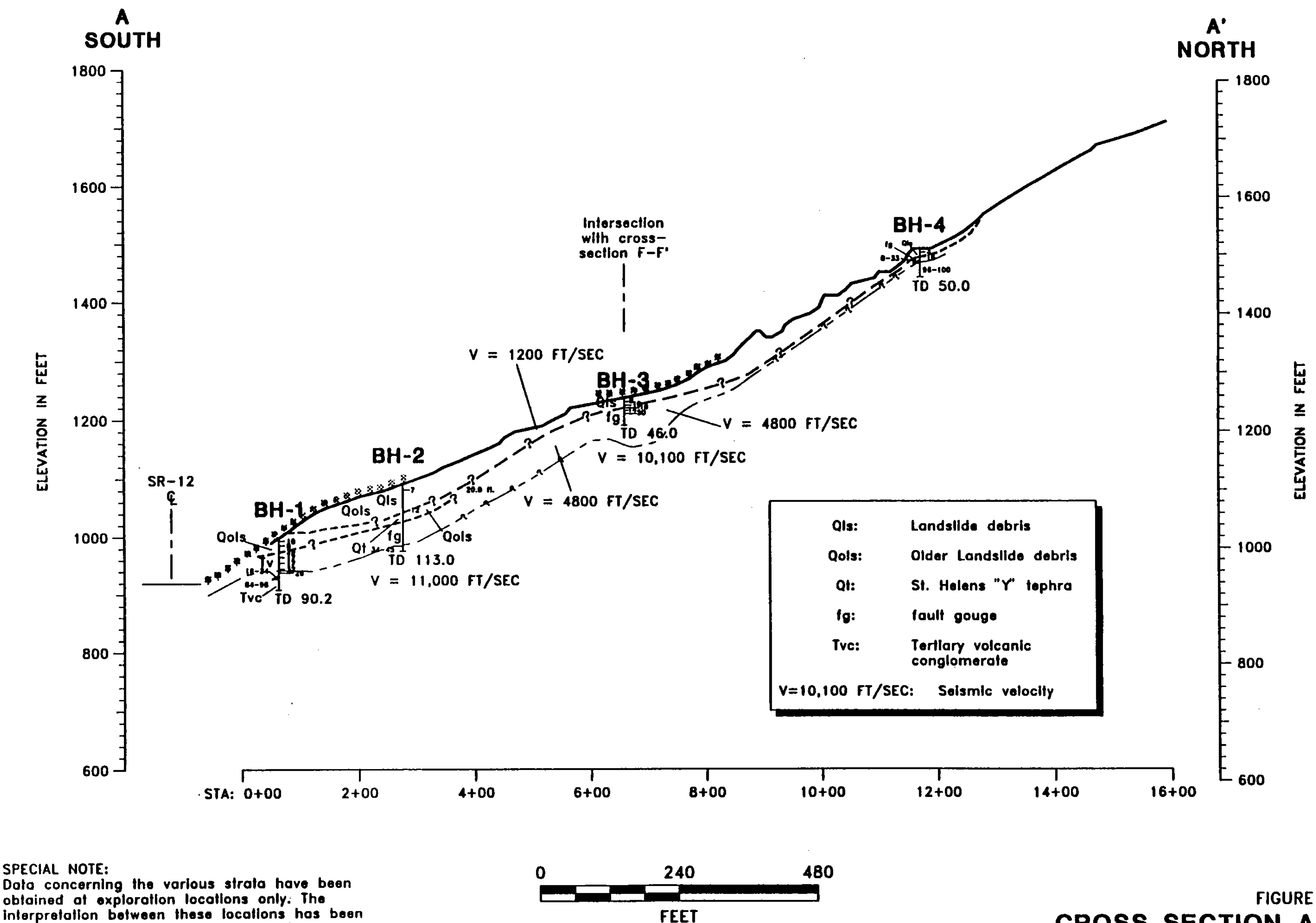


Figure 6. Terrain Unit Map



SPECIAL NOTE:
Data concerning the various strata have been obtained at exploration locations only. The interpretation between these locations has been inferred from geological evidence and so may vary from that shown.

FIGURE 7
CROSS SECTION A-A'
WSDOT/PETERS SLIDE/WA

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Golder Associates

Andesite (Tva) Fresh, massive, fine to medium grained, very strong (ISRM R5) andesite was encountered in the boreholes located east of the November 1994 slide track.

Structural Geology The bedrock geology in the vicinity of the site is composed of Oligocene to Lower Miocene age volcaniclastic rocks and andesite flows. The entire south facing slope upon which the 1994 landslide is located has been mapped as Lower Miocene age volcaniclastic rock by Schasse (1987) who describes these lithologies as volcaniclastic rocks composed of tuff, tuff breccias, lithic breccias, conglomerate, and volcanic sandstone interbedded with lava flows. Deposits overlying the bedrock include colluvium, talus, and volcanic tephra identified as Mount St. Helens Y tephra

These rocks have been locally folded and faulted. Schasse (1987) has mapped a south-southeastward plunging anticline located north and east of the site. The site is located on the southern limb of this anticline. The southerly dipping strata observed in the areas adjacent to the slide are consistent with the antiform mapped by Schasse. The orientation of bedding within the volcaniclastic bedrock in the western lateral landslide scarp generally strikes west-northwest and dips at about 25-30 degrees to the south. Joint spacing varies from several feet to tens of feet. Intersections of these joint sets form high angle wedges. The structures are depicted graphically in the steriographic plot of Figure 8.

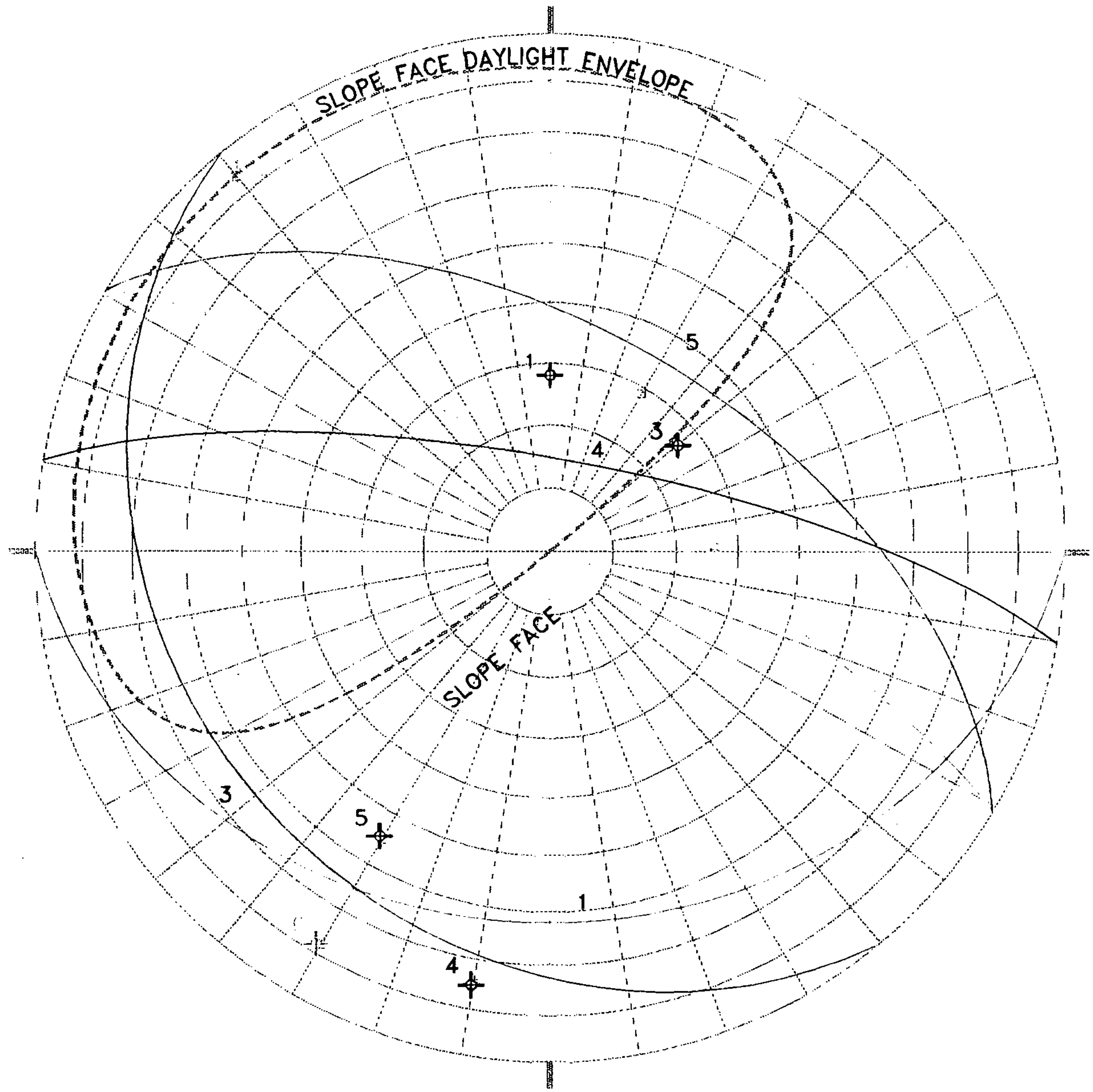
Based on review of pre- and post-94 slide aerial photographs, field observations and the subsurface information gathered from the drilling and geophysical programs, a north-striking fault is located along the approximate axis of the slide track. The fault has produced the 80- to 120- feet high, east-facing fault line scarp (down-to-the east) that currently forms the western limit of the debris slide track. The down-to-the-east scarp is particularly evident along the east-west ridgeline at the top of the slope.

The extensive thickness of clay gouge and brecciated bedrock encountered in the boreholes drilled in the 1994 slide track support the presence of the fault zone. Additional evidence includes the slickensided and polished surfaces on some of the conglomerate blocks and offset strata. The geophysical survey indicate the fault zone is widest at the southern end of the slide track and narrows to the north.

Debris Slide Mechanism Based on the field evidence, the debris slide which occurred on November 22, 1994 is a recurring phenomena resulting from a combination of on-going weathering processes, a localized high degree of weathering or alteration resulting from montmorillonite clay mineralogy, the presence of the steeply dipping fault zone, and infiltration from an unusually heavy rains.

Geomorphic evidence also suggests that debris slide activity occurred in the past. Stratigraphic relationships also suggest that similar slide events have occurred in the past. An area interpreted to be older landslide debris is delineated near the southwest corner of the 1994 slide near test pit TP-1. Field evidence indicates at least one large slide event occurring before Mt. St. Helens Y tephra deposition, one large post-Mt. St. Helens Y tephra- slide event, and the most recent event in 1994. Assuming the Mt. St. Helens Y tephra is between 3300 to 4,000 years old, the field evidence supports conservative estimates of at least two large scale slide events with-in the last 3,300 years, with similar characteristics to that which occurred in November 1994.

This interpretation is supported by the results of test pit explorations and the hummocky surface topography in that area. Landslide debris (1994) overlying the pre-94 slide ground surface and top soil is exposed in a temporary road cut near BH-1. Beneath the modern topsoil, Mt. St. Helens Y tephra overlies what is interpreted as older slide debris.



EQUAL AREA
LOWER HEMISPHERE

MAJOR PLANES

ORIENTATIONS #	DIP/DIR.	
1	28/180	BEDDING
3	26/231	BEDDING
4	75/010	JOINT
5	55/031	JOINT
6	78/031	JOINT
	84/130	JOINT (SLOPE FACE)

POLES

7 Poles Plotted
7 Data Entries

FIGURE **8**
SURFACE MAPPING DATA
WSDOT/PETERS SLIDE/WA

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Golder Associates

We developed a model which applies the periodic failure of accumulated colluvium and talus. This accumulation is interpreted as a loading mechanism with failure probably occurring along the contact of the accumulated colluvium and the underlying fault gouge. This model is consistent with the recurring nature of the debris slide events and the observed weathering characteristics of the bedrock lithologies forming the east-facing rock slope. The low permeability fault gouge perches groundwater and contributes to elevated piezeometric levels during periods of heavy or prolonged precipitation.

Sources of groundwater that could have impacted the stability of the accumulated colluvium include: a spring which discharges onto the slope, surface rainfall infiltration, and fracture flow along the bedding and joint planes daylighting in the east-facing rock slope. Some or all of these potential sources may have had an impact on the initial 1994 landslide. Groundwater was observed seeping from the bedrock discontinuities during the December 1994 and January 1995 field work and ceases to flow in the dryer months. Aerial photographs taken in 1991 and 1993 clearly show an active talus slope along the base of the steep, east-facing rock slope. The relatively rapid accumulation rate of the talus is supported by the complete lack of vegetation on the talus surface. The colluvium/talus accumulates until it reaches a marginally stable condition and then fails when adverse groundwater conditions develop, as would be the case in 1994 after higher than normal rainfall and the intense rainfall event two days prior to the landslide. The geologic process of accumulating colluvium along the base of the east-facing rock begins anew after slope failure.

Debris Slide Analyses Core holes BH-1, BH-2, BH-3, and BH-4 were drilled along the axis of the 1994 debris slide track. These core holes as well as the geophysical data were used to construct the geologic cross sections shown on Figures 5 and 6. The debris slide track is a compound slope, see Figure 7, with the upper slope (north of BH-3) at about 29 degrees. The lower slope (south of BH-3 is situated at around 22 degrees). Stability analyses were performed using the geometry, groundwater conditions and existing soil stratigraphy determined from the 1994 debris slide track investigation to:

- determine the approximate soil and groundwater parameters at the time failure, by "back analyses".
- evaluate the stability of the lower portion of the slope where debris was deposited above the highway and
- to check the stability of a proposed debris catchment embankment.

In conducting the back analyses, it was assumed the colluvium/talus was deposited upon a slope similar to the post failure slope angle of 29 degrees. It was assumed the colluvium/talus had no long term cohesion and the implied friction angle was 29 degrees or greater depending upon the assumed seepage pressures. Based on a variety of assumed groundwater levels at failure, the colluvium/talus likely had a strength of ϕ' =30 to 32 degrees, c'=0. The lower portion of the debris slide track (located on the 22 degree slope in the vicinity of BH-2) had, depending on the assumptions, a Factor of Safety (FS) of about 1.5. The potential for development of a failure surface in the deeper underlying fault gouge was also evaluated. The analysis indicted that the gouge material was not likely contributing to sliding. The analysis assumes the gouge has a ϕ' =29 degrees and c' = 500 psf and that the water surface levels are generally within about 18 feet of the ground surface. Artesian groundwater conditions were encountered in BH-1. These artesian conditions appear to reflect a confined aquifer located at the base of the gouge near the lower portion of the old landslide deposit. Based on the analyses, the gouge is at equilibrium in this area and artesian pressures were not expected to affect the stability of the slope.

In summary, the stability analyses suggested that the build up of colluvium/talus on the upper 29 degrees slope resulted in an unstable condition leading to the debris slide. However, the debris on the lower 22 degree slope was currently stable.

The risks associated with future ground movements impacting the road include:

- MAJOR SLIDE EVENT: The risks associated with another major slide event similar to the 1994 slide depend on the rate of colluvium accumulation, surface and groundwater conditions, and material strength properties. Evaluation of the historic frequency of landslide events provides the best indication of future landslide events. Based on the previous discussions, it appears that major slide events which could bury the roadway occur with a frequency in the range of several hundred years.

- SMALL DEBRIS FLOWS: Localized debris flows likely occur more frequently and may also impact the road. Based on hearsay information, these types of flows appear to have occurred every 25 to 50 years.

- SEISMIC EVENTS: A major seismic event could trigger movement of the colluvium and/or cause rock fall from the steep western scarp.

Mitigation The risks of a major slide event impacting the road could be reduced by 1) elimination of the source of the colluvium. removal of colluvium such that it never reaches an unstable depth, and/or 2) protection of the roadway. The risks of smaller events impacting the road could be reduced by improving site grading and drainage above the roadway and providing a small berm or wall to divert small events.

Alternatives were evaluated to protect the existing highway from future landslide activity with the use of a debris wall, debris embankment, and/or catchment basin. The size of event contained had a major impact on cost of the roadway protection. Due to the large debris volume involved, it was determined to be impractical to design a catchment structures large enough to contain another event the size of the November 1994 slide. These options all involved future maintenance to remove any debris that accumulates in the debris catchment zone. Periodic geologic mapping and monitoring of the slide area would be required.

Alternative Evaluations and Risk Assessment In order to select an alternative. an informal cost-benefit analysis comparing the various alternatives was completed based on a subjective listing of advantages/disadvantages The final selection was made using engineering judgment

The risks associated with each alternative were subjective based on a "degree of belief" and were provided to allow a basis for comparing relative risks only. Based on the small incremental decrease in risk, it was concluded the more costly options were not cost effective and represent a poor use of public funds.

Specific Design Recommendation Meetings were held with WSDOT and FHWA personnel to discuss the various alternatives. The outcome of these meetings was that the likely chosen alternative would consist of construction of a small catchment berm to protect the road from small debris flows that could be generated within the November 1994 landslide track. The general concept was to improve the drainage conditions. regrade the steep areas immediately above the highway. and construct a catchment berm to contain small to moderate debris flows.

Western Rock Slope Movement and Stability Analyses

During construction of the catchment berm in the fall of 1995, movement within the western rock slope was discovered by the contractor. The contractor had originally observed an 8 inch wide tension crack, see Figure 3, on the upper portion of the western rock slope on September 25, 1995. On October 20, 1995, the contractor re-visited the site and found that the tension crack had widened considerably. The following day a combined field review of the western rock slope and scarp was conducted by WSDOT and the contractor.

The unstable rock mass consisted of two massive beds of volcaniclastic conglomerates designated Bed A (lower bed) and Bed B (upper bed) separated by a thin dark gray layer of friable, clayey siltstone as shown in Figure 9. The near vertical tension crack was located approximately 1400 feet upslope of the highway, trended subparallel to the November 1994 landslide track, and could be traced along the ground surface for approximately 200 feet. On the up-slope end of the tension crack, recent movement was observed, as indicated by torn organic forest duff and tilted trees. The lateral displacement of the rock mass at this location was estimated at 6 to 8 feet with the majority of movement being horizontal. A southeast trending tension crack, located at the down-slope end of the failure block was also observed, but was less well defined than the northeast trending tension crack. The width of this unstable rock mass was estimated to be approximately 60 to 80 feet on the upslope end, and well over 200 feet on the downslope end. The estimated height of this unstable rock mass, above the 1994 landslide track was on the order of 200 feet. The top of the unstable rock mass sloped downward from the northeast to the southwest at about 25 degrees.

Site Stability Analyses Once movement of rock in the western lateral scarp became apparent in October, a survey monitoring grid was established by the WSDOT. Based on the survey data taken between October 31, 1995 and December 28, 1995, the block gradually moved downward at an average angle of 50 degrees to the east at rates averaging .2 ft./day of vertical movement and .09 ft./day of horizontal movement. The horizontal and vertical velocities then increased to about .4 to .6 ft/day with a movement vector angled at about 38 degrees to the east until January 13, 1996 when the major block finally failed into the 1994 debris slide track, see Figure 10.

Kinematic failure modes were evaluated using the measured structural data, see Figure 8. The analyses largely discounted any kinematically possible failure mechanism based on the known structural geologic data. We evaluated a block failure geometry using a cross section 200 feet high , with a full depth tension crack and 150 feet base width. A Factor of Safety of 1.0 could be obtained with reasonable rock strength parameters and a water height of at least 125 feet in the tension crack. Since there were no observations or evidence that water pressures were near this high, the rock block failure along a sliding plane parallel to the bedding strike was not considered a viable model.

The apparent deformational history of the western block was puzzling in that the block was apparently failing along a surface that did not correspond to any known or observed structure. In fact the movement vectors indicated movement nearly parallel to the strike of the exposed bedding. In an attempt to more clearly define the structures, WSDOT drilled two exploratory coreholes west of the moving block. Two angled drill holes were planned but rock movement did not allow completion of these boreholes.

Post Failure Evaluation On February 27, 1996 a post failure geotechnical field review was conducted with the following conclusions:

* Section of "missing" rock mass after January 12, 1996 failure.
See Figure 10 for post failure comparison.

Figure 9. Photograph of west lateral scarp area of unstable block taken on Jan. 3, 1996.

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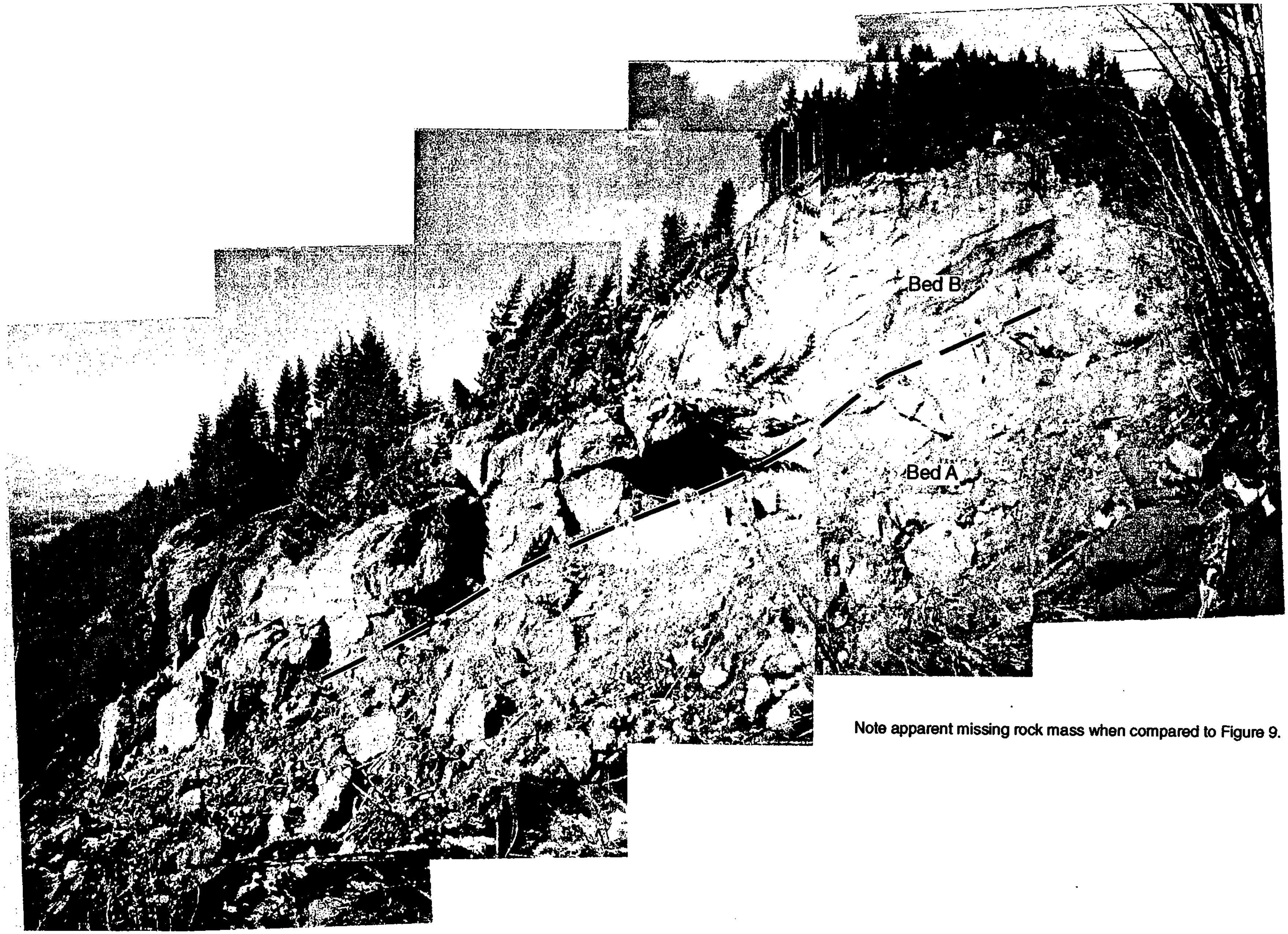

Note apparent missing rock mass when compared to Figure 9.

Figure 10. Photograph of failed west lateral scarp rock slope taken on Feb. 27, 1996.

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- The head scarp area of the failure was bounded by two discontinuities. The orientation of these discontinuities were measured (dip direction/dip) at 090/55 and 175/65. Slickensides that were present on the surface of the 090/55 discontinuity indicate that this discontinuity is probably a strike-slip fault that may persist well behind the current headscarp of the failure. The orientation of these discontinuities did not match any of the discontinuities that had been previously mapped. The combination of these two discontinuities formed a large wedge with a plunge of 120/50. Further analyses of this wedge geometry, utilizing wedge stability charts and friction only calculations, indicated that the wedge geometry may be unstable. Additionally, when the plunge of the wedge geometry was plotted in relationship to the failed rock mass it did not daylight into the 1994 landslide track. In fact the wedge was "buttressed" by approximately 70 to 80 feet of bedrock; making it physically impossible for the wedge failure mode to be the primary mode of failure of the western rock slope.

- The failed rock mass in the headscarp area moved nearly east initially until reaching the debris slide track then approximately 60 feet in an almost due south direction. Vertical displacement in the headscarp area of the failed rock mass was differential with the upslope end of the block moving approximately 100 feet while the downslope portion of the block moving an estimated 30 feet. The rock mass did not disintegrate when it failed. Many of the features of the block are still identifiable, although it is evident that there has been large vertical and horizontal displacements. It is interesting to note that a comparison of pre and post failure photographs of this failed rock mass indicates that significant sections of massive conglomerates, located in the central portion of rock mass, appeared to be "missing". These "missing" sections of the rock mass did not fail into the slide track, and it seems that these portions of the rock mass were displaced vertically (for the most part intact), and are now "hidden" below the 1994 landslide track, see Figure 11.

- The toe of the failed rock mass has encroached into the 1994 landslide slide track approximately 60 to 100 feet. Large blocks of rock (house sized) that were once located in the bottom portion of the western bedrock outcrop detached, rotated approximately 180 degrees, and moved downslope approximately 200 feet. In addition, the encroachment of the failed rock mass into the 1994 landslide track displaced the landslide debris. This resulted in the flow-like movement of the lower portion of the landslide track, displacing material as much as 200 feet downslope of the toe of the failed rock mass, and inundating the area once proposed for the containment berm/catchment basin. In addition, there appears to have been a "bulldozing effect" within the landslide track, as evidenced by relatively large quantities of the underlying reddish brown fault gouge material being brought to the surface. This provides evidence that the failure surface of this failed rock mass was located below the surface elevation of the 1994 landslide track.

- A very weak, black volcaniclastic siltstone was present within the 1994 landslide track. This material was not present in the landslide track prior to the failure of the western rock slope. In addition, the material was not exposed in large volumes within the unstable rock mass prior to failure. A reasonable explanation for this material being present after failure was that it was injected laterally into the landslide track. This material probably represented a weak bed of volcaniclastic siltstone, of unknown thickness, that was underlying Bed A, and located below the surface elevation of 1994 landslide track.

- A lower portion of the rock slope, up-slope of the unstable rock mass, also moved. This area consists of an extension of the lower bed (Bed A). This lower bed may also be underlain by an up-slope extension of the very weak bed of black volcaniclastic siltstone. This weak bed may also extend underneath the 200 feet high, upslope portion of the rock slope.

- Geophysical surveys, using electromagnetics and ground penetrating radar techniques were conducted immediately west of the western rock face. These geophysical surveys suggest that high angle and persistent structure that parallels the western bedrock scarp are likely present to the west of the current scarp face.

- It seems most likely that as the rock mass movement accelerated, more of the load from the mass was transferred onto the weaker underlying siltstone. The ultimate bearing capacity of the siltstone bed was exceeded in early January 1996. The acceleration of the block during this period, as the lower beds began to deform, can be seen on the graph of survey data in Figure 11. Block movement continued for about the next ten days, doubling each day until the block moved into the debris slide track on January 12, 1996.

Summary and Conclusions

The Peters Road Landslide was reactivated during heavy rains in the fall of 1994. The initial debris slide covered SR 12 with 40 feet of rock and mud. The slide was confined to a NNE trending fault zone that offset a rock mass exposed in about 200 vertical feet of an escarpment on the west side of the slide. This rock mass eventually slid into the debris slide track , coming to rest on January 13, 1996, as a result of a bearing-type failure. The rock failure involved at least 200,000 yd^3 of debris material. It was determined by the project team that the advantages of further mitigation on the rock and debris slope would have limited long term effectiveness. Moving the highway a safe distance away from the immediate slide "runout" area is more attractive in terms of costs and benefits. Currently, the highway realignment is in the beginning phases of design and will include a catchment berm adjacent to SR 12.

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Rock Slope Stabilization at Dunderberg Mt. Rockland County, New York

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During 1994 and 1995 the New York State Department of Transportation conducted a rock slope remediation project along approximately two miles of Rte 9W in Rockland County, at the foot of Dunderberg Mountain, approximately 40 miles north of New York City. The highway is a main traffic corridor overlooking the west bank of the Hudson River. The area had a history of rockfalls due to the weathered nature of the rock. Minimal catchment area provided little or no protection to vehicular traffic in the event of a rockfall.

The site presented numerous problems which included: the proximity of dwellings and a railroad freight line, high traffic volume, steep terrain consisting of bare rock, and a high, steep, talus - covered back slope. Rockfall mitigation was accomplished through a variety of techniques tailored to the particular requirements of each location.

The southern half of the project consisted of rock slopes ranging in height from twenty (20) to one hundred (100) feet which overlooked homes and a busy rail freight line. Controlled presplit blasting and rock bolting were used to stabilize the rock slopes and improve sight distance along this approximately one mile of roadway. The northern half of the project included rock slopes similar in height to those of the southern section, above which lay a talus pile that extended for approximately two hundred yards up the forty five-degree back slope. In this area NYSDOT engineering geologists recommended a variety of stabilization methods. These methods included: rock scaling by hand, rock bolting, construction of concrete buttresses to support very large unstable blocks and the installation of rock catchment fencing at the toe of the back slope.

Work began in April 1994 and concluded in October 1995. Construction included approximately 70,000 cu. yds. of rock excavation, 3,000 lineal ft. of rock bolts, two concrete buttresses and 1,800 lineal ft. of rock catchment fencing. This paper presents the variety of rock slope problems encountered and the specific stabilization techniques which were employed.

REGIONAL GEOLOGY

Dunderberg Mt. lies within the Hudson Highlands physiographic province of southeastern New York, a narrow, elevated terrain consisting of Mid-Proterozoic (Grenvillian) age metamorphic rocks, which extends northeastward across Orange, Rockland, Dutchess, Westchester and Putnam counties. The Highlands are a part of the Reading Prong geologic province, which extends from Pennsylvania to Connecticut (Isachsen et al, 1991).

The project area is approximately 5 miles from the Ramapo - Canopus Fault, which serves as the boundary between the Triassic age Newark Basin and the Hudson Highlands, and lies within the Ramapo Seismic Zone, an area characterized by faults ranging in age from Proterozoic to recent. Faults within this zone exhibit complex histories and range in character from thrust faults with associated semiductile fabric to normal or oblique dip slip faults characterized by brittle structures (Ratcliffe, 1980).

Glacial erosion along faults in this part of the Hudson River Valley has carved steep rock faces along the valley walls. In fact, the valley is a true fiord where the Hudson River crosses the Hudson Highlands. The steep outcrops and the effects of unloading during deglaciation have resulted in talus at the foot of the rock slopes in the area. Part of the project area lies within such a zone: the roadway is essentially carved into both the mountainside and the talus pile. At three locations within the project limits unreinforced masonry walls constructed during the 1930's support the talus pile at the edge of pavement.

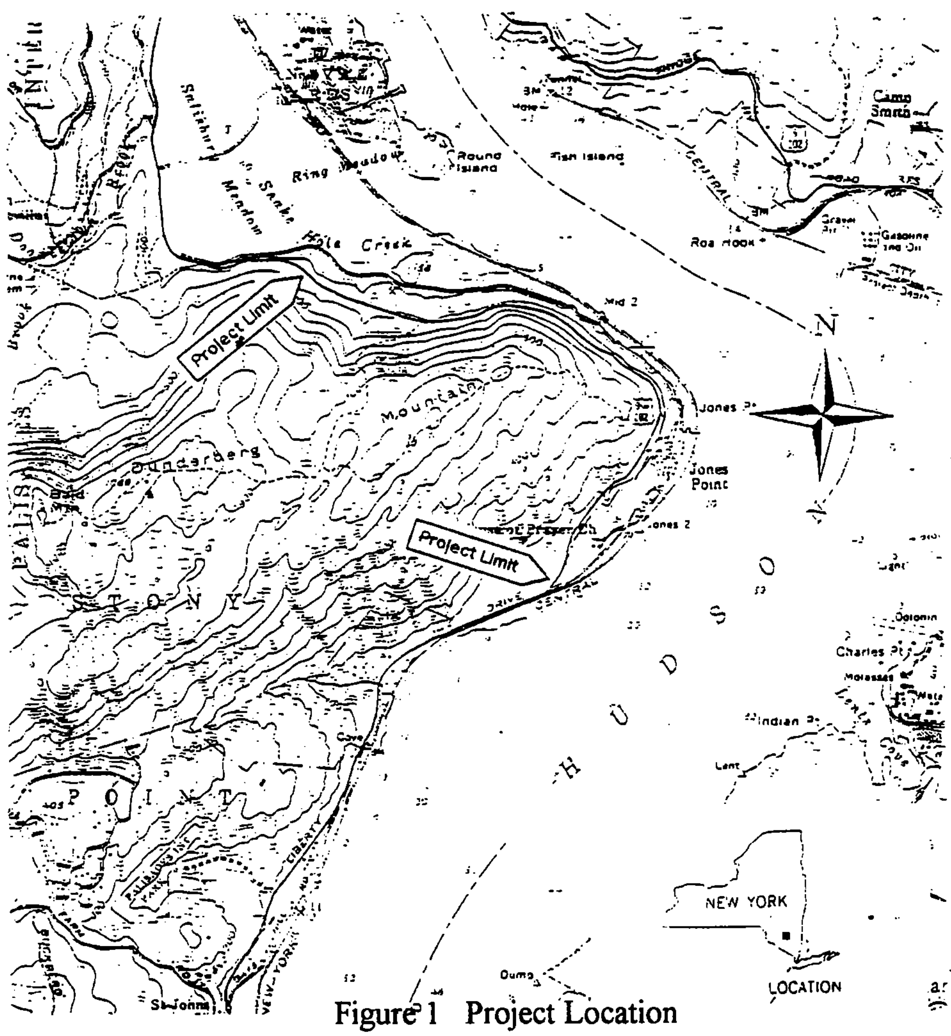

Figure 1 Project Location

The bedrock within the project limits consists of a horneblende granite gneiss characterized by a moderate foliation which is defined by mafic vs. felsic compositional layering and subparallel alignment of inequant mineral grains. Semiductile shear zones and Paleozoic dikes transect the rock cuts at widely spaced intervals.
Numerous joint sets exist. Mineral coatings and slickensides on joint surfaces are common. The principal joint sets in the area are shown in figure 2 .The intersection of these joints has resulted in potential plane and wedge failures. These failure planes coupled with fracturing from previous production blasting, differential weathering, and movement of the talus which lies above the rock face, created a zone of chronic rock fall.

JOINT ORIENTATION

STRIKE	DIP	FEATURES
98	52N	
100	43S	
134	60N	mineralization on joint surfaces
66	60N	mineralization on joint surfaces
76	15N	
30	80E	
76	90	
160	90	

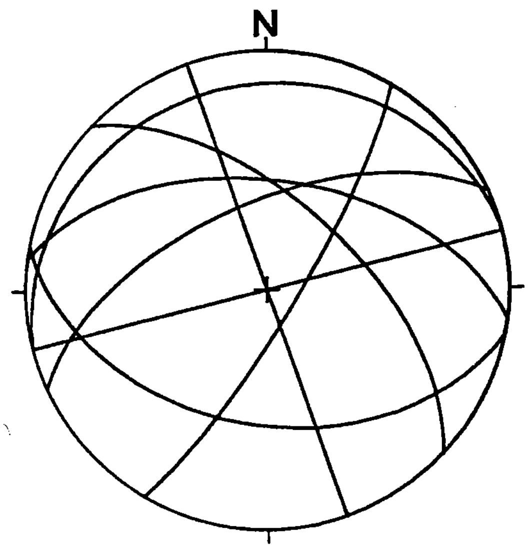

Figure 2 Orientation of principal joint sets.

DESIGN

The rock slopes located within the project were of longstanding concern. Rock falls were a common occurrence, which combined with limited setback from edge of pavement (3 to 10 feet) and severely restricted sight distances produced a significant risk to the traveling public. The highway rises from either end of the project to a short relatively flat section in the middle with rock slope heights varying between 15 and 100 feet. During the statewide rock slope evaluation program conducted by Departmental Engineering Geologists in 1988, these rock slopes were placed in the highest risk category, prompting a request from the Regional DOT office to provide rock slope stabilization recommendations. The initial remediation recommendations consisted of scaling and bolting of unstable rock, and construction of a concrete buttress at one location. These recommendations were not implemented. Over the next several years, a more comprehensive remediation program was developed. The new remediation program included the recutting of rock slopes where warranted, and where back slope conditions allowed the procedure to be performed safely and not destabilize the talus slope located atop some rock cuts.

The lithology of the project was characterized by granitic gneiss with weathered and highly fractured zones (including faults). Several joint sets occur and have variable spacing. Measurements of joints showed a prevalent trend of dip readings between 60 and 65 degrees. The rock slope at the extreme southern end of the project was generally sound with loose blocks sitting on well developed joints and occasional fractured blocks. This section was on a straight road section with adequate sight distance. Rock stabilization at this location was designed to remove unstable material by scaling and to lock key blocks of rock in place by rock bolting. These methods provided a stable slope economically. The next set of rock slopes in the southern portion of the project extended over 2,200 feet to the north and occurred on a continuous, inclined curve. In addition to limited sight distance for descending traffic (southbound), these rock slopes had minimal offsets and no ditch depth resulting in negligible catchment area.

Rock slopes in this section ranged in height from 20 to 80 feet with relatively gentle back slopes. Of great importance in this section was the relative lack of talus on the back slope in proximity to the rock slope. Back slope conditions facilitated the operation of equipment to conduct drilling and blasting, and did not present the problem of possible talus destabilization which strongly influenced the design of the northern section of the project.

The general agreement reached by Departmental Engineering Geologists of the design unit was that a 2 vertical on 1 horizontal recut of the rock (approximately matching the prevalent joint dips) combined with a 14 ft. wide by 4.5 ft. deep catchment ditch at the toe of slope, would produce a stable rock slope with greatly improved catchment and the added benefit of increased sight distance. (See figures 3 through 6).

A short transitional section in the middle of the project contained the most stable rock slopes. The roadway reaches its maximum elevation here and the rock slopes are at their minimum height. Rock slope stabilization design in this section addressed localized areas of unstable rock. Remediation methods were limited to scaling, rock bolt installation and removal of selected boulders from the back

Fig 3 2V on 1H presplit. Note coincidence of joint surface and presplit angle.

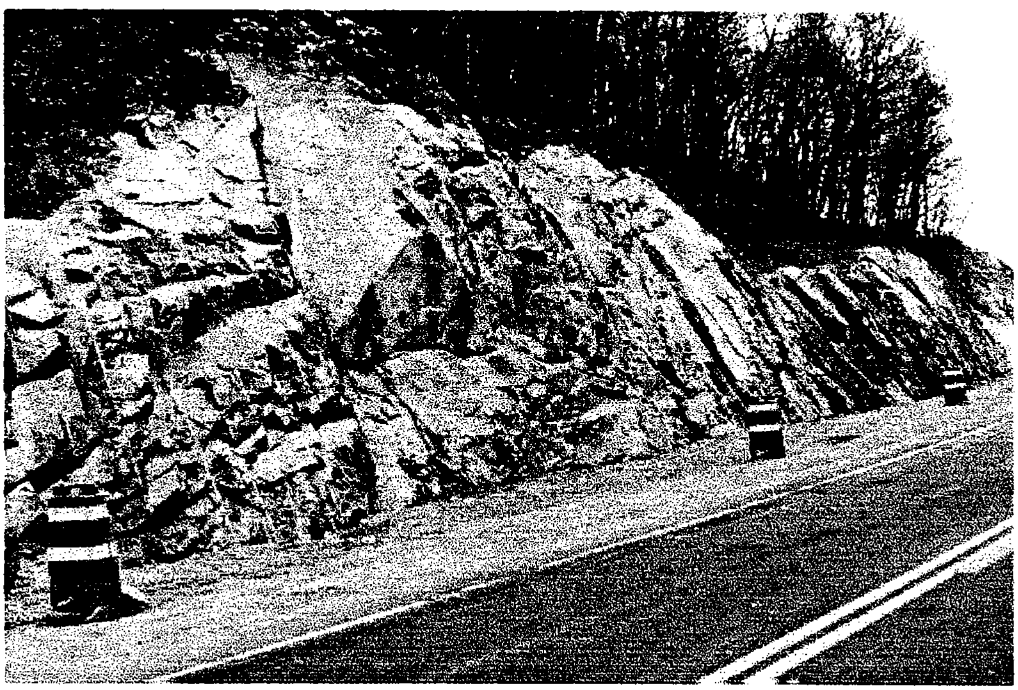

Fig 4 2V on 1H presplit. Five joint orientations are visible.

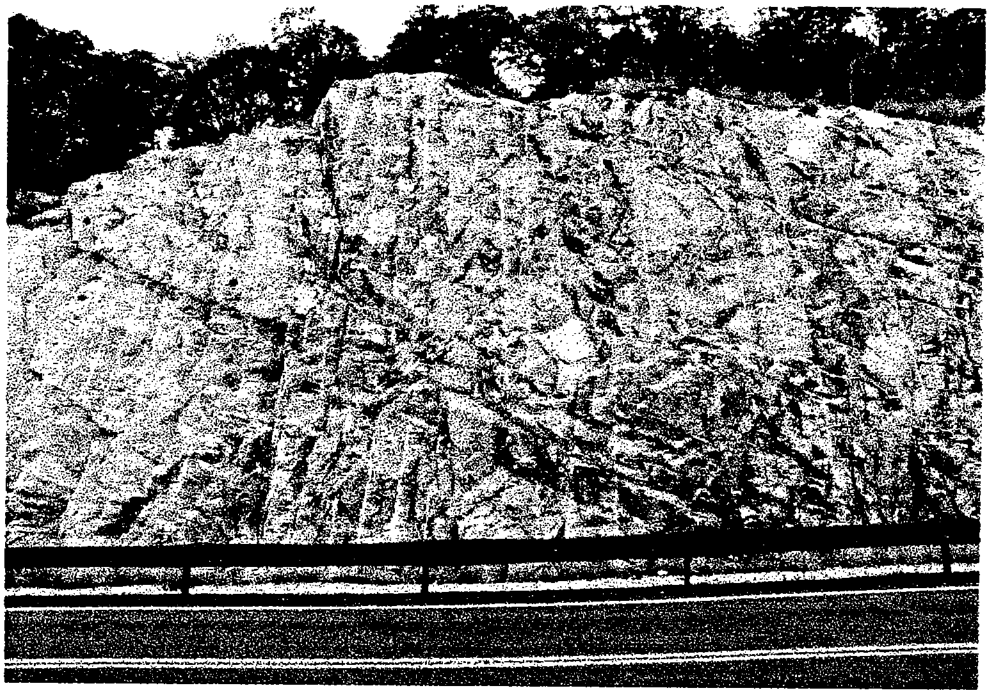

Fig 5 2V on 1H presplit. Foliation is visible sloping from upper left to lower right.

Fig 6 Phylonitic shear zone in center of photograph.

slope. In addition, the amount of talus accumulated on the back slope began to increase dramatically in this area. The potential for roll out of talus from the back slope prompted the inclusion of rock catchment fences at selected areas along the top of the rock slopes.

The rock slope design of the northern section of the project proved to be the most problematic. The roadway is sinuous and on a steep grade with extremely limited sight distance. Rock slopes here are up to 55 feet high, with restricted catchment area and frequently occurring rockfalls. There were also three unreinforced rubble stone masonry retaining walls built in the 1930's, the southernmost of which had developed an open longitudinal crack. The rock in this section was highly jointed and contained many large unstable blocks. When initially considering the rock slope stabilization design, it was felt that these rock slopes were prime candidates for recutting. Subsequent inspection of back slope conditions above the rock faces revealed a steep slope composed of a large accumulation of talus lying at the angle of repose and stretching several hundred feet up Dunderberg Mtn. The concern over destabilization of the talus slope necessitated modification in the design approach to the rock slope stabilization. After several site visits and much discussion, it was decided that the benefit of recutting the rock slopes was outweighed by the potential destabilization of the talus on the back slope. The proposed rock slope design therefore relied heavily upon scaling, bolting, construction of buttresses, and rock catchment fences. Due to the heavy jointing of the rock, extensive use of rock bolts was proposed to minimize the "chasing" of loose, unstable rock into the slope and driving up project costs. In some areas, rock had to be removed via scaling and the key rocks anchored in place with rock bolts to provide a stable rock slope.

Rock catchment fences were designed for use in two applications. Light duty rock catchment fences were intended for use on the back slope to minimize the possibility of talus reaching the slope face and creating a fall. Medium duty rock catchment fences were designed for use on the rock slope face and at the toe of slope to keep rock falls generated on the rock slope or the talus slope from reaching the road. At most locations of the northern section, multiple stabilization methods were used in conjunction to provide a stable final rock slope.

In addition, new buttresses were designed to fill in between and extend south from the existing retaining walls. The rock in this area was particularly prone to causing problems if extensive rock scaling was attempted since it could result in destabilizing the rock and talus higher up the slope. The new buttresses were designed to include a rock facing to blend in with the existing rubble stone masonry buttresses.

The crack in the southern preexisting wall was monitored by installing tilt plates and taking periodic measurements to track movement. After measurements taken over several months indicated negligible movement and Departmental Engineers determined that the wall was sound, it was decided not to disturb the wall, and to leave the crack open to promote drainage.

CONSTRUCTION

In the interest of simplicity, the discussion of construction of the project will be divided into separate north and south sections as previously described.

SOUTH

Presplit blasting in this area was complicated by:

- Steep irregular terrain consisting in large part of bare rock on which the drillers had to maneuver. Air track drills were commonly restrained by winch cables.
- Proximity of houses and rail line: Rte 9W in this area is built on a talus pile that lies at the toe of Dunderberg Mt. The ground surface descends from the roadway to the bank of the Hudson River at the angle of repose. A major rail freight line built on the river bank and a number of houses downhill of the rock face were completely vulnerable to any rock which crossed the road. NYSDOT arranged for the presence of a railroad flagman during all blasting operations which could potentially impact the railroad.
- Time constraints:
  Rte 9W is the principal traffic corridor in the area, providing access for commercial traffic from the south to the Bear Mt. Bridge which crosses the Hudson River. Residents of the area were also concerned about the potential for unscheduled traffic interruptions resulting from construction activity. In order to minimize traffic disruptions, NYS DOT implemented road closure on Tuesday and Thursday between 9:00 A.M. and 5:00 P.M. to facilitate blasting and removal of excavated rock. To ensure the timely completion of the rock excavation, the contract included liquidated damages of $1,000 per day for any blasting activities which extended beyond the specified contract completion date.

- Limited blast catchment area.
  Rte 9W in this area is a two-lane highway with a climbing lane. Prior to the execution of this project, the offset between the edge of pavement and the rock face was minimal. Limited catchment area was available to contain the shot rock. In order to prevent damage to houses and the railroad below the rock face and to facilitate the clearing of the roadway within the designated road closure window, the blasts were designed to direct the shot parallel to the roadway. All blast plans were reviewed by a departmental Engineering Geologist prior to loading. Each blast was videotaped and subsequently reviewed by the blaster and the Engineering Geologist to determine if revisions were necessary.

BLAST DESIGN

Two and one half inch diameter presplit holes were drilled on three ft. centers. Three inch production holes were drilled on a six ft. x six ft. pattern. The row of production holes closest to the presplit line were battered at the presplit angle. Presplit holes were loaded with a maximum 2 lb. base charge held below finished grade, and a continuous column charge of 0.25 lb. per ft. presplit dynamite and 25 grain detonating cord. The column charge was brought to three ft. below top of rock and stemmed with 1A crushed stone. A powder factor of 0.75 to 1.0 lb. per cu.yd. and a maximum explosive charge of 50 lb. per delay period were maintained throughout the blasting operations. Explosives were detonated with electric 25 ms blasting caps used in conjunction with a sequential timer. Figure 7 schematically depicts a typical ignition pattern.

Drilling and blasting proceeded steadily over a distance of 2200 lineal feet of roadway in which the rock face varied in height from 20 to 100 feet, averaging 30 feet in height. Two to three blasts of 1000 to 2000 cu.yd. were shot each week. Extensions of the road closure window to accommodate clearing the roadway after large blasts were occasionally necessary. With the exception of one incident, in which a large rock traveled across Rte 9W and down the embankment, damaging the railway, the blasting was completed without injury or property damage.

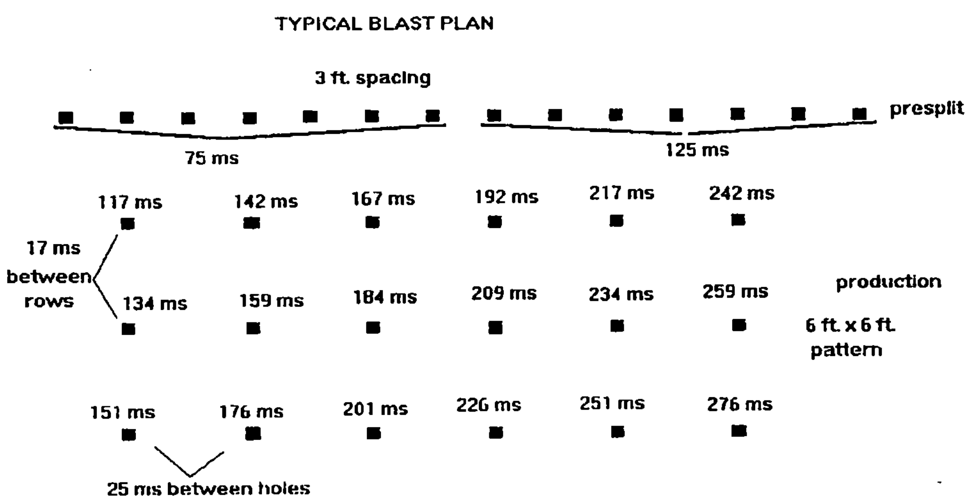

Figure 7 Typical ignition pattern for a presplit blast.

NORTH

Construction in this area was carried out in five distinct steps:

1. Given the prime directive to leave the talus undisturbed, the rock face was scaled by means of hand labor. Workers either climbed the slopes and worked while tied off on safety ropes, or worked out of a man lift. Workers were directed to remove loose rock from the face, and to avoid disturbing the vegetative mat at the rock face/back slope intersection. At two locations the unstable rock was too large to remove by hand. These areas were scaled mechanically and with blasting.

2. After scaling had been completed, 1 ¼ inch resin rock bolts were installed to stabilize the rock face and key blocks at the toe of the talus field. Typically, the rock bolts were tensioned to 70 kips, but in some cases they were installed as untensioned "shear pins" to hold key detached blocks in place.

3. A light duty rock catchment fence was installed on the back slope as close to the top of the rock face as possible. The fence was designed utilizing common construction components which were both readily available, and covered by existing departmental material specifications. The fence was designed to be flexible and to attenuate the force of falling or rolling rock by distributing the force across a broad area of the fence. The fence was constructed with rebar posts and J-hook tie back anchors spaced on 8 ft. centers, with three 3/4 inch guiderail cable longitudinal cables. Tie backs from each post to J-hook anchor were also 3/4 inch cable. Chain link fencing was then attached to the posts and longitudinal cables. Workers were instructed to avoid unnecessary disruption of the vegetative mat. Irregularities in the ground surface were addressed either by digging into high areas and/or notching the bottom of the fence, or in the case of low areas, adding a section of chain link fencing at the bottom to ensure that the bottom of the fence was tight to the ground surface. After the fence installation was completed, rocks which had been disturbed on the road side of the fence were removed. (See figures 8 through 10.)

4. In two locations large undermined blocks were supported with reinforced concrete buttresses. These were built in four stages. 1) In order to provide a safe work area, the loose blocks were secured with 1 ¼ inch resin rock bolts.2) A reinforcing mat was constructed and anchored to the rock face by grouting rebar into holes drilled into the rock face on 2 ft. centers. 3) Drainage was addressed by inserting 2 inch PVC pipe into the form at 5 ft. centers two feet above finish grade, as well as into any rebar anchor hole which produced water during drilling, or any large crack. These pipes were brought flush to the form. 4) Forms were erected and concrete poured. (See figures 11 through 15.)

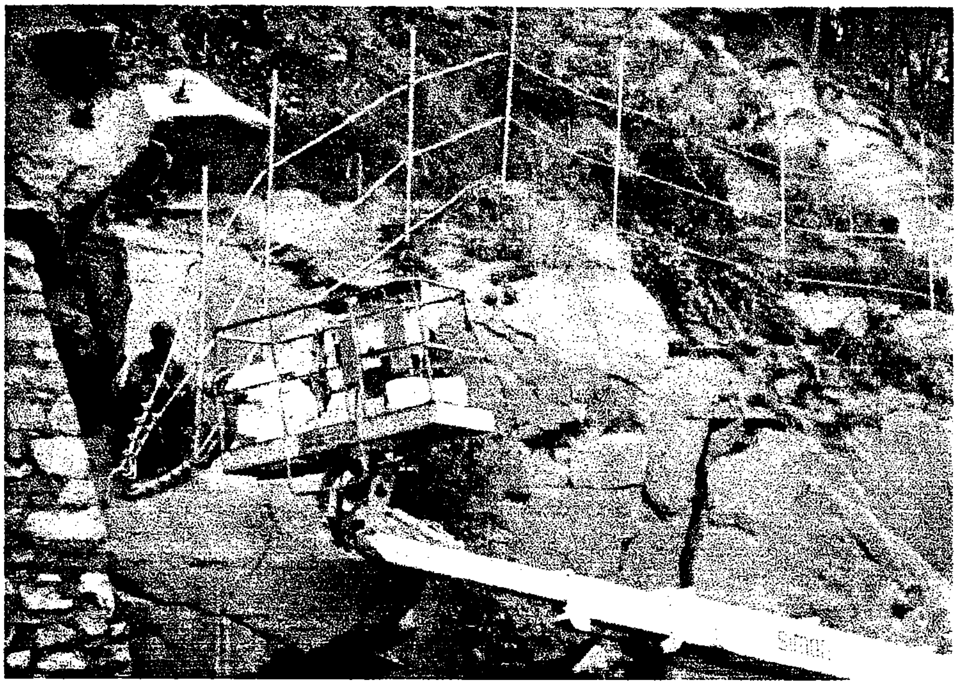

Fig 8 Light impact rock catchment fence installation.

Fig 9 Light impact rock catchment fence installed above a scaled and rock bolted slope. Note mafic dike intruding country rock from the left margin of the photograph.

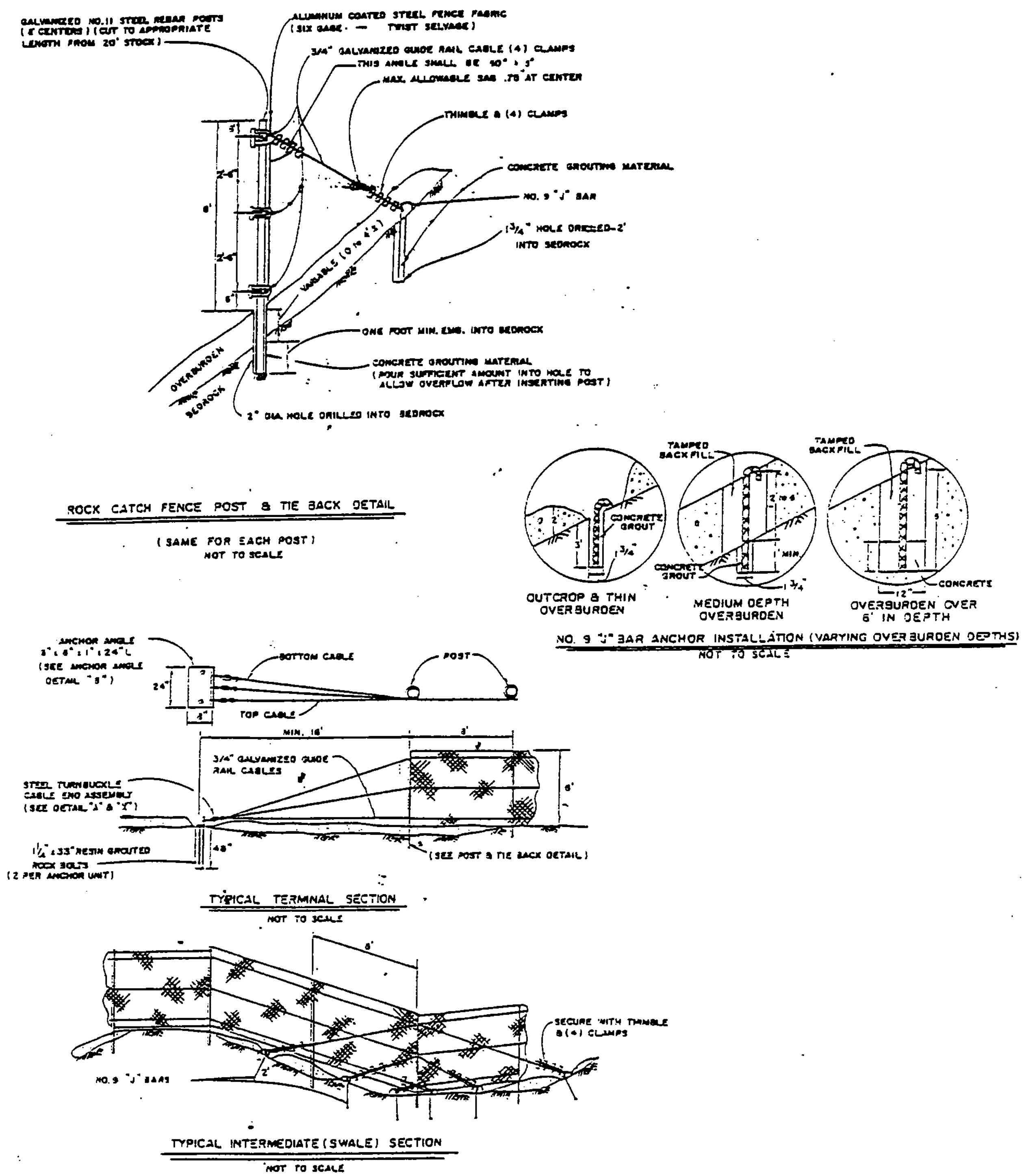

Figure 10 Light duty rock catchment fence

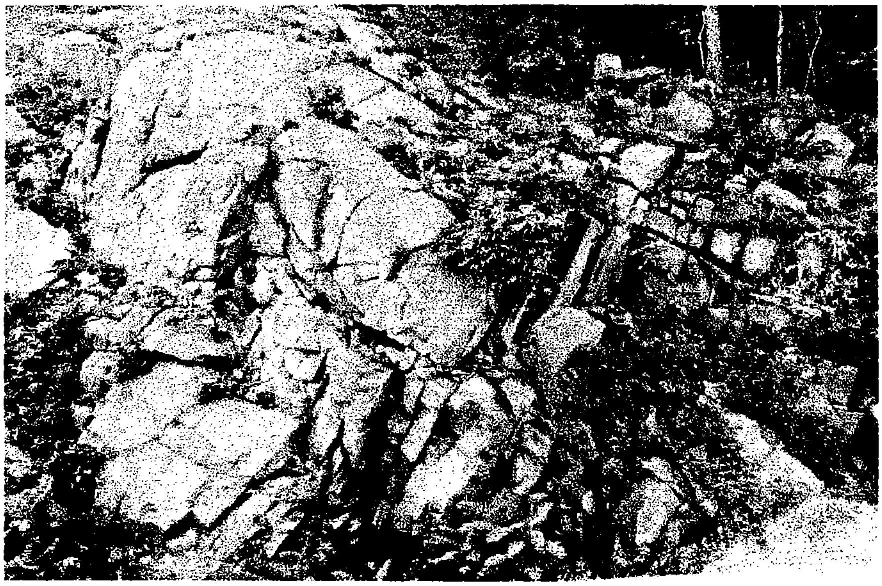

Figure 11 Undermined block which was supported by the buttress pictured in figs 12 thru 14.

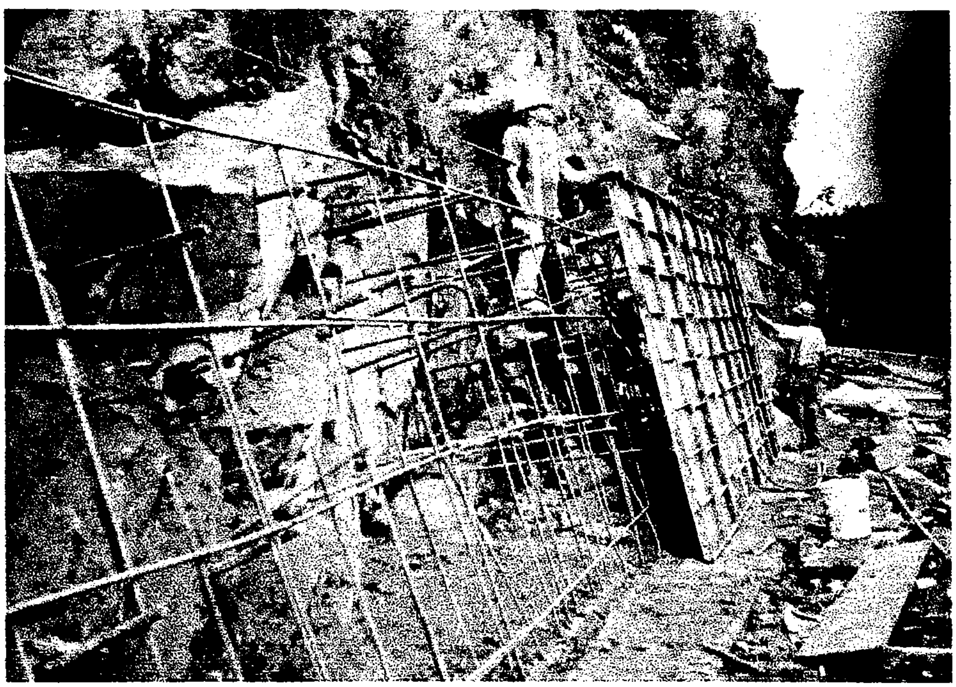

Figure 12 Reinforcement and concrete forms being placed for buttress and adjoining parapet wall.

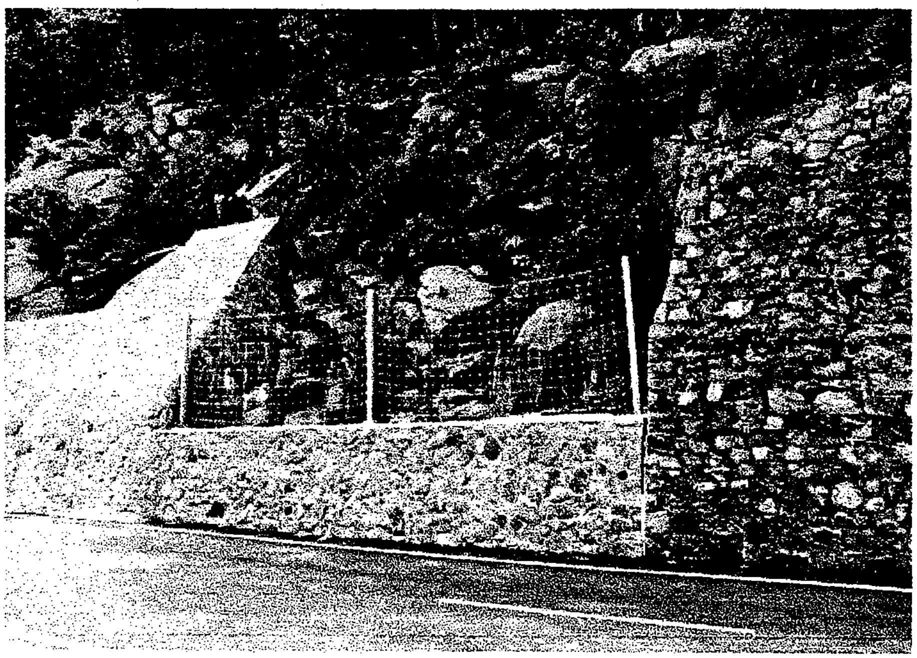

Fig 13
Completed buttress and parapet wall with rock catchment fence. In Fig 13 note rock bolts installed as “shear pins” in loose blocks above the fence. The wall at the right side of both photographs dates from the 1930's.

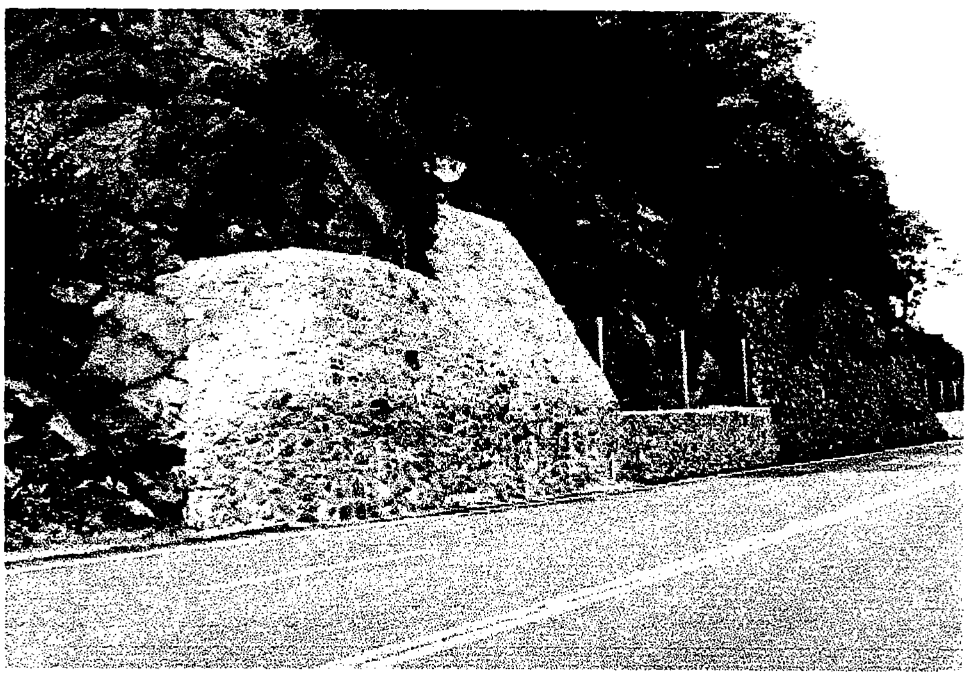

Fig 14

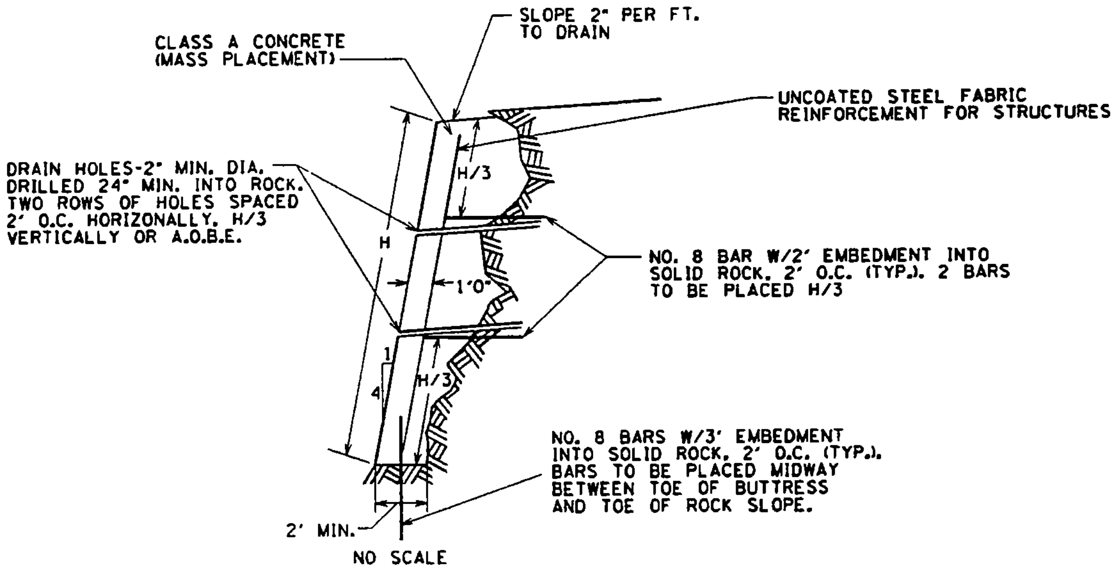

Figure 15 Typical Cross-Section of Concrete Support Buttress

5. At two locations the talus rested directly upon zones of weathered rock and soil which were themselves sources of rockfall. These areas could not be disturbed due to the proximity of the talus pile. Computer rock fall simulations were utilized to determine the appropriate height of a rock catchment fence. To provide the required fence height and to stabilize the toe of the weathered rock zone, a concrete parapet was constructed using similar techniques as described for the buttresses, and a medium impact rock catchment fence was built upon it (Figures 13 and 14).

AESTHETIC CONCERNS

Dunderberg Mt. is part of the Palisades Interstate Park system, a network of undeveloped forest straddling the N.Y. - N.J. border. The area is crisscrossed with hiking trails and provides a woodland haven for the residents of the nearby urban areas. Environmental and aesthetic concerns are significant. In an effort to mitigate the visual impact of the various rock stabilization measures, the NYSDOT chose a black vinyl coating for all chain link mesh, and opted to face the concrete buttresses with a veneer of local stone. The results were well received. The black chain link mesh effectively camouflaged the long runs of fencing. The stone veneer on the buttresses proved to be architecturally attractive and compatible with the style of the original stone walls.

Summary

The Rte. 9W - Dunderberg Mountain project presented many unique situations. Construction of stable rock slope configurations had to be implemented in such a way as to maintain the scenic integrity of the project site while accounting for the complex geologic conditions. This entailed using many of the remediation methods in the rock slope stabilization repertoire including: presplit blasting, rock catchment fence installation, rock bolting, scaling, and construction of buttresses and parapet walls.

Construction of presplit rock slopes in the southern area dramatically increased sight distance and catchment area along a steep twisting section of road. The presplit slope angle coincided with the dominant structural features of the rock and resulted in stable rock faces with minimal extra excavation. The multiple joint orientations have resulted in small scale fracturing and spalling of the finished presplit faces. This condition is unavoidable and its effect appears to be limited to the accumulation of small rock fragments in the catchment ditch.

The fresh rock surfaces on these slopes stand out. However, large exposed rock faces previously existed in this section. As the surface of the newly exposed rock cuts weather and age, the color will return to the same as existed on the pre-project rock cuts, resulting in minimal change in the long term visual impact of this section.

Design and construction of the northern part of the project was guided by the principle of leaving the talus undisturbed. This removed the presplit option and resulted in reduced opportunities for expanded catchment area or sight distance improvement. Given this limitation, construction in the northern section was a success insofar as it eliminated chronic rock falls and stabilized areas of potential rock slides.

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2. Ratcliffe, N.M., Brittle Faults (Ramapo Fault) and Phylonitic Ductile Shear Zones in the Basement Rocks of the Ramapo Seismic Zones of New York and New Jersey, and Their Relationship to Current Seismicity, _In_ Manspeizer,W.ED., Field Studies of New Jersey Geology and Guide to Field Trips: 52nd Annual Meeting of the New York State Geological Association, 1980

GEOLOGIC INVESTIGATION OF THE GORDON CANYON SLIDE

State Route 260, Payson Show-Low Highway
Gila County, Arizona

Nick M. Priznar (Geologist, Arizona Department of Transportation, Phoenix AZ)
Kenneth M. Euge R.G. (Principal Geologist, Geological Consultants, Phoenix AZ)

BACKGROUND

Location

The Gordon Canyon Slide is located on the north side of State Route 260, in the vicinity of milepost 280, which is approximately 100 miles northeast of Phoenix, Arizona. The site is situated on the south-facing slopes below the Mogollon Rim, a northwest-southeast trending escarpment that bisects Arizona. The escarpment is, a physiographic remnant of a Mesozoic highland (Stokes 1960) that marks the boundary between the Colorado Plateau Province and the Central Highlands Transition Zone (Figure 1).

Climate and Vegetation

The project area is quite luxuriant in vegetation. Considerable stands of Ponderosa pine, Juniper and Oak dominate the regional landscape. Normal annual precipitation of 24-28 inches per year undoubtedly contributes to the vegetative diversity of the region. Approximately 55 percent of the annual precipitation occurs between the months of October and March. Winters are usually mild with patchy snow from December to March, which result in a mean annual snow accumulation of 12 inches (Figure 2). The lowest temperatures are recorded in January with daily averages that range from 15 to 53 degrees Fahrenheit. The highest temperatures are recorded in August with daily averages that range from 50 to 89 degrees Fahrenheit.

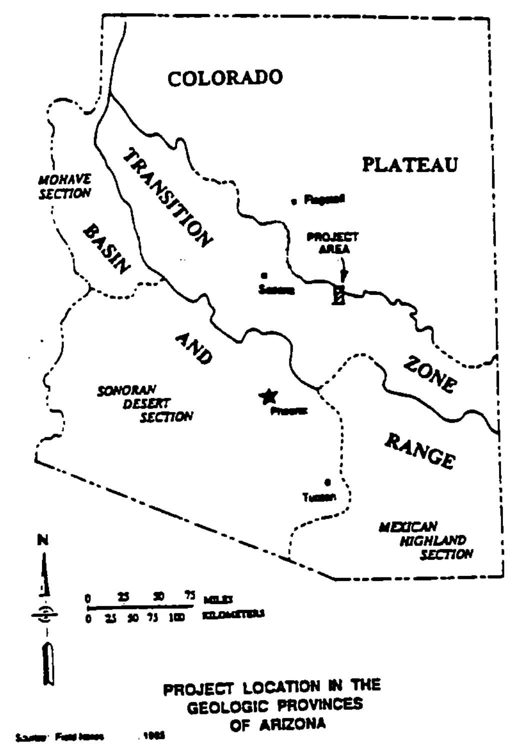


Figure 1

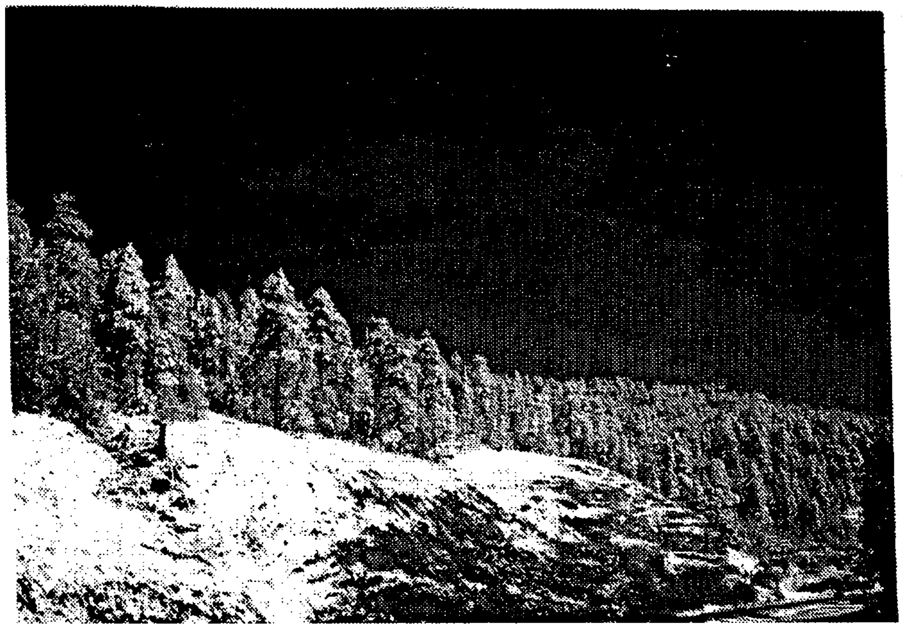
Highway Corridor and Geometry

As late as 1982, State Route SR-260 in the project area was a sinuous, steep grade, 30-foot-wide, low speed, rural, two lane highway, that snaked up the face of the Mogollon Rim escarpment. It essentially consisted of rough hewed hillside cuts and poorly drained, deep, random fill embankments.

In 1983 an embankment landslide occurred due to saturation of the fill caused by seepage from springs buried by the fill in the vicinity of milepost 280. The slope failure resulted in closure of the highway. Other embankment failures included shoulder sloughing, general fill settlement that required extensive maintenance.

Throughout the late 1980's extensive reconstruction of the roadway alignment occurred. In the vicinity of the present project, the random fill was removed and replaced with a new four lane paved section and bridge. The roadway alignment was also straightened and widened by excavating into the natural cliff face escarpment adjacent to the roadway. Presently the existing highway corridor consists of a modern four lane highway which connects the two communities of Christopher Creek and Herber, Arizona.

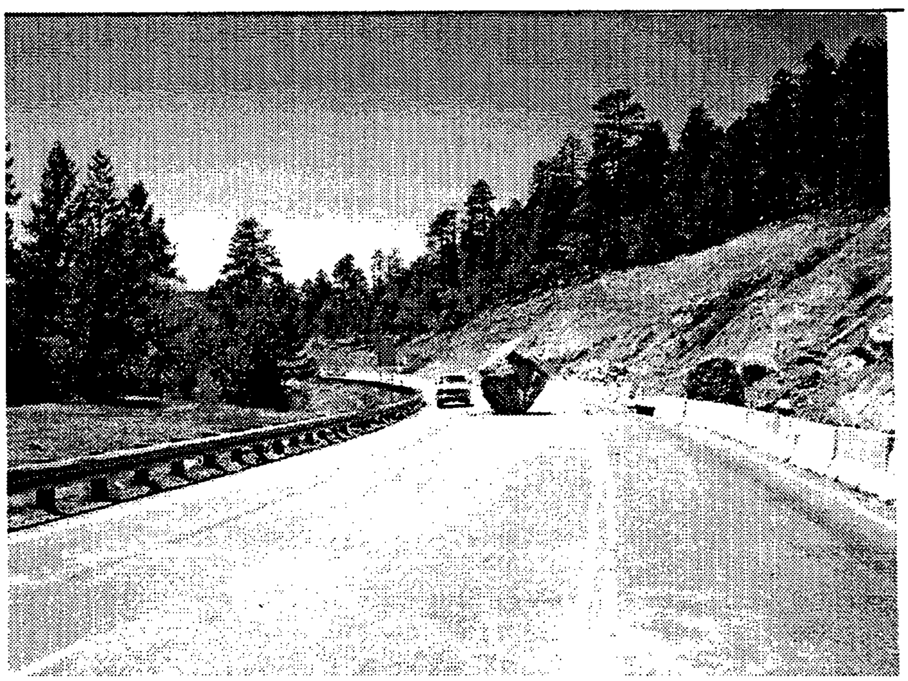
Project History



The new roadway cut for the west approach to the Gordon Canyon Bridge was completed in 1988 with a slope ratio of 3/4 to 1 (horizontal to vertical). The cut exposed 30 to 35 feet of nearly horizontally bedded light red to orange siltstone and mudstone with occasional greenish mudstone of the Supai formation.

Figure 2: These moderately well cemented sedimants are overlain by a well bedded, blocky limestone outcrop referred to as the Fort Apache member of the Supai formation. Immediately overlaying these rock units is a colluvium deposit, 70 feet thick, consisting of large boulders of limestone and calcareous sandstone in a matrix of silty sandy clay.

Figure 3



In the winter of 1991, a failure occurred within the upper portion of the cut slope, when the colluvium became saturated and began flowing downslope, cascading over the stable, underlying siltstone and mudstone cut face. This material in the 20 ft wide ditch at toe of the cut slope (Smith 1992) partially closing the four-lane highway.

In March 1992, additional slope movement occurred with large scale creep transporting large qualities of "mud, trees, and boulders" downslope filling the cut ditches and spilling 12 to-20 foot diameter boulders across the highway (Figure 3).

Extreme wet winter conditions in the following year resulted in statewide flooding. In the project area heavy winter rains and an accompanying snow melt infiltrated into the slopes, increased the hydrostatic loads, and triggered numerous landslides along the highway right of way. In the Gordon Canyon area all four lanes of highway became blocked with landslide debris. These conditions resulted in a temporary closure of SR-260.

In May of 1993, intensive geological and geotechnical studies were initiated by ADOT to investigate the existing landslide conditions and to develop design alternatives to mitigate problems associated with this unstable slope section of SR-260. An interesting aspect of these studies was the ADOT approach to utilized geophysics as a means to focus exploration efforts at the site. Electrical resistivity techniques were employed to model the site stratigraphy and groundwater conditions of the site which were believed to be the controlling mechanisms of the landslide at Gordon Canyon.

GEOLOGIC INVESTIGATION

PREVIOUS STUDIES

Several geological studies have been made by various researchers to understand the stratigraphic relationships within this portion of the Colorado Plateau/Mogollon Rim area (Sattershwaith, 1951; Winters, 1963) and in particular, the depositional environments of the Supai Formation and its Fort Apache limestone member (Frazier, 1961; Gerrard, 1964).

In 1984, the Arizona Department of Transportation (ADOT) contracted geological and geotechnical services to develop criteria to redesign highway embankments, for the planned widening of State Route 260.(Dames & Moore, 1984). Supplemental to the Dames and Moore study, additional engineering design studies were contracted by ADOT to STS D'Appolonia Ltd. (1986) to develop recommendations to mitigate slope stability problems in the vicinity of Gordon Canyon, and in adjoining fill embankments. All of the previous studies were carefully evaluated to develop a reasonable geological model of the site and its stratigraphic and hydrogeological relationships.

FIELD INVESTIGATIONS

Geologic investigation utilized a variety of techniques to identify critical structural relationships and subsurface conditions at the Gordon Canyon Slide. Limited direct observation of the exposed bedrock slope was possible and geologic mapping was conducted in this area. The landslide affected, colluvial slope prevented direct access to the slide mass. The landslide material obscured the three dimensional nature of the area, and more aggressive investigation techniques had to be undertaken to directly examine the landslide.

Geophysical Technique

Subsurface conditions in the landslide area were initially evaluated with the geophysical technique, of electrical resistivity. A resistivity survey was conducted to delineate the depth of the colluvium and to identify the position of water bearing zones within the slope and landslide mass. A surface resistivity sounding method was employed to measure in-situ electrical properties of the slope the using a Schlumberger electrode array. Three electrical resistivity sounding sites were carefully selected to model the geologic section of the Gordon Canyon Slide area. The depth to bedrock was estimated to be about 100 feet based on a preliminary geologic model of the site. Therefore measurements were taken to a maximum spacing of 1,000 feet, with a total spread length of 2,000 feet.

The soundings resulted in the identification of five distinct geoelectric resistivity layers. Two of the layers were interpreted to be relatively dry colluvium. The third layer was interpreted as saturated colluvium. The fourth, a high resistivity layer, was interpreted to represent limestone. The fifth layer exhibited a low resistivity and was interpreted to represent the underlying saturated mudstone/claystone red beds that are exposed in the road cuts (Figure 4 & 5).

Geologic Mapping

Limited direct observation of the exposed bedrock slope was possible and geologic mapping was conducted in the area that was not disturbed by the land slide mass. Color aerial photographs and a detailed topographic map was used as the base of the geologic map. Benchmark elevation and stations were established through the use of global positioning satellite survey (GPS). Bedding planes were traced along the base of the road cuts to identify the stable outcrops of the exposed formations. Seeps and springs were also mapped and correlated to the stratigraphic features in which they occurred.

Geologic structural data, including orientations of both bedding planes and joint patterns, were collected during this operation in anticipation of performing a kinematic analysis of the existing bedrock conditions.

Core Drilling Program

Combining the results of the geophysical study with the geologic map, facilitated the design of a core drilling program to further refine the subsurface interpretation and to gather geotechnical samples for laboratory testing. The Gordon Canyon Slide is within the Tonto National Forest and conventional access to the site was not permitted. The entire drilling investigation was conducted with equipment that was manually transported up the slopes.

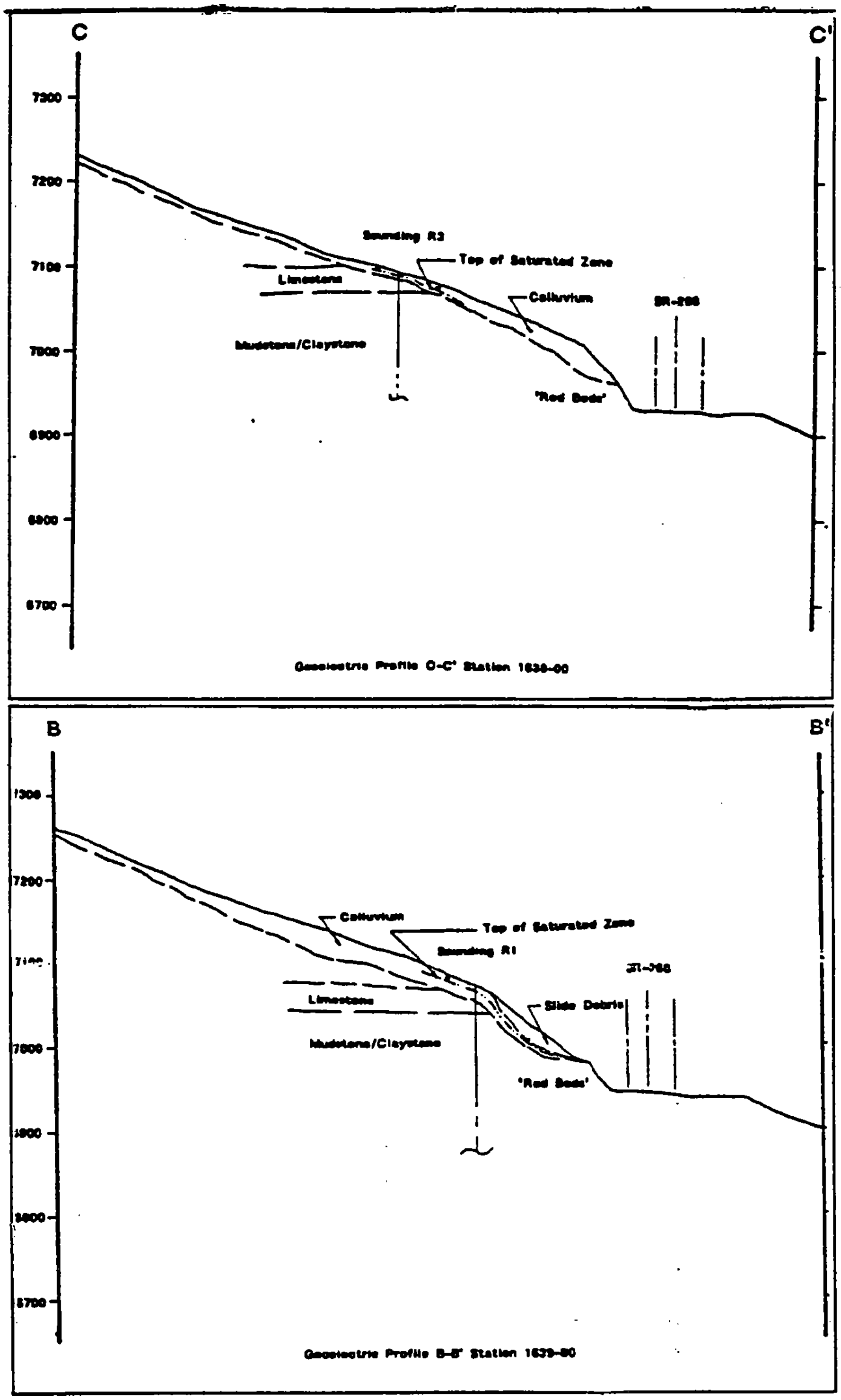
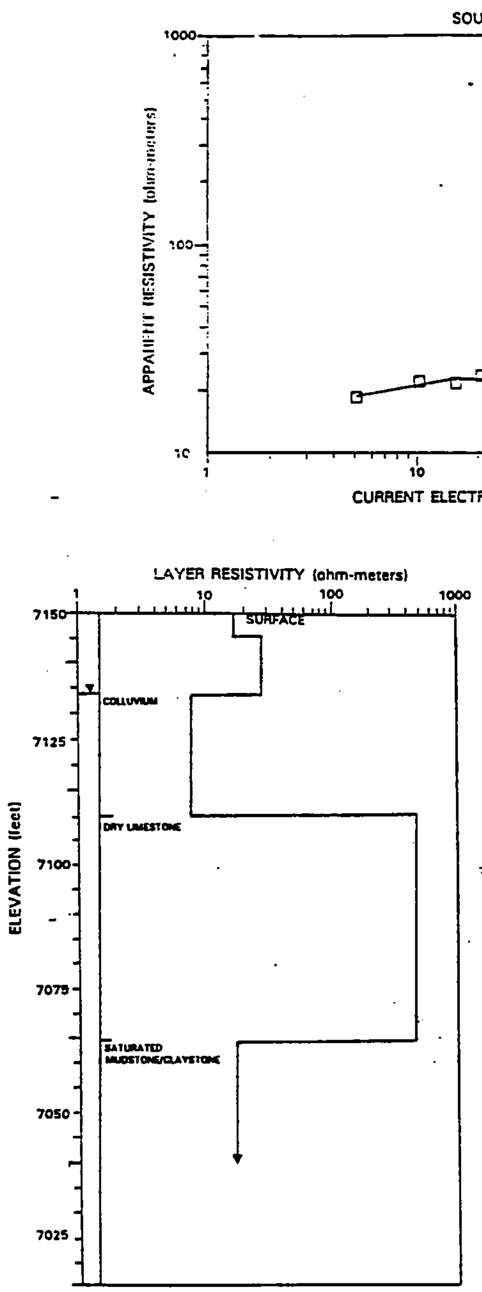


Figure 4

Aesthetic and natural conditions were preserved throughout the investigation by using a Winkie drill with environmentally innocuous drilling fluids.

Five rock core borings were drilled at sites deemed to have the best potential to produce optimum stratigraphic information and provide samples of the failed mass for laboratory testing. Each boring was cored with a dual-wall BW-

44 core barrel equipped with diamond set bits in accordance with ASTM D-2113. Drilling fluid consisted of water and biodegradable synthetic polymer. Detailed geologic logs were constructed from visual examination of the core obtained.



OPTIMUM GEOELECTRIC MODEL

LAYER RESISTIVITY (ohm-meters)	LAYER DEPTH (feet)	LAYER ELEVATION (feet)	ESTIMATED LAYER UNCERTAINTY (%)
15	0-4	7150-7146	10%
30	4-16	7146-7134	10%
8	16-39	7134-7111	20%
800	39-85	7111-7065	20%
20	BELOW 85	BELOW 7065	25%

Electrical Resistivity Survey
SR-260 @ Milepost 280

FIGURE 5
RESISTIVITY DATA AND INTERPRETED GEOELECTRIC MODEL
SOUNDING R2

Project No. 93-131-C May 1994

GEOLOGICAL CONSULTANTS

GEOLOGY OF THE GORDON CANYON SLIDE

Rock units within the Gordon Canyon slide area include clastic and non- clastic sedimentary rock units deposited during the later part of the Paleozoic Era. The Paleozoic formations include Permian age shallow marine deposits of the Upper Supai Formation overlain by Quaternary age colluvium and recent landslide debris (Figure 6).

Upper Supai Formation (Ps)

The Upper Supai Formation, including the Fort Apache member, is exposed in the road cut adjacent to the west bound lanes of SR-260 and in isolated outcrops within the cut. In the slide area, approximately 40 feet of the Upper Supai 'red beds' were exposed in the cut face. An undisturbed section of the Fort Apache member, 60 feet thick, overlie the red beds east of the slide area.

Supai Formation Red Beds (Psl and Pss)

A portion of the upper Supai Formation, stratigraphically below the Fort Apache member, consists of a dark reddish brown, dark orange brown and brown, intercalated sandstone, sandy siltstone, and shaley claystone and mudstone characteristic of clastic terrigenious sediments (Psl). Light greenish gray mudstone marker beds are also present near the base of the cut. The sedimentary units are variably cemented as indicated by alternating layers of soft zones separated by thin, calcareous ledge forming beds. The unit is thin bedded (less than one inch to one foot) and well stratified. Structurally, the unit is nearly flat-lying.

Conformably overlying the Fort Apache limestone member of the Supai Formation is a light red, greenish gray, and olive yellow siltstone and sandstone (Pss). Outcrops and large boulder-size fragments of the sandstone are quite prominent in the upslope areas stratigraphically above the Fort Apache Limestone. The unit is moderately well bedded to massive with indistinct bedding evident. The sandstone units are well indurated, well sorted, and weakly cemented, while the siltstone units are poorly cemented and non -calcareous. The contact is very prominent in the drill hole cores; however, at the ground surface the contact is indistinct due to weathering and colluvium cover.

Fort Apache Limestone Member (Psa)

The Fort Apache Member of the Upper Supai Formation consists of light gray, pinkish gray, and light brownish gray silty and sandy limestone and coarse crystalline limestone with thin clastic interbeds. The limestone contains some clastic sand-size particles indicating this unit was probably deposited in a shallow marine environment where clastic sediments were mixed with non-clastic limy sediments. This limestone unit is hard, well indurated and thin to thick (3 feet to 10 feet) bedded. Structurally, this member is nearly flat-lying and transitionally conformable with the Supai member stratigraphically above and below it.

The Fort Apache Limestone is also broken by sets of near vertical joints that are open as much as one foot at the surface. Typically this member forms resistant, blocky, steep slopes and cliffs that are easily recognized throughout the area.

Colluvium

Colluvial sediments are exposed in the upper portion of the cut slope and it also mantles portions of the natural slope above the top of the cut. The colluvium is composed of brown, orange brown to dark reddish brown, unconsolidated to poorly consolidated detrital sediments including sandy silt, sandy clay, and silty clay with gravel to boulder size fragments of sandstone and limestone. Discounting the presence of large fragments, the soil matrix is classified as silt (ML), clayey and silty sand (SC/SM), and silty clay (CL). The consistency of the colluvium ranges from loose to medium dense and stiff to very stiff. The thickness of this unit is estimated to be quite variable ranging from less than 10 feet up slope from the road cut to more than 70 feet of landslide-affected colluvium near the cut slope face.

Landslide Debris

The colluvium at this site incorporates the Gordon Canyon landslide debris. The slide mechanism appears to involve slump failure followed by an earthflow type slope failure of the water-saturated sediments. Failure was likely induced by the excavation and oversteepening of the cut slope which was partially saturated by groundwater underflow. Several areas upslope from the top of the cut show evidence of shallow soil creep, the slow imperceptible downslope movement of the regolith (soil) zone and saturation of the colluvium.

The landslide debris is composed of the same sediments making up the colluvium (Qco). However, due to the relative rapid downslope movement of the saturated soils, the soil structure/layering has been completely destroyed. Numerous large blocks of the Fort Apache limestone and sandstone from the overlying member of the Supai Formation are found throughout the landslide debris. The landslide soils are unconsolidated, locally loose, uncemented and poorly sorted. At the time of the initial site reconnaissance, the sediments were very moist with groundwater seeps issuing from the base of the slide mass.

Geologic Structure

Bedrock units exposed in the site vicinity are well bedded clastic and biochemical sedimentary rock. Primary bedding plane contacts are conspicuous within all units. Contact boundaries between adjacent beds range from smooth, to undulatory, to rough. Structure orientation data are presented on the site Geologic Map and the Geologic cross sections. The continuity of the strata and the uniform, flat-lying structure of the Supai 'red beds' was confirmed by continuously tracing bedding planes across the existing cut face below the landslide.

The bedrock structure is reflected in the bedding plane orientations measured at the site. Figure 7 is a stereographic dip vector plot of the bedding. The strike and dip of the beds define essentially flat-lying beds that exhibit a very low inclination angle. A contoured plot of dip pole concentrations (Figure 8) clearly depicts dominate dip inclination of the structure towards the south. The beds, although prominently daylighted by the SR-260 cut slope, are essentially flat-lying and therefore, do not appear to contribute to the Gordon Canyon Landslide.

Secondary joint structure also affect the rock mass primarily within the Fort Apache limestone member of the Supai Formation. At least two high-angle joints sets dip toward the east and west. The joints are relatively continuous forming large, interlocked orthogonal blocks.

Hydrogeology

Groundwater conditions at the Gordon Canyon Landslide were evaluated using geological mapping, resistivity soundings, and subsurface exploratory drilling. Depth to saturated zone(s) were initially interpreted from the resistivity soundings. The interpretations were either verified or modified by incorporating information derived from the direct surface and subsurface exploration of the landslide area. At the time of the geophysical investigation, the surface soils were very moist to very damp. Also the major drainage channels were flowing with small volume discharges or were exhibiting spring and seepage flows at various points along their thalweg. Numerous springs and seeps were observed issuing from the cut slop face. The most prominent flows were from a few isolated open joint discontinuities in the Fort Apache member and along the contact between the Fort Apache member and the underlying 'red beds' of the Upper Supai Formation. A prominent line of sweeps were also observed at or near the base of the slide debris. The discharge volume from these seeps is sufficient to wet the slope below seepage points. This condition has occurred in the slide area and is, in part, responsible for the unstable condition that presently exist at the Gordon Canyon site.

Interpretation of the resistivity data indicates shallow groundwater either within the colluvium or, where the colluvium is thin, at the contact between the colluvium and underlying Supai Formation bedrock. The bedrock is believed to function locally as a leaky aquitard which slows the vertical infiltration of water. This is also evident at the ground surface where springs are found in upslope areas that feed small ephemeral streams. Additional evidence can be observed in the cut slope face where seeps issue from the base of the earthflow debris at the bedrock contact.

SOILS AND LABORATORY TESTING

A laboratory testing program was performed to obtain representative geotechnical data to complete the engineering geologic evaluation of the cut and natural slopes and to perform the slope stability analyses. The laboratory work consisted of bulk rock density determinations, direct shear test, point load tests uniaxial and triaxial compression tests.

Strength tests were performed on core samples to determine shear strengths for rock discontinuities and representative bedrock materials. The tests were performed at the University of Arizona, Geomechanics Laboratory, Tucson, Arizona. The condition of the rock cores (such as friability, length, and discontinuities) required various test methods to obtain meaningful test results which would provide usable values of cohesion and internal friction for input to the stability analysis.

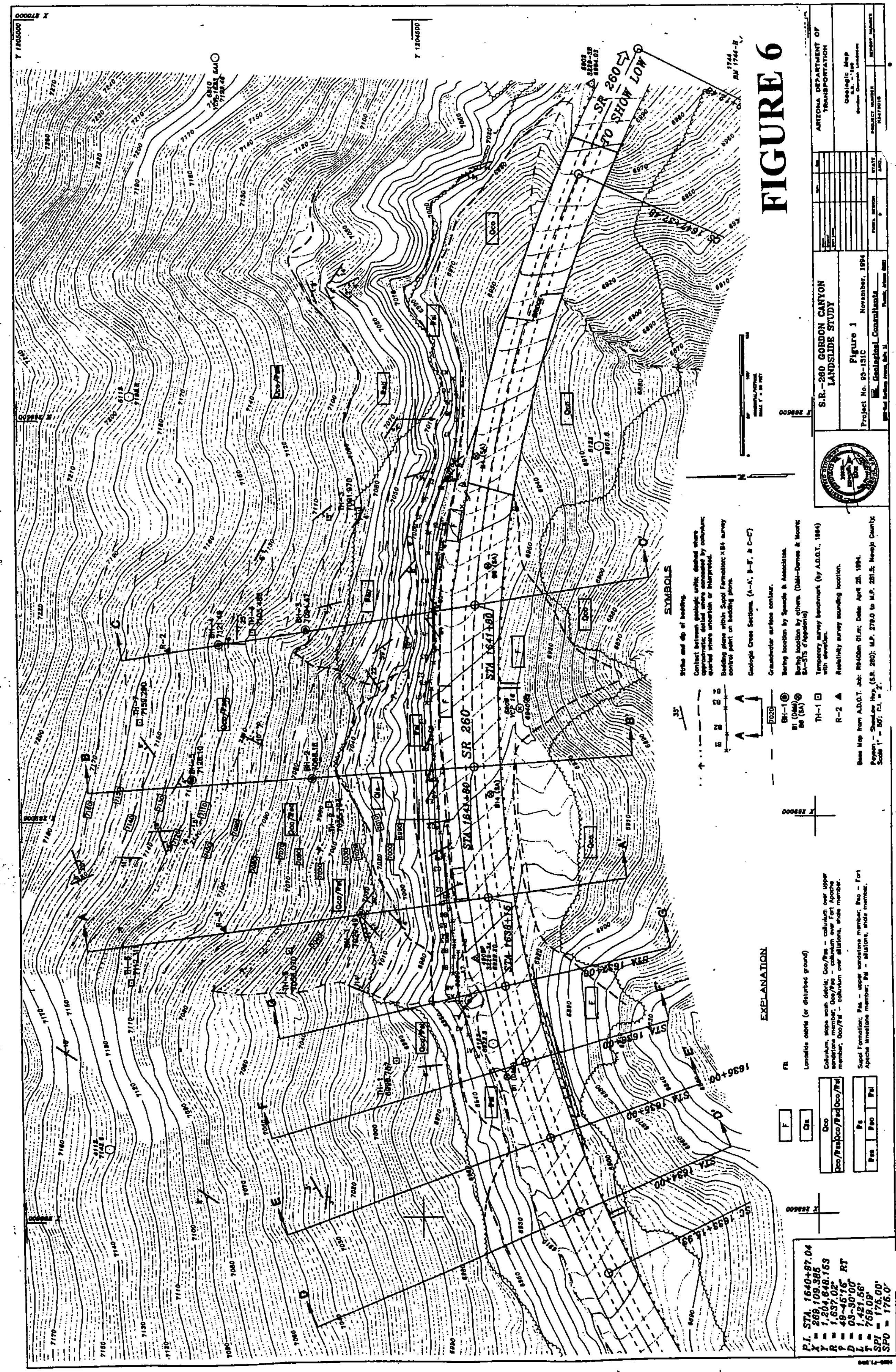

SR 260
TO SHOW LOW
EXPLANATION
SYMBOLS
S.R.-260 GORDON CANYON LANDSLIDE STUDY
Figure 1
Project No. 93-151C
November, 1994
Geological Consultants
ARIZONA DEPARTMENT OF TRANSPORTATION

FIGURE 6

ENGINEERING GEOLOGY OF THE GORDON CANYON LANDSLIDE

The Gordon Canyon landslide is well expressed at the ground surface. Several large fragments of the Supai Formation including blocks of sandstone and Fort Apache limestone are found within the unconsolidated slide debris below the low, near vertical headwall scarp. No tension cracks were observed upslope from the scrap; however, evidence of soil creep is clearly discernable in the upslope areas. The presence of creep affected soils provide evidence of nearly continuous downslope movement of the soils section in response to gravity and the effects of soil wetting by groundwater. No major tension cracks were observed upslope that might indicate that massive failures have abated at this site.

The landslide appears to have involved about one-half of the road cut. The failure plane, or slip surface, was not exposed. The failure plane along which the landslide initially propagated appears to have developed within colluvial deposits that mantle the Supai formation sedimentary bedrock to relatively shallow depths. The failure plane is believed to be a curvilinear surface along which rotational-type displacement occurred. It is conceivable that portions of the failure surface could include portions of the bedrock/colluvium contact. According to reports (ADOT, 1991 & 1993), the initial slump/rotational failure occurred following a period of intense precipitation forming a prominent headwall scarp at the breakaway line. During the slide movement the high water content of the slide mass provided the means to disaggregate the mass, converting the failure to an earthflow.

Aerial photographs, topographic maps, and geological mapping show that the bedrock structure in the vicinity of Mogollon Rim in this area controls the development of the geomorphic land surface. The bedrock units are generally flat-lying or very slightly dipping (less than 5°) (inclined) at very low angles towards the south. The undisturbed bedrock orientation in the natural environment tends to form a self-supporting, relatively stable natural slope. Bedding planes, although they are daylighted in the natural slopes, are not inclined out of slope at an adverse, unfavorable angle and therefore they are self-supported. Also, the bedding plane dip angles are flatter than the friction angle of the bedding planes which enhance their resistance to sliding.

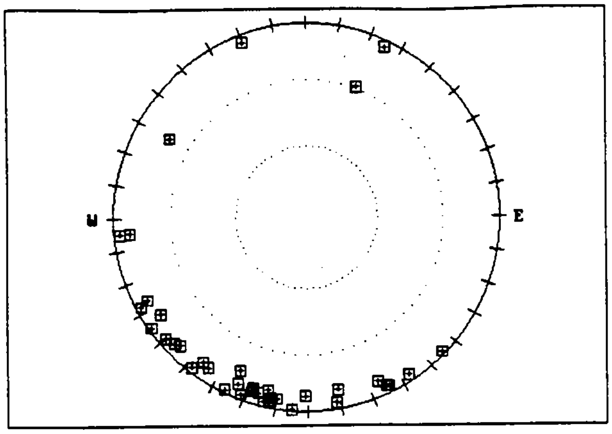
The realignment of SR-260, has had a substantial impact upon the natural geologic environment. The roadway realignment and associated excavations resulted in oversteepened cut slopes. The oversteepened cuts have had little, if any, impact on the horizontally stratified bedrock. However, because of the unconsolidated character of the colluvium and the fact that the cut slope angle exceeds the angle of repose, a potential adverse stability condition existed.. The introduction of groundwater resulted in excessive hydrostatic loads and wetting of the colluvium that induced the slope failure. The initial failure which occurred following extensive precipitation indicates water is a significant factor contributing to this landslide. The introduction of water, either naturally or artificially, caused failure to occur more rapidly by increasing driving forces and reducing shear strengths within the colluvium and along potential failure planes.

Engineering Geologic Analysis

The geological analysis, including stereographic and limit equilibrium analytical techniques, were used to evaluate natural and existing rock cut slope stability. The stereographic analysis indicates the presence of one principal concentration of rock discontinuities related to bedding planes within the Supai Formation. The concentrations are defined based upon the contoured plot of discontinuity dip vectors (Figure 8). The Gordon Canyon area cut slope faces toward the south at an azimuths ranging from 155° to 190°, averaging about 180°. In order to define the potential failure limits (critical zone) within bedrock units on the stenographic net, the cut slope angles (ranging from about 53° to 61°) in the slope face direction (180°) and a conservative discontinuity friction angle estimated to be about 20° is used to limit the conditions for each potential failure mode (plane (Figure 9), wedge block (Figure 10), and toppling failure (Figure 11)).

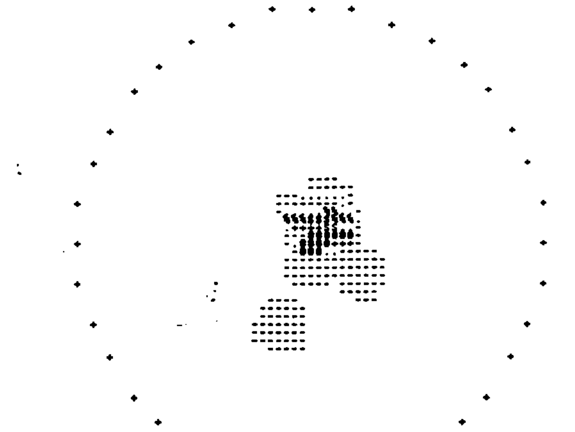
Limit Equilibrium Analysis of Existing Slope

The output from the subsurface exploration program provided direct information to calculate estimates of Factors of Safety (FS) against failure for the existing rock cut slope geometry and to analyze various alternatives to improve slope stability. The discontinuities kinematically determined to exhibit the greatest potential for plane and wedge-type failure are used as input parameters to calculate estimates of Factors of Safety (FS) against failure for the existing rock cut



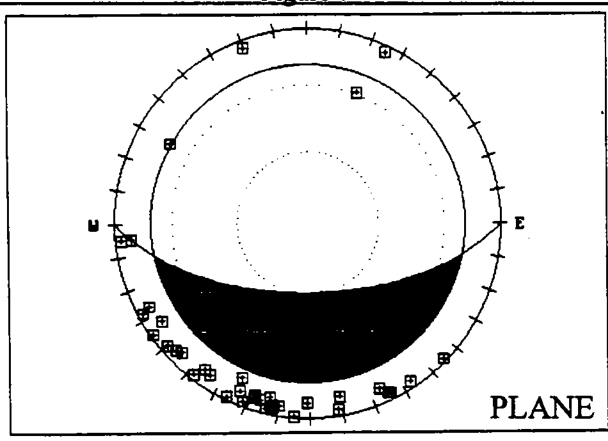
EQUAL-AREA STEREONET: A:\SR-260.DAT
Dip Vector Plot
Number of Stations = 38

Figure 7



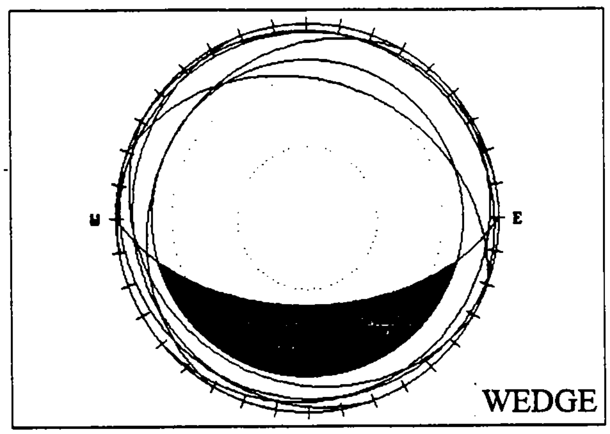
TITLE: SR-260 GORDON CANYON LANDSLIDE
CONTOURED PLOT OF DISCONTINUITY POLES

Figure 8



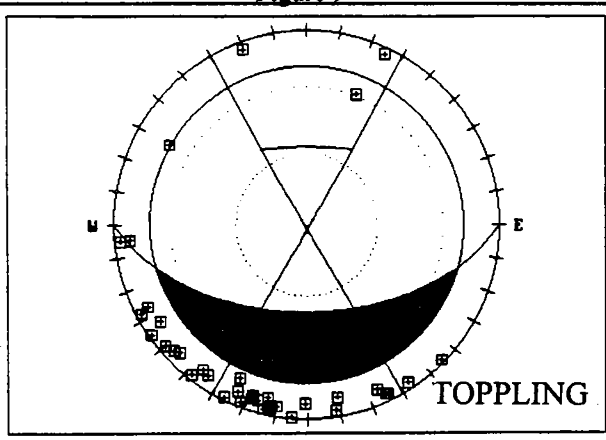
MARKLAND TEST PLOT: A:\SR-260.DAT
Friction Angle = 20 degrees
Slope dip direction = 180 degrees, Dip = 61 degrees
Number of Stations = 38

Figure 9



MARKLAND WEDGE ANALYSIS: A:\SR-260.GRT
Friction Angle = 20 degrees
Slope dip direction = 180 degrees, Dip = 53 degrees

Figure 10



MARKLAND TEST PLOT: A:\SR-260.DAT
Friction Angle = 20 degrees
Slope dip direction = 180 degrees, Dip = 53 degrees
Number of Stations = 38

Figure 11

slope geometry using the PLANE computer program (Watts, 1986). PLANE calculates the factor of safety assuming the bedrock block(s) fail translationally along a bedding plane surface daylighted at the toe of slope (Figure 12). The following rock properties were used in the PLANE analysis:

Rock unit weight = 130 pounds per cubic foot
Discontinuity friction angle = 20° (assumed for conservatism)
Cohesion = 0 pounds per square foot (assumed for conservatism)

The computer programs PC-STABL5M (FHWA, 1989) and XSTABLE were also used analyze the slope stability of the SR-260 road cut. Results are summarized in Table 1.

TABLE 1
CALCULATED FACTORS OF SAFETY FOR EXISTING ROCK CUT SLOPE
SR 260 - Gordon Canyon Landslide

PARAMETER	CROSS SECTION					
	AA		BB		CC	
Slope Face Height	82'		120'		100'	
Slope Face Direction	180°		180°		180°	
Cut Slope	53°		53°		61°	
Failure Surface	17°		17°		17°	
CASE ANALYZED:	FACTOR OF SAFETY					
W/ Tension Crack	PLANE	STABL5M	PLANE	STABL5M	PLANE	STABL5 M
Dry	1.74	1.22	1.74	1.21	1.74	1.55
100% Saturated	0.66	0.71	0.67	1.12	0.63	1.00
75% Saturated	0.93	--	0.94	--	0.91	--
50% Saturated	1.23	--	1.24	--	1.22	--

Analysis of Possible Mitigation Alternatives

The static stability of the existing rock slope is FS = 1.21, assuming a dry condition. The worst case FS is 0.63 for a 100 percent filled tension crack, wet slope conditions. Mitigation of future potential massive slope failures required corrective measures to improve the factor of safety to an acceptable level of FS = 1.3±0.05 (ADOT, 1993), accommodate the site stratigraphy and groundwater conditions. The proposed mitigating alternatives are directed at:

1. Reducing the driving loads imposed on the cut by laying back all or portions of the road cut slope to a flatter angle;
2. Remove the slide-affected materials;
3. Controlling or removing groundwater from the critical slope sections(s); and
4. Using biotechnical measures to improve surface stability and mitigate erosion.

Several mitigation alternatives were evaluated for the SR-260 Gordon Canyon Landslide Project. A summary of the stability analyses are summarized in Table 2. The mitigation alternative selected for this study reflect the refinement of the geological model provided by the subsurface exploration program. The proposed mitigation alternative (Figure 12) include:

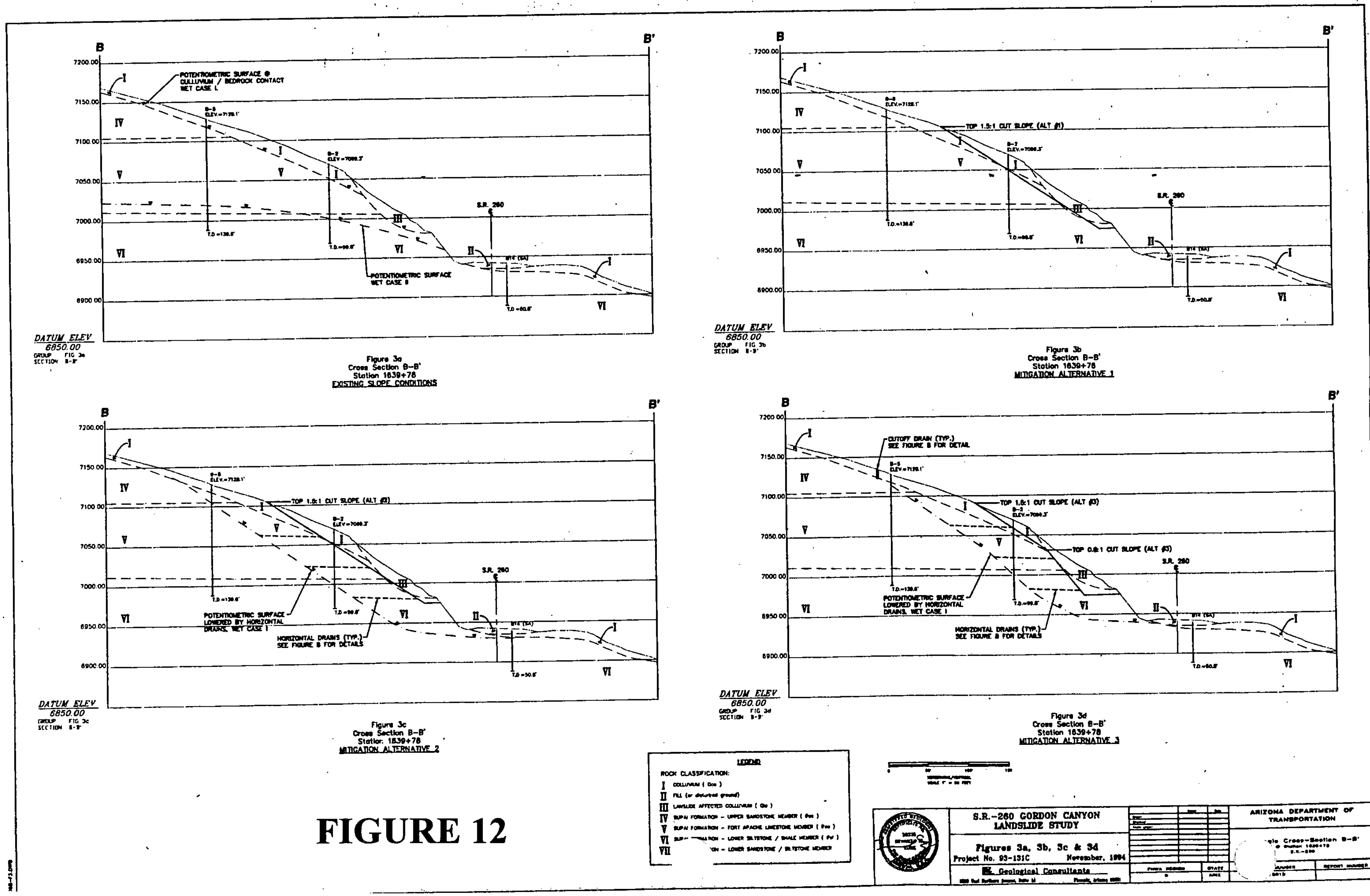


Figure 3a
Cross Section B-B'
Station 1639+78
EXISTING SLOPE CONDITIONS
Figure 3b
Cross Section B-B'
Station 1639+78
MITIGATION ALTERNATIVE 1
Figure 3c
Cross Section B-B'
Station 1639+78
MITIGATION ALTERNATIVE 2
Figure 3d
Cross Section B-B'
Station 1639+78
MITIGATION ALTERNATIVE 3
DATUM ELEV
6850.00
POTENTIOMETRIC SURFACE @ COLLUVIUM / BEDROCK CONTACT WET CASE L
POTENTIOMETRIC SURFACE WET CASE II
TOP 1.5:1 CUT SLOPE (ALT #1)
TOP 1.5:1 CUT SLOPE (ALT #2)
TOP 1.5:1 CUT SLOPE (ALT #3)
TOP 0.8:1 CUT SLOPE (ALT #3)
POTENTIOMETRIC SURFACE LOWERED BY HORIZONTAL DRAINS, WET CASE I
HORIZONTAL DRAINS (TYP.) SEE FIGURE 8 FOR DETAILS
CUTOFF DRAIN (TYP.) SEE FIGURE 8 FOR DETAIL
S.R. 260
LEGEND
ROCK CLASSIFICATION:
I COLLUVIUM (Qcs)
II FILL (or disturbed ground)
III LANDSLIDE AFFECTED COLLUVIUM (Qls)
IV SUPAI FORMATION - UPPER SANDSTONE MEMBER (Psu)
V SUPAI FORMATION - FORT APACHE LIMESTONE MEMBER (Pfa)
VI SUPAI FORMATION - LOWER SILTSTONE / SHALE MEMBER (Psl)
S.R.-260 GORDON CANYON LANDSLIDE STUDY
Figures 3a, 3b, 3c & 3d
Project No. 93-131C
November, 1994
Geological Consultants
ARIZONA DEPARTMENT OF TRANSPORTATION

FIGURE 12

TABLE 2
RESULTS OF SLOPE STABILITY ANALYSES PROPOSED MITIGATION ALTERNATIVES

SR 260 - GORDON CANYON LANDSLIDE

ALTER-NATIVE	LOWEST CALCULATED FACTOR OF SAFETY								
	AA			BB			CC		
	Dry	Wet I	Wet II	Dry	Wet I	Wet II	Dry	Wet I	Wet II
0	1.21	0.71	0.80	1.21	1.12	1.21	1.55 (1)	1.00	1.39
1	1.35	0.80	0.84	1.91	1.13	1.55	1.55 (1)	--	--
2	1.35	1.22	--	1.91	1.58	--	1.55 (1)	1.37 (2)	--
3	1.35	1.20	--	2.01	1.67	--	1.55 (1)	1.37 (3)	--

Notes: Wet Case I - Groundwaer @ Qco/bedrock contact or lowered by installed drains.
Wet Case II - Lower potentiometric groundwater surface.
(1) No layback.
(2) No layback; existing slope with horizontal drains
(3) No layback; existing slope with horizontal drains and upslope cutoff drain.

Alternative 0: Do nothing; existing conditions.

Alternative 1: Maintain existing slope in undisturbed bedrock, construct 20-foot wide bench at bedrock/colluviumcontact, and lay back bedrock/colluvium slope to 34° (or 1 1/2:1 H to V)).

Alternative 2: Maintain existing slope in undisturbed bedrock, construct 20-foot wide bench at bedrock/colluvium contact, lay back bedrock/colluvium slope to 34° (or 1 1/2:1 H to V), and install horizontal drains to control groundwater.

Alternative 3: Maintain existing slope in undisturbed bedrock, construct 20- to 50-foot wide bench at bedrock/colluvium contact, lay back exposed bedrock to 53°, lay back colluvium slope to 34° (or 1 1/2:1 H to V), and install cutoff drain up-slope to intercept groundwater at colluvium bedrock contact, install horizontal drains at selected levels to control groundwater seeps in bedrock.

The results of the slope stability analyses indicate mitigation measures can be employed at the SR-260 slope to stabilize the slope to an acceptable factor of safety of about 1.2.

Construction Consideration

Each alternative was evaluated to estimate the earthwork quantities and other materials involved in construction. A comparative summary of construction costs for each alternative was developed.

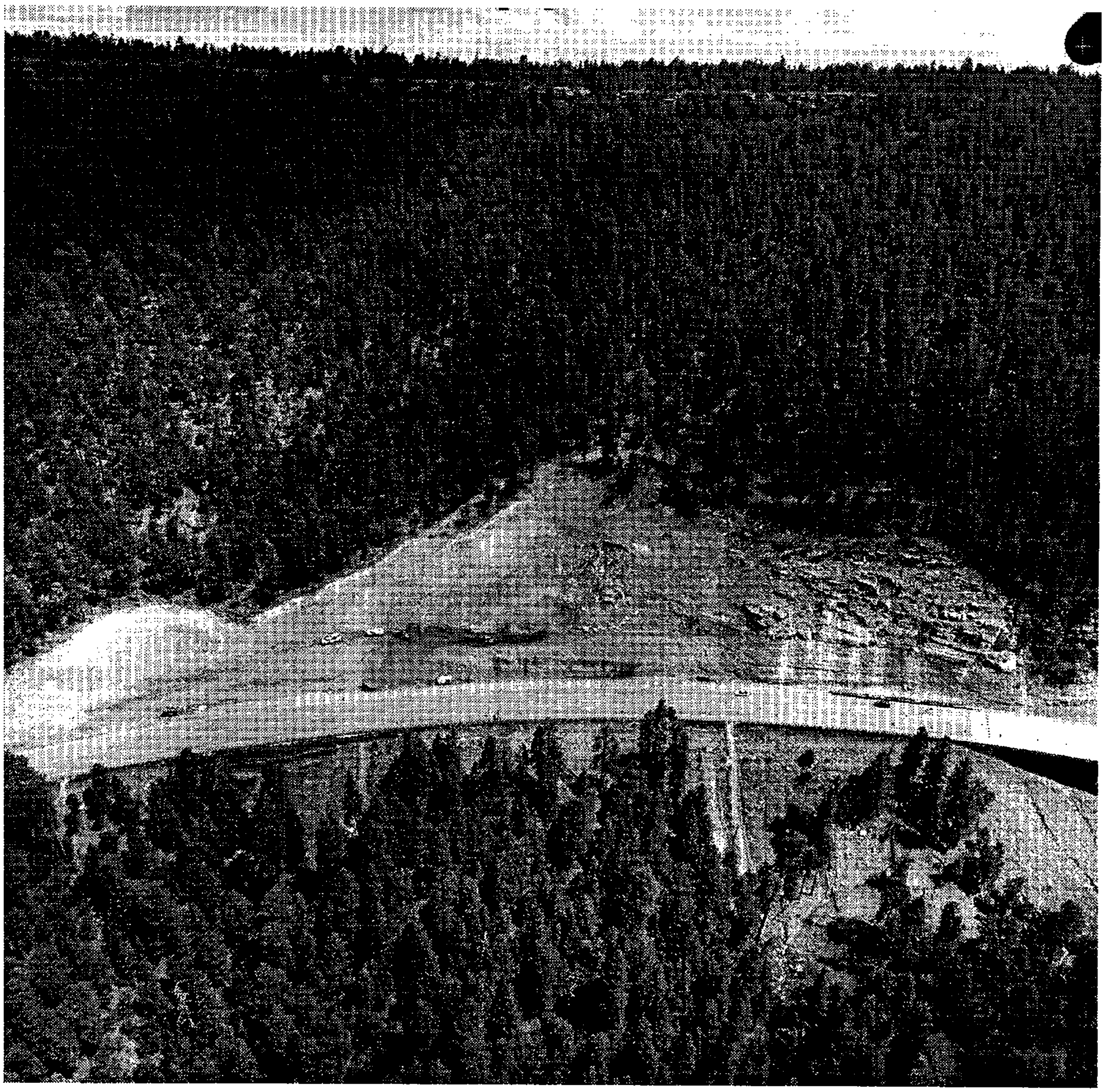
Conclusions

The failure mode of the Gordon Canyon Landslide was a rotational-type slump failure which transitioned to an earthflow following the initial slide failure. The initial failure occurred within unconsolidated colluvium sediments draped on the underlying sandstone, limestone and shale units of the Supai Formation. A portion of the failure surface is probably the colluvium/bedrock contact in the lower portion of the failed mass. The failure does not appear to displace the relatively flat underlying bedrock.

The cause of the failure was probably the result of a reduction in shear strengths within the creep affected colluvium due to hydrostatic loading and wetting. Numerous seeps and springs, many with free-flowing water were reported shortly following the failure.

Subsurface exploration and the interpretation of resistivity soundings data indicate the presence of dual groundwater system at the Gordon Canyon Landslide site. Groundwater is believed to be presented at the colluvium/bedrock

FIGURE 13
OBLIQUE AERIAL VIEW
GORDON CANYON SLIDE



contact with the bedrock restricting the vertical movement of water, forcing the water to move laterally toward the highway road cut. Most of the water in the zone probably originates from local surface infiltration and from bedrock fractures and joints that intersect the bedrock/colluvium contact. Because of the spring affect, the colluvium can be easily saturated by underflow. This groundwater underflow is the principal culprit in the failure mechanism at this site.

Stability analyses at each cross section indicate the existing slope in dry conditions is in stable equilibrium (FS > 1). However, assuming wet conditions, the calculated factor of safety is reduced to FS < 1.

Based on the results of this investigation the following preferred alternative (Figure 13) was developed:

- o Maintain the lower portion of the existing cut slope as-is.
- o Excavate a 20- to 50-foot wide horizontal bench at an elevation at or slightly below the base of the landslide mass.
- o If at the rear of the bench bedrock is exposed, lay back the bedrock slope to about 53° (0.8:1 H to V).
- o Lay back the colluvium section of the slope to at least 34° (1 1/2:1 H to V) and install ground water control devices.

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Reinforced Earth Retaining Wall for Landslide Control, Snake River Canyon, Wyoming

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Abstract

A project involving the use of an in-situ reinforced earth wall for control of a landslide located in the Snake River Canyon of Wyoming is described. The Blue Trail Landslide is located along the proposed route of U.S. Highway 26-89, which runs parallel to the Snake River in western Wyoming. The slide is located in the Lower Cretaceous Bear River Formation, which consists of interbedded shales, siltstones, and thin limestone.

An INSERT wall has been selected as the most effective technique for mitigating the effects of the Blue Trail landslide. This technique, which is a trademark product of Nicholson Construction Company, utilizes reticulated micropiles to reinforce a soil mass in such a way that the reinforced soil acts like a gravity retaining structure to support the unreinforced soil on the upslope side of the wall. The proposed stabilization of the Blue Trail Landslide will be the subject of a monitoring and analysis study by the Wyoming Department of Transportation (WYDOT) and the University of Wyoming. This paper describes the engineering geological features of the slide, presents the proposed instrumentation and monitoring plans, and provides discussion on the major geotechnical design and analysis considerations for the use of reticulated micropile walls for landslide stabilization.

Description and History of the Blue Trail Landslide

The Snake River Canyon--often referred to as "The Grand Canyon of the Snake"-- lies immediately southwest of the geomorphic valley feature known as Jackson Hole. The Snake River Canyon Highway is the principal route providing access to the Jackson Hole region and two major National Parks (Yellowstone and Grand Teton). The Blue Trail Slide is located at M.P. 127.3 (Sta. 14+279 to 14+700) of U.S. Highway 26-89, within the Snake River Canyon between Alpine Junction and Hoback Junction, Wyoming, as shown in Figure 1.

The Bureau of Public Roads began construction of a road through the Snake River Canyon from Hoback Junction to Alpine Junction around 1936 using Civilian Conservation Corps (CCC) laborers. Records indicate that the road through the Snake River Canyon was first paved around 1947. The road was upgraded to its current condition between

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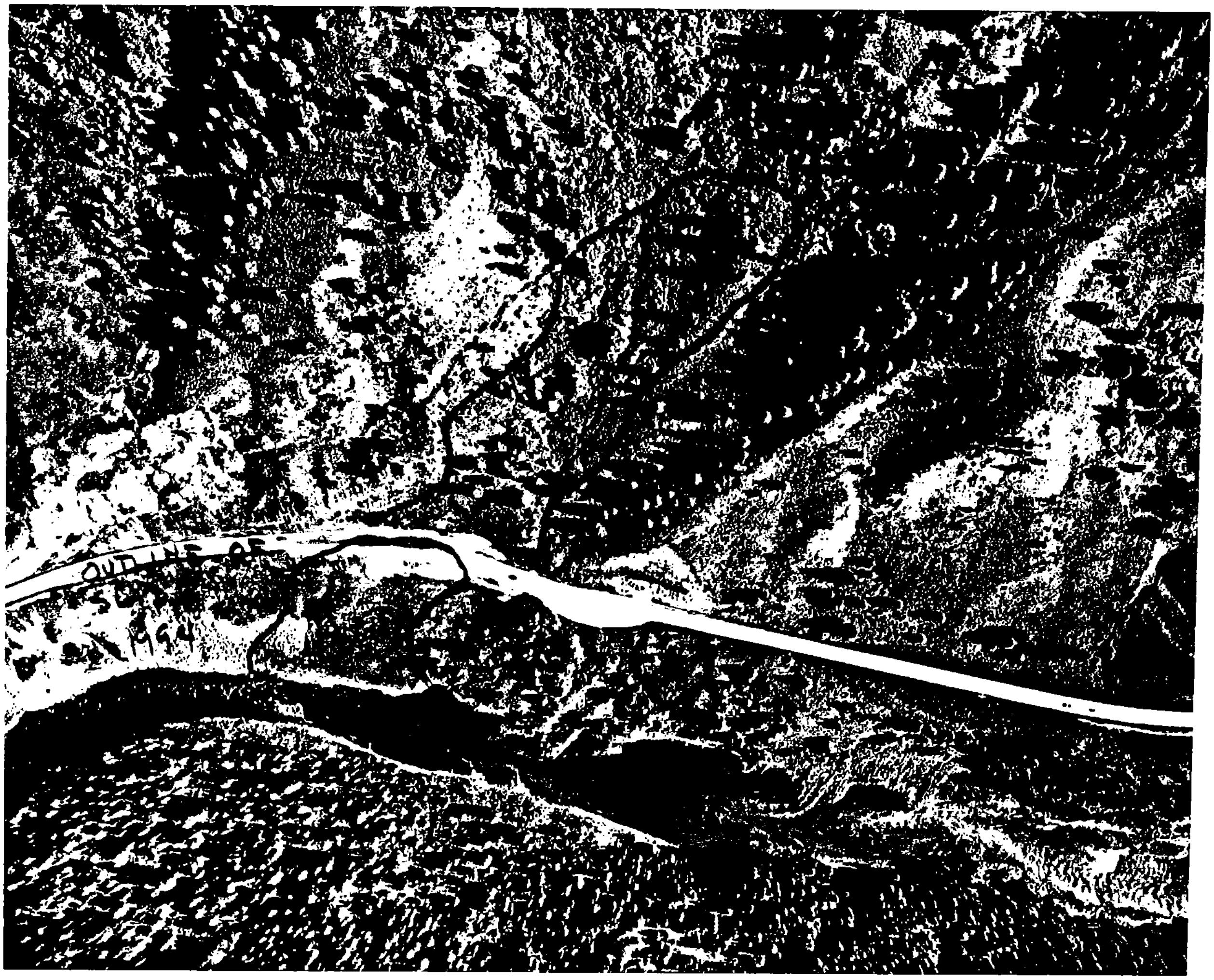


Figure 1. Aerial View of the Blue Trail Slide Area, Snake River Canyon, Wyoming

1955 and 1960. In November of 1969, the Blue Trail landslide was first investigated by the Wyoming Highway Department by drilling 11 test holes. The slide movement reoccurs seasonally, resulting in continual maintenance problems since the highway was constructed. Over the years, the generations of patches have totaled up to 3.7 m (12.1') of Plant Mix Pavement across the most active portion of the slide.

Significant roadway dislocation has resulted from this landslide over the past 27 years. The slide trends N70°W (35° Rt. skew) and stretches from the edge of the Snake River, 61 m (200.1') below the road, to approximately 300 m (984.3') above the existing roadway. The headward zone is active approximately 6 m (19.7') upslope of the present roadway and the mass extends vertically beneath the highway to depths of 10 to 12 m (32.8' to 39.4'). The slide debris above the existing highway is currently stable.

Subsurface materials within the landslide consist of a heterogeneous mixture of detrital sandstone, siltstone, shale, and limestone with sand, silt, and clay. Slide debris is predominantly brecciated rock that has deteriorated to the consistency of clayey gravel. The slide debris varies in thickness from 3.5 to 15 m (11.5' to 49.2') with various perched water tables confined to permeable zones. Water levels are expected to vary significantly with the changing seasons. The landslide debris is underlain by the Lower Cretaceous Bear River Formation. The eastern edge of the slide is very near the contact between the Bear River and underlying Gannet Formation. The Bear River Formation consists of interbedded shales, siltstone, and thin limestone with a strike of N67°W and a dip of 27°W. The Lower Cretaceous Gannet Group is made up of siltstone, sandstone, and cross-bedded quartzite. Figure 2 shows outcrops of the Gannet Formation during the site investigation.

The sliding mass is a *block-type* failure with the movement occurring along a relatively weak layer at the soil-rock interface. Observations indicate that movement occurs during wet periods of the year and when the Snake River is at its highest. Movement is initiated when the river erodes toe material from the lower portion of the slide, thereby reducing support and allowing the upper portion of the slide to move downhill. At the same time slide debris is at a high water content, increasing the driving force and possibly reducing the effective stress along the failure surface, thereby reducing the available resisting forces. The combination of increased driving forces and decreased resistance is believed to be the mechanism which triggers downward slide movements.

A geotechnical investigation and report were completed for the purpose of developing recommendations for the treatment of the geologic hazard posed by the Blue Trail Slide to the proposed highway reconstruction (Chen-Northern Inc., 1989). This report recommends "the most positive solution to the problem is a bridge alternative". The proposed structure would have a length of 366 m (1200.8') and span most of the slide. After reviewing this proposal, the bridge alternative was eliminated by WYDOT because it was determined that all four pier locations were to be founded within the slide debris.

Additional drilling was conducted by WYDOT to identify the slide plane, establish depth to bedrock, and obtain samples for laboratory testing. From the information gathered, an integrated slope stability analysis was performed using the program XSTABL. Assuming that the slide is close to a state of limit equilibrium (factor of safety ~1.0) stability analyses were conducted to develop realistic effective stress strength parameters for the failure surface. These analyses yielded a residual cohesion $\bar{c} = 0$ and friction angle $\bar{\Phi} = 14$ degrees. Subsequent analyses utilizing these parameters were then conducted to assess various means of increasing the stability of the slide.

Figure 2. Investigation of the Blue Trail Slide, October 1994

Typical Roadway Section

The proposed typical roadway section will consist of two 3.6 m (11.81') traveled ways and two 2.4 m (7.87') shoulders, as shown in Figure 3. There will be a plant mix surfaced area 1.7 m (5.58') wide from the outside edge of the shoulder to the top of the wall. This area will be used to place the guardrail. The west bound lane of the roadway will have a 6.7 m (21.98') clear zone measured from the outside edge of the traveled ways.

Selection of Landslide Control Method

Various design alternatives were analyzed in an attempt to stabilize the landslide and to achieve an acceptable factor of safety, considered to be FOS $\geq$ 1.3. Methods considered included lowering the grade and utilizing lightweight fill to reduce driving forces, toe berms to increase resisting forces, and alignment shift to avoid the slide. The narrow canyon limits and close proximity of the Snake River restricted the possibilities of any substantial alignment shift. Various combinations of the above techniques were analyzed using block and circular methods of analysis (XSTABL) and resulted in only minimal increases in the FOS.

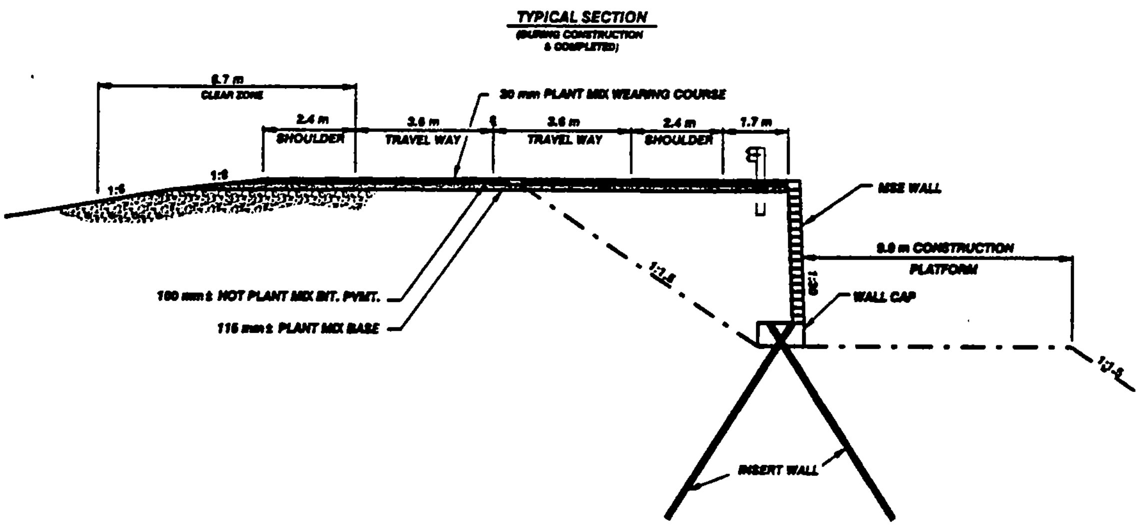


Figure 3. Typical roadway section for U.S. Highway 26-89 at the Blue Trail Slide, Snake River Canyon, Wyoming

With these conventional options determined to be ineffective, alternative techniques were considered. A short list of geotechnical construction companies specializing in ground modification was acquired, which included Nicholson Construction Company. Nicholson engineers proposed a Type A INSERT WALL℠ to correct the slide problem. INSERT (**IN** **S**itu **E**arth **R**einforcement **T**echnique) Wall is a Nicholson Construction trademark wall consisting of steel reinforcing elements installed and grouted over a regular pattern, acting together with the ground to provide a zone of structurally improved soil. The reinforced soil mass can be designed to provide the required FOS=1.3 against sliding. The reported advantages of this type of wall which are relevant to the Blue Trail Slide stabilization project include the following:

Can be constructed in any type of ground. The Blue Trail Slide incorporates a wide variety of geomaterials, ranging from soft soils to intact bedrock. Construction of any type of retaining structure will require excavation of these materials. Because most of the excavation required by an INSERT wall involves drilling for micropiles, excavation volumes are minimized and difficulties are limited to problems associated with drilling relatively small-diameter holes. Several drilling methods are available (Bruce, 1989), the most common being rotary drilling with water flush via a single casing. A potential difficulty at the Blue Trail Slide site is the presence of brecciated rock fragments.

Can be installed under limited access conditions. The steep slope, close proximity to the Snake River, and narrow canyon make this site difficult to access for excavation and construction. Relatively lightweight drilling equipment required for micropiles makes access easier.

Minimal environmental impact, during and after installation. The Snake River Canyon is an environmentally sensitive ecosystem. Minimal disturbance to soils, water resources, wildlife, and scenery is a major consideration in the selection of construction methods.

Cost effectiveness. A preliminary design indicates the Blue Trail Slide will require two INSERT walls to stabilize the landslide, due to the long slide geometry (Avery, 1995). Nicholson's preliminary cost estimate for design and construction of this type of wall is approximately $2,500 per lineal foot of wall. Total estimated cost of the walls is $1,450,000, which is substantially less than the bridge alternative and more efficient from an engineering standpoint.

Proposed INSERT Wall

To achieve the required factor of safety for the Blue Trail Slide, Nicholson Construction has proposed a two-wall system. The upper wall will be placed approximately adjacent to the edge of the roadway with the lower wall approximately 55 m (180.45') down slope. The upper wall will be approximately 95 m (311.68') long, with the lower wall being approximately 82 m (269.03') in length. The proposed layout of the INSERT wall is shown in Figure 4.

The upper INSERT wall will consists of a structural concrete cap approximately 1.5 m (5') wide by 1.2 m (4') high. Micropiles in the upper wall are 114 mm (4.5") diameter high strength 551.5 MPa (80 ksi) steel pipe grouted in 152 mm (6") diameter holes as shown in Figure 5, spaced on 508 mm (20") centers. The micropiles will be battered 30° degrees from vertical alternating upslope and downslope. The upslope micropiles will be 19.8 m (65") long and the downslope micropiles will be 25.9 m (85') long as shown in Figure 6.

The lower INSERT wall will consists of a structural concrete cap approximately 1.5 m (5') wide by 1.2 m (4') high. The micropiles in the upper wall are 114 mm (4.5") diameter high strength 551.5 MPa (80 ksi) steel pipe grouted in a 152 mm (6") diameter hole spaced at 381 mm (15") centers. The micropiles will be installed 30° degrees from vertical alternating upslope and downslope. The upslope micropiles will be 12.2 m (40") long and the downslope micropiles will be 15.2 m (50') long. The lower wall will have tiebacks spaced at 2.3 m (7.5') centers to provide stability during and after an earthquake as shown in Figure 6.

Technological innovations in geotechnical engineering construction often precede the development of rigorous design methods. The use of micropile-reinforced earth for retaining walls such as the INSERT wall is an example of an innovative commercial

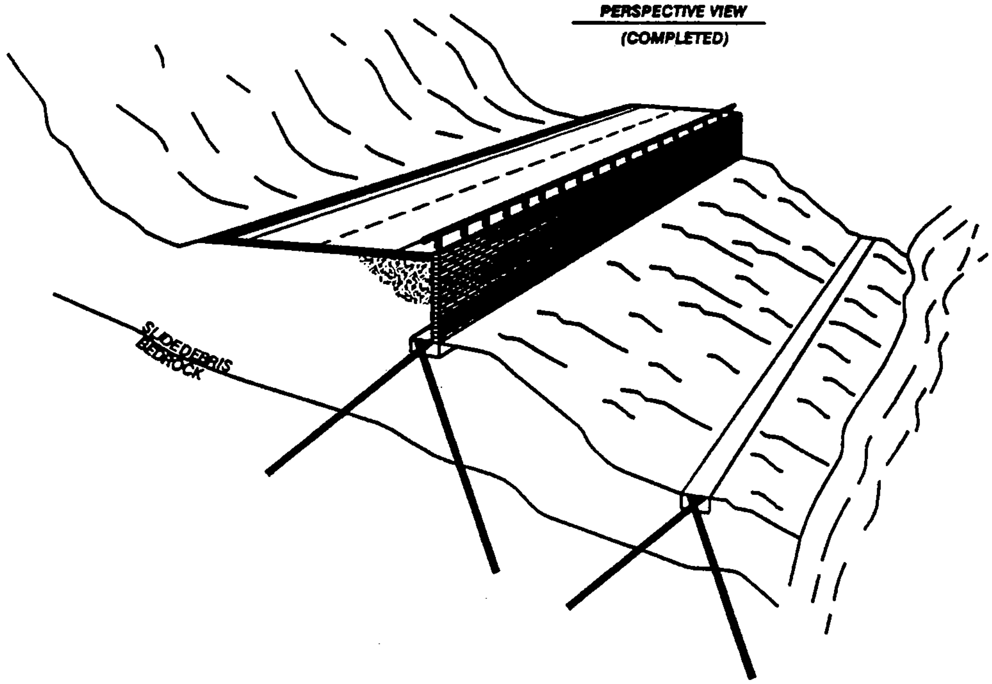


Figure 4. Proposed INSERT wall for stabilization of the Blue Trail Slide, Snake River Canyon, Wyoming

application for which design procedures are still evolving, based on studies of field performance and analysis of soil/structure interaction. The general steps in preliminary design of an INSERT wall are as follows (Pearlman et al., 1992):

1. Conduct slope stability analyses to determine the additional resisting force on the failure plane required to achieve the desired factor of safety
2. Check the potential for structural failure of the micropiles due to loading caused by the moving soil
3. Check the potential for failure by plastic flow of the soil around the micropiles

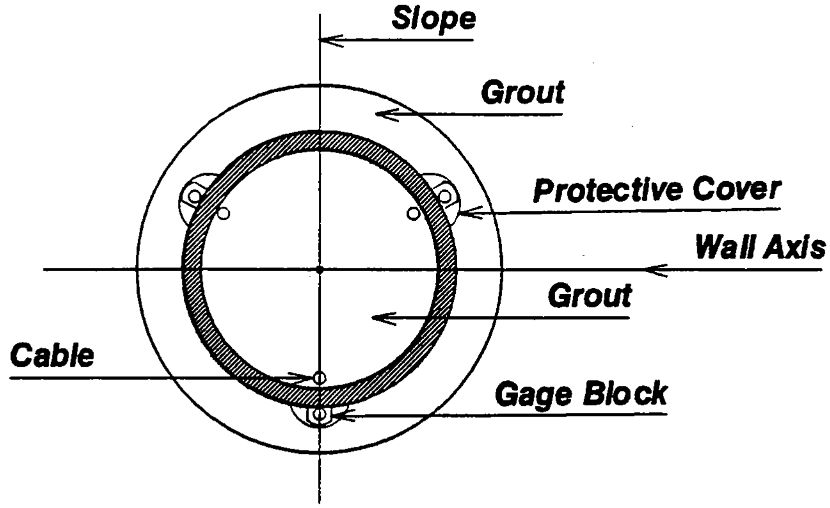


Figure 5. Cross section of INSERT wall micropile with strain gage locations

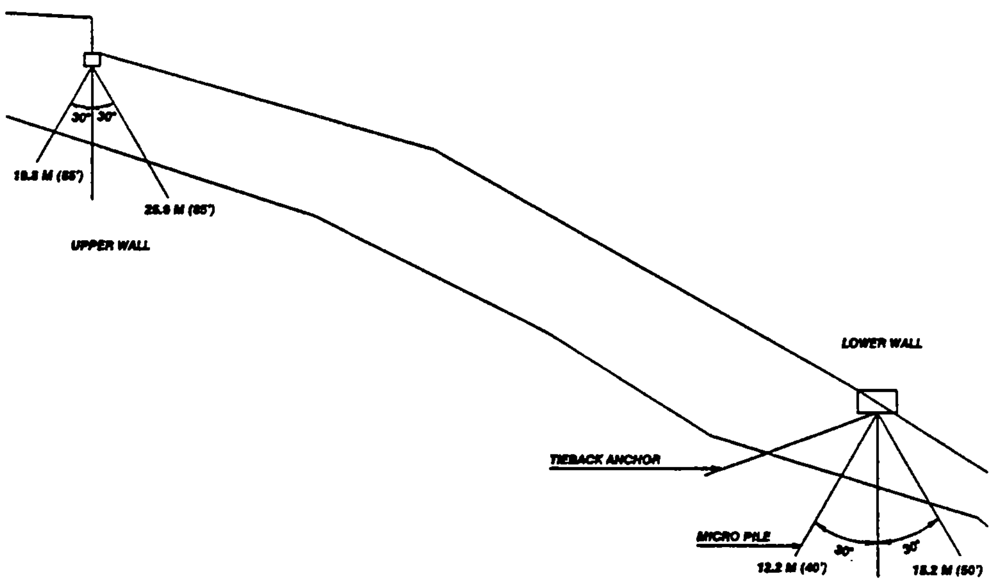


Figure 6. Cross section of the INSERT wall for the Blue Trail Slide

Step (1) above can be conducted using conventional slope stability analyses. Step (2) involves calculating the lateral forces acting on the micropiles due to the sliding mass of soil. Pearlman et al. (1992) describe a procedure for calculating the ultimate horizontal resistance of micropiles, defined as the lateral force causing yielding of the steel pipes or crushing of the concrete around the central reinforcing bar, based on the method of Fukuoka (1975). This method requires the coefficient of subgrade reaction of the soil or rock above and below the failure plane. The Fukuoka method also enables the designer to calculate bending moment and lateral deflection of the micropile versus depth. Step (3) is used to insure that micropile spacing is adequate to prevent plastic flow of soil around the micropiles. The analysis is based on a method developed by Ito and Matsui (1975). A design chart given by Pearlman et al. (1992) provides design spacings as a function of micropile diameter, depth to the failure surface, and soil strength in terms of undrained shear strength for undrained loading or soil friction angle for drained loading conditions.

In addition to lateral loads analyzed in Step (2), movement of the sliding mass also mobilizes axial loads in the micropiles. The magnitude of axial load transfer must be considered to determine the resisting force provided by the reinforced soil mass. Axial load transfer occurs primarily through side resistance, or skin friction, which is calculated as the summation of shearing resistance available along the soil/pile interface. Methods for calculating side resistance of other types of deep foundations (e.g., drilled shafts) plus experience generally are used, however research is needed to develop a reliable procedure for predicting micropile side resistance, based on soil strength properties and considering the effects of construction procedures.

The INSERT wall for the Blue Trail Slide was designed for both static and dynamic loading conditions (Avery, 1995). The static loading case was designed to have a factor of safety of 1.3. The dynamic loading case was designed to have a factor of safety of at least 1.0. For the static condition the upper wall will provide a resisting force of 222.4 kN (50 kips) per lineal foot of wall. To meet the requirement of a factor of safety greater than 1.0 against earthquake induced lateral accelerations of 0.11 g for the dynamic loading condition, the tiebacks were placed in the lower wall only. To maintain stability during and after an earthquake, the wall design required an additional 177.9 kN (40 kips) per foot of wall.

A conceptual computer rendering of the Blue Trail Slide area following construction is shown in Figure 7. The view point of the conceptual rendering is approximately the same as that of Figure 2.

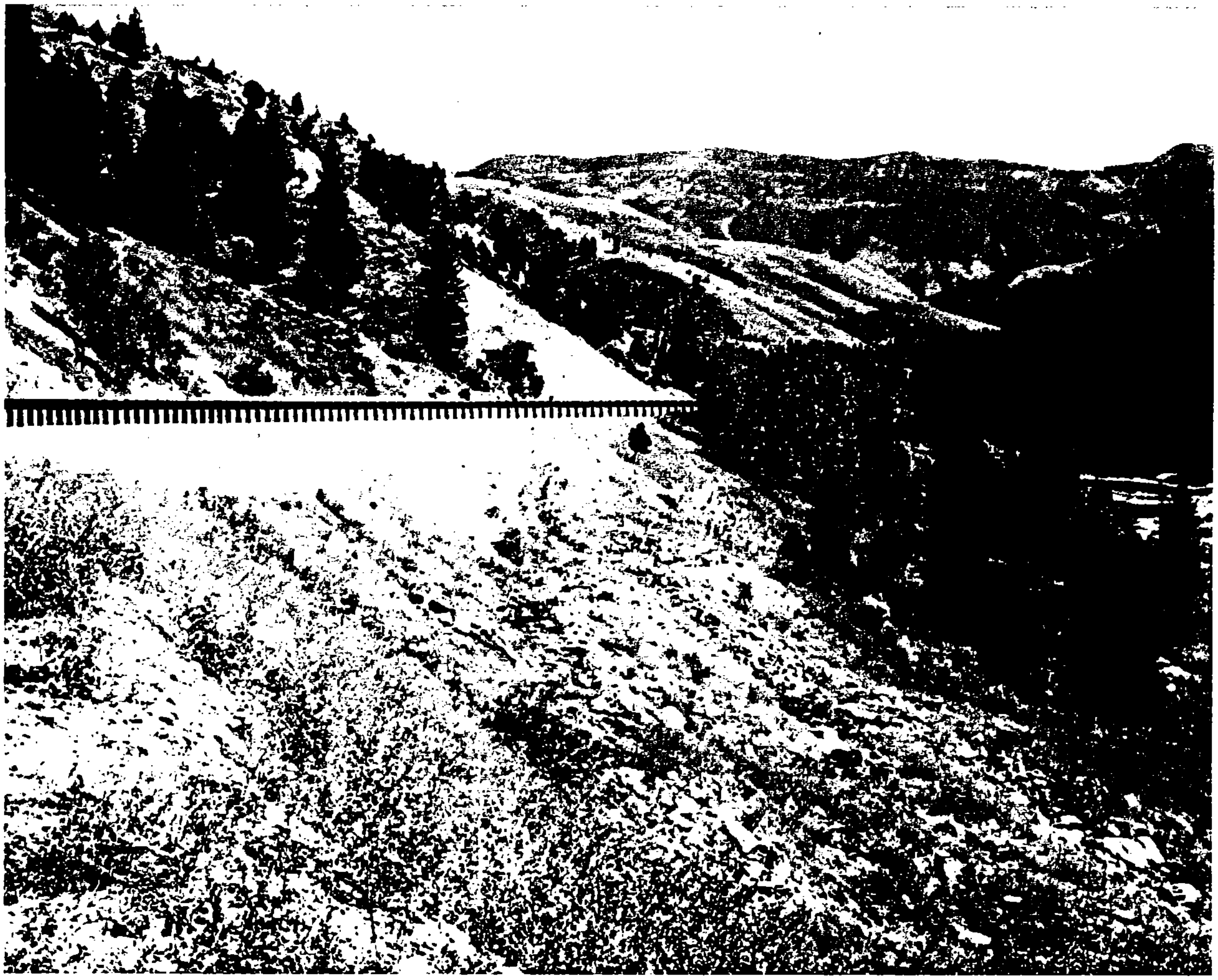
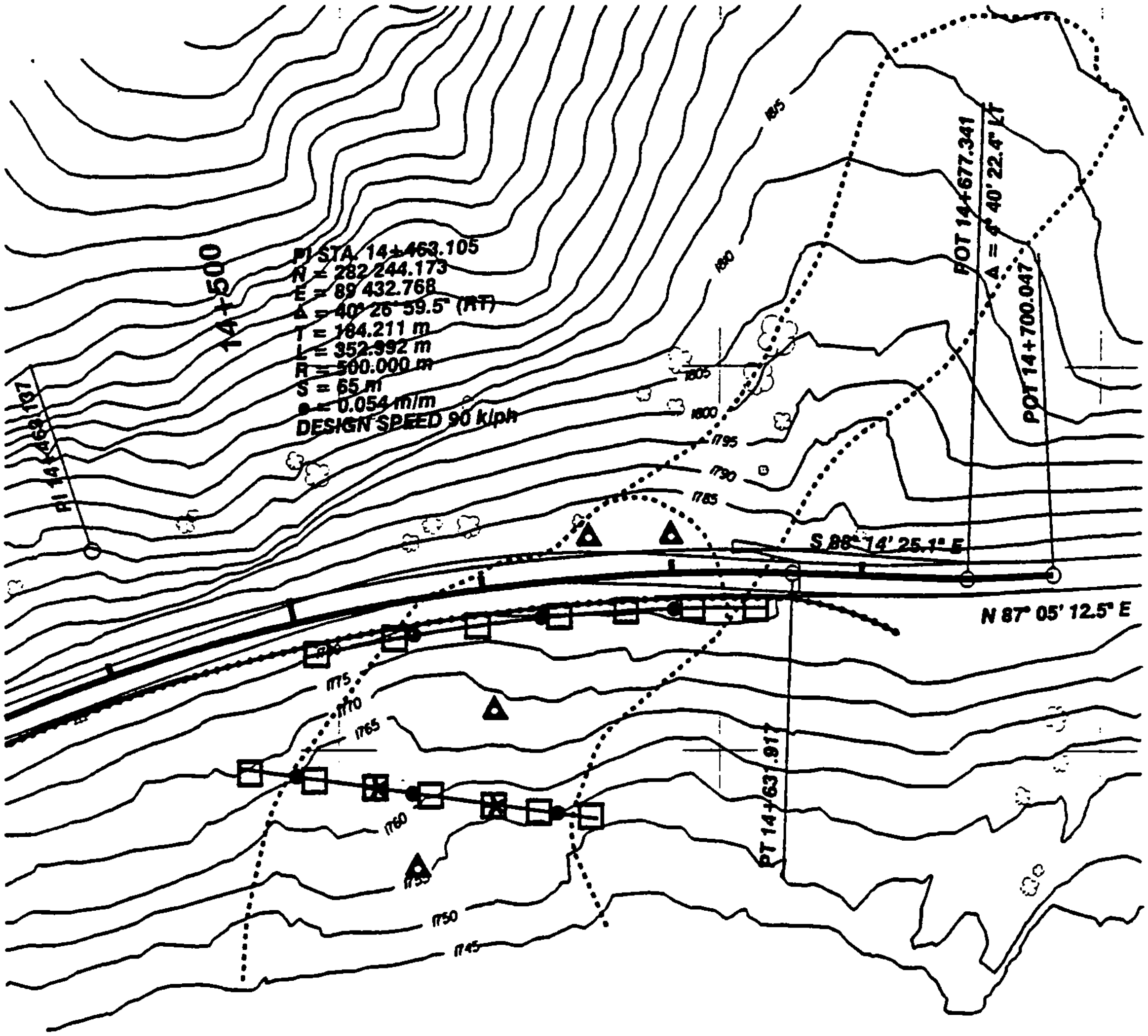


Figure 7. Conceptual design of the completed INSERT wall for the Blue Trail Slide, Snake River Canyon, Wyoming

Instrumentation and Monitoring

The goal of the proposed instrumentation and monitoring plan is to address the following design and performance characteristics of the earth retaining technique:

1. Types and amounts of wall movement
2. Load transfer to the micropiles and the tieback anchors
3. Actual pore water pressures in the slide material
4. Effects of pore water pressure on slide stability
5. Cost-effectiveness of the INSERT wall as a landslide stabilization method for the WYDOT



X - strain gaged micropiles

O - Slope Inclinometer Casing Location

□ - Tilt Plate Location

* - Load cell Location

Δ - Piezometer Location

Figure 8. Instrumentation location plan for Blue Trail Slide, Snake River Canyon, Wyoming

Various instruments will be installed and monitored in order to assess the above factors. The instrument types and locations are shown in Figure 8 and discussed below.

Wall Movements

Slope inclinometers will be used to measure post-construction movement of the landslide. Slope inclinometer casings will be placed at the ten locations indicated in Figure 8. Slope Indicator Model 1000 inclinometer will be used.

Settlement and rotation of each wall will be determined by the use of a tiltmeter and field surveying. Tilt plates will be located as shown in Figure 8. The tilt plates are proposed to be located along the top of both INSERT walls at an even spacing depending on the length of each wall. There will be approximately 14 tilt plates used, seven on each wall. The tiltmeter used will be a Slope Indicator model number 50304410 and the tilt plates will be Slope Indicator model number 50307300. A settlement cell will be used to measure any settlement which may occur in either wall. The settlement cell will be a Slope Indicator model number 51408300. The field survey will be used to determine x, y, and z coordinates on each of the tilt plates as an additional check for settlement and rotation. The walls have been designed to be able to have the cap buried as much as 1.5 m (5') below grade for aesthetic reasons. The tilt plates may not be used if this option is chosen.

Load Transfer

Strain gages will be used to determine actual loads in the micropiles. The strain gages will be placed in groups of three around the micropile and at three different elevations along the length of the micropile. Each instrumented micropile will have nine strain gages as shown in Figures 5 and 9. There will be three pairs of micropiles strain gaged on each wall, for a total of 12 instrumented micropiles. Each pair of micropiles will consist of two micropiles separated by approximately 0.5 m and will consist of one micropile in the upslope location and the second will be in the downslope location. Therefore, one micropile will be in tension and the other in compression. The location of the micropiles which will be strain gaged is shown in Figure 8. This layout of the strain gages will require approximately 108 gages. The strain gages will be Slope Indicator model number 52602100 and will be welded in place with protective covers.

Load cells will be used to determine the actual load in the tie backs. The load cells will be placed as shown in Figure 8. A total of two load cells will be used on the lower wall. The load cells will be Slope Indicator model number 51301225.

Pore Water Pressures

A piezometer (Slope Indicator Model No. 52612510) will be used to measure actual pore water pressures at different locations in the slide area. There will be approximately four inclinometer casings used to insert the piezometer as shown in Figure 8.

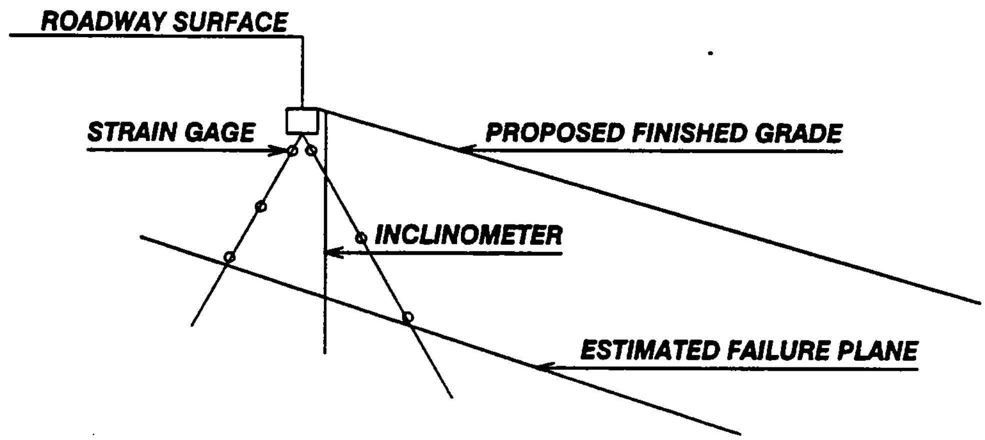


Figure 9. Strain-gaged micropiles

Data Collection

Data from the instrumentation will be collected using a CR10 and PC 208 software from Slope Indicator. The data will be downloaded to a laptop computer in accordance with the monitoring plan for transfer to the University of Wyoming for analysis.

Monitoring Plan

The monitoring plan calls for the strain gages and the load cells to be read three times a day for the first year. The inclinometers, tilt plates, piezometer, and the settlement cell will be read monthly. The x, y, and z coordinates of the tilt plates will be checked annually. After the first year the monitoring plan will be reviewed and any changes found necessary will be made. The number of strain gage and load cell readings required in the monitoring plan will require the use of two data collectors, one for each wall. The data collector has the capability to handle 16 channels or readings. This can be expanded to a total of 64 channels when using four multiplexers. Each data collector will require three multiplexers to handle the number of strain gages in each wall.

Summary and Conclusions

The Blue Trail Slide in the Snake River Canyon located in northwestern Wyoming has been a continuous maintenance problem for the past 27 years. During that time, the roadway has subsided approximately 3.7 m (12.1'). The slide plane is located at the interface of the Lower Cretaceous Bear River Formation and the underlying Gannet Formation, with the sliding mass defined as a block type failure. The high subsidence can be traced to the combined effects of the subsurface material(a mixture of sandstone, siltstone, shale, and limestone, with sand, silt, and clay) and soil movement which occurs primarily during the spring season when the slide debris is at a high water content with the Snake River at its highest levels, causing erosion of the lower portion of the slide.

Several design alternatives were evaluated during the design phase of the project. These alternatives consisted of lowering the grade and utilizing lightweight fill to reduce the driving forces, toe berms to increase the resisting forces, alignment shifts to avoid the slide area, the use of a structure to span the entire slide, and the installation of the INSERT wall. The INSERT wall was determined to have the most advantages, fewest disadvantages, and was the most cost effective of all the alternatives investigated.

The size and extent of the Blue Trail Slide has necessitated the use of a double-wall configuration to control the movement of the slide. This will be the first time the contractor, Nicholson Construction, has installed this configuration of a wall system.

Due to the sensitive nature of the environment and the history of the slide, the INSERT walls will be instrumented and monitored for at least a two-year period to determine the types and amounts of wall movement. Load transfer to the micropiles and tiebacks, as well as the pore water pressures in the slide material, and their effects on the slide stability, will be evaluated. Finally, after actual construction costs are tabulated, the cost-effectiveness of the INSERT wall for stabilization of the sliding mass will be closely examined and applied to future feasibility studies of other slide stabilization projects.

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ABSTRACT

WEDGE STABILITY ANALYSIS - GEOMETRIC AND GROUNDWATER ENHANCEMENTS

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The concept of the three dimensional wedge and its method of analysis in rock slope stability was first developed by John Bray at London University and was subsequently described in detail by Hoek and Bray in their book "Rock Slope Engineering". This technique has been used excessively since that time and has found use in a wide variety of applications. However, there are two limitations to the analysis method which somewhat limit its use.

First, in steep mountainous areas where the line of the intersection between the two sliding planes does not intersect the upper slope surface, the algorithms do not allow formation of a wedge, despite the fact that a tension crack can define the back surface of the wedge. Second, the analysis only permits either completely dry or fully saturated wedges, with changes in water pressure being simulated by changing the water density. A revised method of analysis will be discussed which overcomes these limitations and also develops a three dimensional picture and model of the slope. These models assist in the correct positioning of the tension crack and slope reinforcement, if required.

Seismic Refraction as a Method for Determining Thickness of Organic Sediments

By Priscilla Duskin, New York State Dept. Of Transportation, Albany, NY

Abstract

The New York State Department of Transportation has used seismic refraction as a tool to find depth to bedrock for nearly 50 years. Recently this old method was successfully used in the unusual application of finding the thickness of organic sediments on a highway relocation.

The project, the Utica-Rome Expressway, is located in central New York State and involves construction of a large embankment through a wetland to be used for relocation of Rte. 49. The geology of the embankment site consists of silt, sand, and clay from an old glacial lake. In some areas, a significant thickness of organic material overlies the lake sediments. The organic layer presents a problem for design engineers because of excessive settlement from its low strength and high compressibility. The designers needed to know the thickness of the organic layer in order to decide whether the organics should be excavated prior to embankment construction, or if some other method of alleviating the settlement could be selected. Because of time constraints, there were an insufficient number of boreholes to determine the extent of the organic layer.

Seismic refraction operates by introducing a compressional wave into the ground and measuring the arrival time of the refracted wave at an array of geophones. The resulting arrival time data is plotted on a graph of time versus distance of the energy source to the geophone. The inverse of the slope of the graphed line is the seismic velocity of the material. Seismic velocity is directly related to the density of the material; the denser it is, the greater the velocity. When determining depth to rock, the overburden seismic velocity is considerably lower than the seismic velocity of the rock. It is this difference of seismic velocities which makes depth calculations of the overburden layer possible.

Normally, water-saturated sediments have a seismic velocity of about 4,900 feet per second, which is the speed of a compressional wave in water. However, the presence of organic material in sediments can have the effect of reducing the seismic velocity, because the decay of organic material releases carbon dioxide gas. The gas decreases the density of the sediments, so that the seismic velocity of the organic sediment is about 1,100 feet per second, even if the sediments are under water. This is a significant velocity contrast with the underlying sand and silt, and permits calculation of the thickness of the organic gaseous silts.

A preliminary seismic array was shot between two boreholes which had encountered organic silts, and the calculated depth of the organic layer agreed with the depth from the boreholes to within one foot. Seismic arrays were then set out along the proposed embankment site at the project, and depth of organics was calculated at six points. This involved one day of field work and a few hours of data reduction, and precluded the necessity of more drilling in the wetland.

Introduction

Geologists at the Geotechnical Engineering Bureau of the New York State Department of Transportation have been applying seismic refraction to geotechnical engineering designs since the Bureau's inception in 1946. During the era of the design and construction of the Eisenhower Interstate Highway System, as many as three seismic refraction crews would be working continuously to find depth to bedrock for highway cuts. Although this workload has been reduced considerably in the past twenty years, seismic refraction is still used by NYSDOT as a supplement to data obtained from boreholes. Its uses are to find depth to bedrock for highway cuts, bridges, and retaining walls, as well as finding the thickness of saturated granular material for use as water well supplies for various State institutions.

Recently, geotechnical design engineers at NYSDOT encountered a sizable thickness of organic sediments in an area where an embankment was to be constructed. The proposed embankment site is near the Mohawk River, east of the city of Utica, New York. The soils at the site consist of alluvial silts and sands overlying an old glacial lakebed. In some areas of the project site, organic silts and peat are present above the alluvium. The presence of organic sediments poses a design problem because of possible embankment foundation failures stemming from long-term settlement of the organic layer under the proposed embankment. There are several solutions for embankment construction over organic sediments, the choice of which is dependent on the thickness and lateral extent of the organics. There were a number of boreholes progressed at the Utica site, but some of these did not penetrate the full thickness of the organic strata, and there was uncertainty as to whether more borings could be done in time for the embankment design. It was at this time that the Geology section of the Geotechnical Engineering Bureau was contacted to see if seismic refraction could be used to obtain additional subsurface information in areas of known organic soils.

Engineering Significance of Organic Soils

Organic deposits can be classified into the following categories: organic clay; organic silt; and peat and muck. Organic clays are found in ocean bays, such as Long Island Sound and San Francisco Bay. Organic silts are found in alluvial floodplains and swamps. Peat and muck, which are nearly all plant matter with very little mineral soil, are common deposits in swamps, bogs, and glacial kettle holes. Peat is formed in the early stages of plant decomposition and is brown in color. When decomposition is more complete, the original plants are less recognizable, and the black residue is called muck. Organic silts and peats are sometimes interbedded, and non-organic silts may contain pockets of either organic silt or peat. Organic matter in soils may coat individual grains of mineral soil with a film of organic constituents. Organic molecules may also combine with clay minerals, especially montmorillonite clays (Hunt, 1972). In New York State, peat is generally found in proximity to glacial deposits, but organic silt can be found in Recent alluvial deposits. The moisture content of organic silts and clays ranges from 30% to 140%, while the moisture content of peat is from 200% to 3000% (Fig. 1).

All organic deposits create engineering problems for transportation systems because of their low strength and high compressibility. These problems are greatest in organic deposits located at or near the surface. Deeply buried organic materials have been compressed to a denser, stronger state and

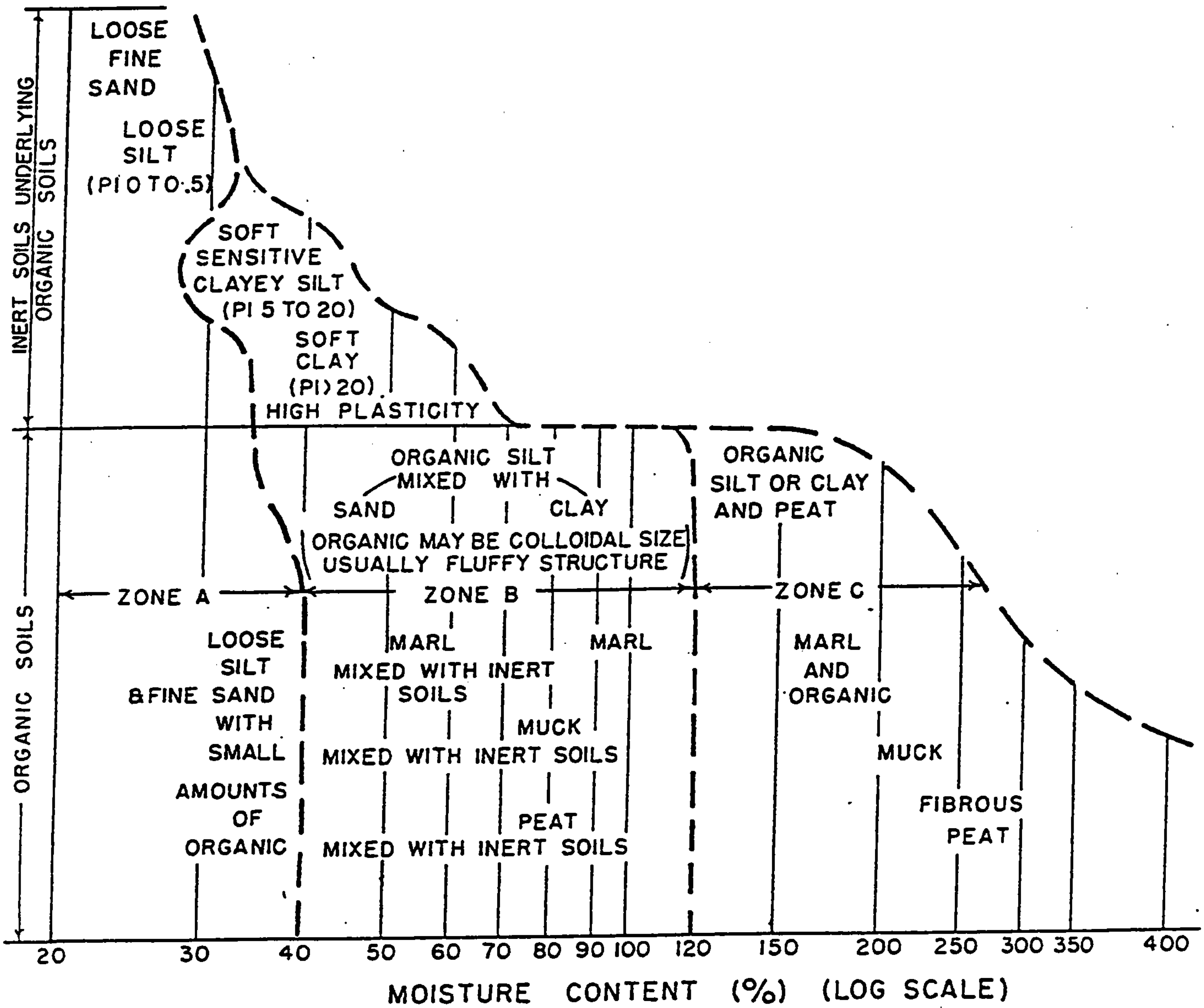


Figure 1. Engineering characteristics of organic soils (from Design Manual, N.Y.S.D.O.T Geotechnical Engineering Bureau)

will undergo less settlement for the same superimposed load. In New York State, the vast majority of known organic deposits are at or near the surface.

When an embankment is to be constructed in an area of peat or organic silt with peat, there are several engineering options. One is to entirely excavate the organic deposit and replace it. This requires extra time and expense in the design because excavation and disposal of the organic material requires an environmental impact statement. The excavation trench must be opened and filled within the space of a few days, and must be kept full of water in order to keep the surrounding material from failing, because organic deposits have a very low shear strength. If the organic layer is very thick (greater than 20 feet), excavation may not be practical.

Another method of constructing an embankment in organic deposits is to use a surcharge, possibly in combination with wick drains to accelerate consolidation and strength gain. The embankment is built to a height greater than the design height and left in place for six months or more, so that the underlying sediments are consolidated by the surcharge. The time needed for the settlement depends on the thickness of the organic deposits. After the settlement period is complete, the surcharge is removed and embankment construction continues.

For a thick peat deposit, the displacement method of soil removal has been used (Sinacori et al., 1951). This is done by placing sufficient embankment material on the site to cause a rapid shear failure of the underlying soil. The resultant mudwave is then excavated and removed (Fig. 2).

Ground improvement techniques, such as columns of stone emplaced by dynamic compaction, can also be used when constructing embankments on organic deposits.

As can be seen, there are several engineering designs available for the construction of an embankment on peat or organic silt with peat. Choosing the best and most cost-effective method requires a thorough subsurface exploration of the area. Often the best method of building a roadway through organic deposits is to move the proposed alignment away from the organic deposit. If this is not possible, then borings must be progressed so that the depth of the organic matter can be determined, and the material can be tested for moisture content, primary and secondary consolidation properties, shear strength, and percent of organic content. The installation of borings in an area of organic sediments is indispensable; however, a surface geophysical survey may be useful in defining the thickness and areal extent of organic deposits, and can supplement the borehole information or even decrease the number of boreholes the designer requires. Seismic refraction is one geophysical method of determining the extent of organic deposits.

Brief Review of Seismic Refraction

Seismic refraction is a surface geophysical method which is used to find the thickness of layers of earth materials. It works by means of introducing a compressional wave into the ground, then measuring the velocity of the wave through the strata. It is most useful as a subsurface exploration technique when done in conjunction with the installation of boreholes.

The method employs one or two cables, each having connections for 12 geophones. The cables are

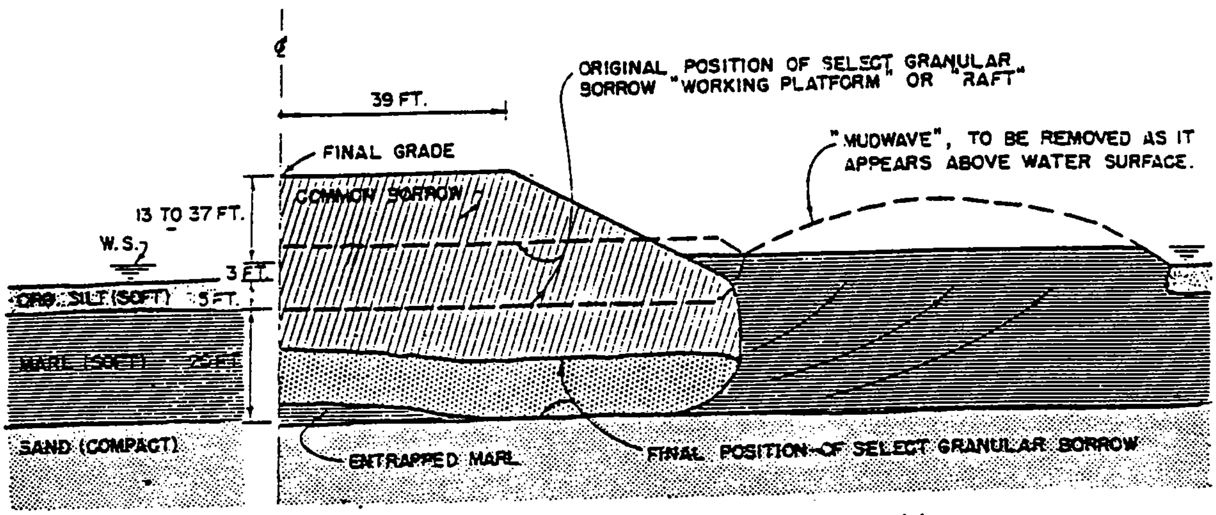


Figure 2. Sinking an embankment by displacement due to its weight.

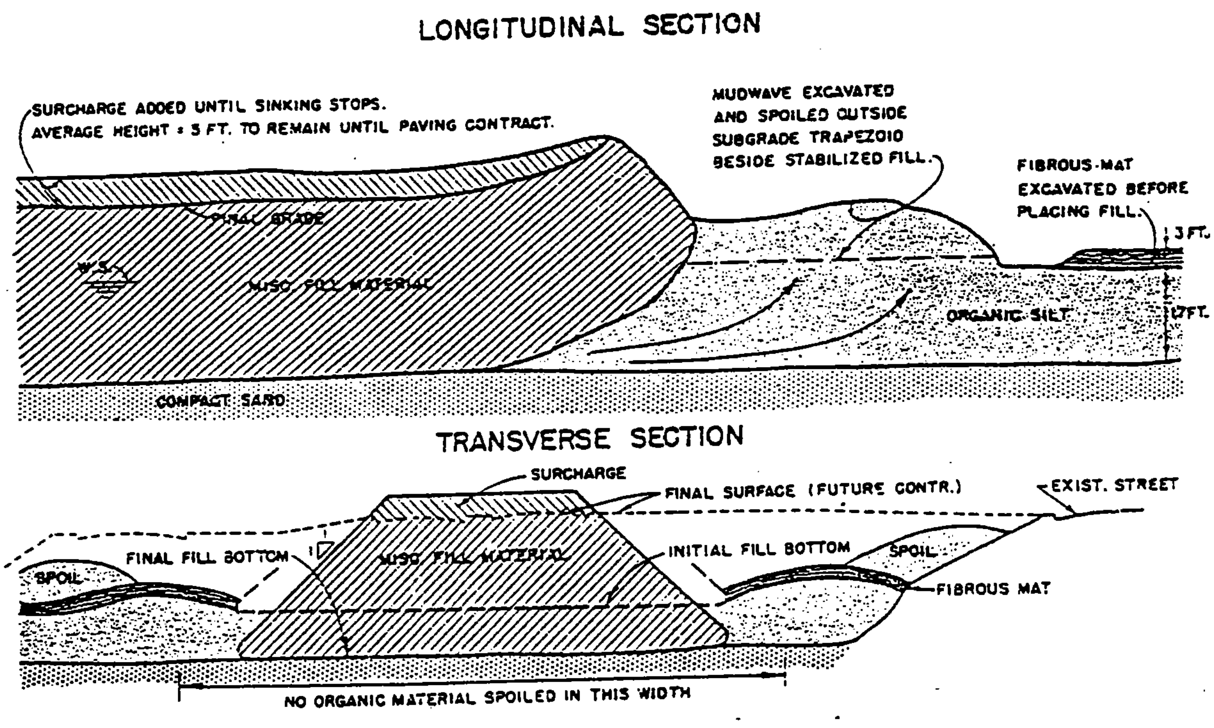


Figure 2. Removal of soil by displacement (from Sinacori et al., 1952)

connected to a seismograph. A buried charge is detonated at the end of each cable, and the seismograph records the time it takes for the compressional wave to arrive at each geophone. Because the distance from the energy source to each geophone is known, the wave arrival data can be plotted on a graph of time (in milliseconds) versus distance from source to geophone. This yields a line having a slope of $1/V_1$, V_1 being the seismic velocity of the uppermost strata. When the compressional wave reaches the interface with the next lower layer, the wave travels along this interface at the seismic velocity of the second layer and is refracted to the surface. The slope on the plot of the arrival times then changes to $1/V_2$. The depth to the second layer can then be calculated (Fig. 3).

The basic premise upon which seismic refraction depends is that the earth strata are in distinct layers, each having a seismic velocity greater than that of the layer above it. In general, this is the case. However, if an uppermost layer has a seismic velocity greater than that of the underlying layer(s), the method will not work.

Seismic refraction, like most surface geophysical methods, supplements the information obtained from boreholes. It has several advantages over drilling, such as: speed of data collection; the ability to distinguish between bedrock and very large boulders; laterally extensive geologic information rather than data at a single point; and portability over land and water. The last item is a particular advantage in defining the extent of organic deposits, because the swamps and wet areas where organics occur are particularly difficult sites to set up a drill rig.

Origin of Organic Soils

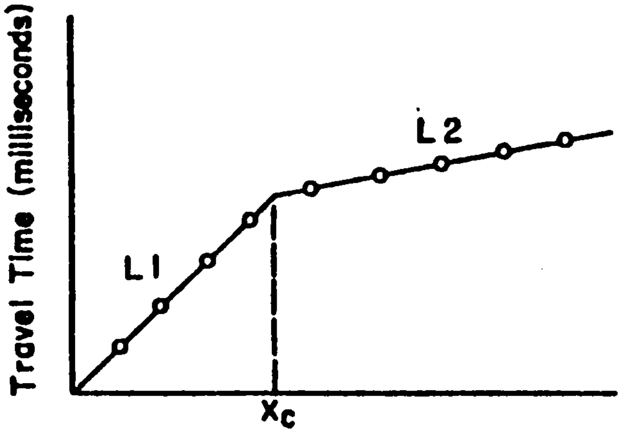
Organic matter accumulates on low, wet areas which have poor drainage. Examples are depressions between rock ridges, kettle holes, old oxbow lakes on floodplains, and former glacial lake basins which have drainage impeded by other glacial deposits.

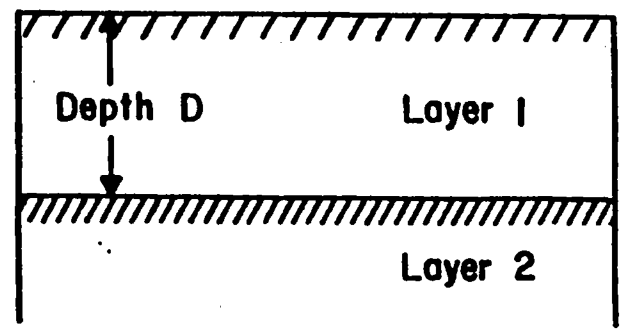
The plant matter in an organic deposit consists mainly of cellulose and lignin, which are both complex carbohydrates. Lignin is about 25% of the dry weight of wood, and is more resistant to decay than cellulose. Peat is composed of decaying plant matter and the breakdown byproducts, such as humic acid. The end product of plant degradation is coal (Brownlow, 1979).

In a dry environment, plant matter is decomposed by aerobic bacteria. In areas which have a high water table, such as swamps and floodplains, dissolved oxygen is quickly depleted, and decomposition of plant matter is done by anaerobic bacteria.

The microbial process involved in the breakdown of plant matter by anaerobic bacteria takes place in two stages. Stage 1 is the hydrolysis of cellulose into simple sugar, $C_6H_{10}O_5 \rightarrow C_6H_{12}O_6$. The sugar is then converted into volatile organic acids, mostly acetic acid, CH_3COOH.

During Stage 2 of the anaerobic process, the organic acids are used by methane-forming bacteria to produce energy for the bacteria, which results in the formation of methane, CH_4, and carbon dioxide gas, CO_2, as byproducts (Wardell et al., 1983).





L 1 = Layer 1

L 2 = Layer 2

V 1 = Velocity of Layer 1 = 1/Slope of L 1

V 2 = Velocity of Layer 2 = 1/Slope of L 2

X_c = Critical Distance

$$D = \frac{X_c}{2}\sqrt{\frac{V2 - V1}{V2 + V1}}$$

(For Two Horizontal Layers)

Figure 3. Time/distance plot for simple two-layer structure

If the chemical system of the above two stages is balanced, the pH of the system is around 7. However, if the nutrients available to the bacteria are deficient, particularly in nitrogen, the Stage 1 bacteria will predominate and produce excess organic acid, and the pH of the system will drop to a level where the Stage 2 bacteria cannot exist. When this happens, no more methane and carbon dioxide will be produced.

There are several factors other than nutrient availablity which affect the rate of plant decomposition and gas production. Temperature is a major factor, in that these biochemical processes only occur within a temperature range of 15 to 45 degrees Celsius; the rate of decomposition roughly doubles with every 10 degree increase in temperature. Also, the percentage of lignin affects the rate of decompostion, because lignin, unlike cellulose, does not hydrolyze readily. Lignin occurs in woody plants, rather than grasses, mosses, or ferns.

The anaerobic process of plant decomposition produces carbon dioxide and methane gas in roughly equal quantities. Methane rises out of the system and is released as "swamp gas", because it is a non-polar molecule and is not soluble in water. However, not all the carbon dioxide produced during plant decomposition is released as gas, because it is a polar molecule, as is water. As a result, some of the carbon dioxide remains in solution in the water. The weight of overlying water or sediment increases the pressure on the system and causes the carbon dioxide gas to be even more soluble in water. This is the principle behind carbonated beverages, which are bottled at pressure greater than that of the atmosphere so that the CO_2 gas will remain in solution.

The geophysical significance of the dissolved gas in the water is that the gas has the effect of reducing the seismic velocity of the water or water-saturated sediments. Normally, water-saturated sediments have a seismic velocity at least as high as that of water, which is 4875 feet per second. However, the dissolved gas in organic sediments reduces the seismic velocity to 1100 to 1400 feet per second, even in an area that is completely under water. It is this low velocity phenomenon which makes it possible to use seismic refraction to find the thickness and extent of organic sediments.

The occurrence of a gas velocity in water-saturated sediments was first noticed by this writer five years ago, while shooting refraction lines for a proposed bridge crossing over the Mohawk River near Schenectady, New York. The river here flows over a thick sequence of Pleistocene gravels, so that one would expect a seismic velocity of about 5000 feet per second over a rock velocity of about 14000 feet per second. The first seismic array was located in shallow water near the south bank of the river, and showed 16 feet of material having a velocity of 1300 feet per second overlying gravel. This seemed inexplicable until a retired NYSDOT geologist recounted how he had seen the same thing 25 years earlier on the Hudson River near Albany, and explained that it was the result of "gaseous silts." Other seismic arrays in the Mohawk River did not exhibit this behavior, because that section of the river is part of the New York State Barge Canal system and is regularly dredged through the main channel. The dredging removes the organic silts at the river bottom, so that the quiet water at the edge of the river is the only area where organic silts can accumulate.

Utica-Rome Expressway, Rte. 49 Relocation

The Utica-Rome Expressway project is located northwest of the City of Utica, New York, and will connect I-790 to State Route 49. This involves the relocation of Rte. 49 to the south, in a position parallel to the New York State Thruway, and the expansion of Rte. 49 from a two lane, unlimited access road to a four lane, limited access highway. The length of the project is approximately four miles.

The project is situated entirely within and along the Mohawk River Valley. The terrain ranges from relatively flat within the valley to gently rolling on the terraces along the valley. The terraces are composed of granular glacial outwash, remnants of outwash plains on the valley sides which have been eroded. The central part of the valley was once the site of a glacial lake. The glacial lake deposits consist of interbedded silt, clay, and fine sand. The present river flows over the glacial lake sediments, and has deposited Recent alluvial sediments over a wide valley bottom. The alluvium has areas of poorly drained soil with high water table conditions. Within these areas are pockets of organic silt and interbedded organic silt and peat. These soils have moisture contents ranging from 60% to 140%. The flora at the site consists of wetland grasses and reeds.

The Rte. 49 relocation requires the construction of six new structures, along with mainline and ramp embankments. Organic silts were encountered at many of the proposed embankment locations. There were a number of shallow auger holes progressed along one proposed embankment for the eastbound lane of Rte. 49, at Station 17+500 to Sta. 17+860, but many of them did not penetrate the full thickness of the organic deposits. Because of uncertainty as to whether the drillers could be re-mobilized at the site to drill through the organic sediments, the decision was made to try seismic refraction as a "quick strike" method to find the thickness of the organics.

A preliminary seismic array was set up between two boreholes in another section of the project which had similar organic silts. The boreholes, DAX-162 and DAX-166, showed 20 feet and 15 feet, respectively, of dark brown organic silt with pieces of wood and fibers over silty fine sand. The seismic array was set up using 12 geophones, spaced 10 feet apart. The geophone cable was connected to a Bison Geopro 8024 seismograph, and dynamite was used as the energy source. When the seismograph data was analyzed, it showed material having a seismic velocity of 1100 feet per second over material with a seismic velocity of 4900 feet per second. The low velocity material had a thickness that agreed to within one foot of the organic silt thickness in the boreholes.

The next step was to shoot two seismic arrays in the area of Sta. 17+500 to Sta.17+860. The shallow boreholes had encountered organic silts in a swamp in the middle of the section of proposed roadway. Two seismic double arrays were set up in the swamp, a short distance from the Thruway. Because of the proximity of the heavy Thruway traffic, high-frequency filters were applied to the seismic record. This is because traffic generates a higher-frequency vibration than dynamite does, so that the filters reduce the traffic noise and do not degrade the seismic signal The data showed a layer of low-velocity material, 12 to 16 feet thick, overlying higher-velocity alluvial material (Fig. 4). The entire seismic survey required one day of field work, including mobilization time, to give the thickness of the organic silt layer at six points on the project, at a spacing of about 130 feet between points along the center line.

Drury Lane Connector, Stewart Airport

The proposed Drury Lane Connector under design will increase the number of entrance routes to

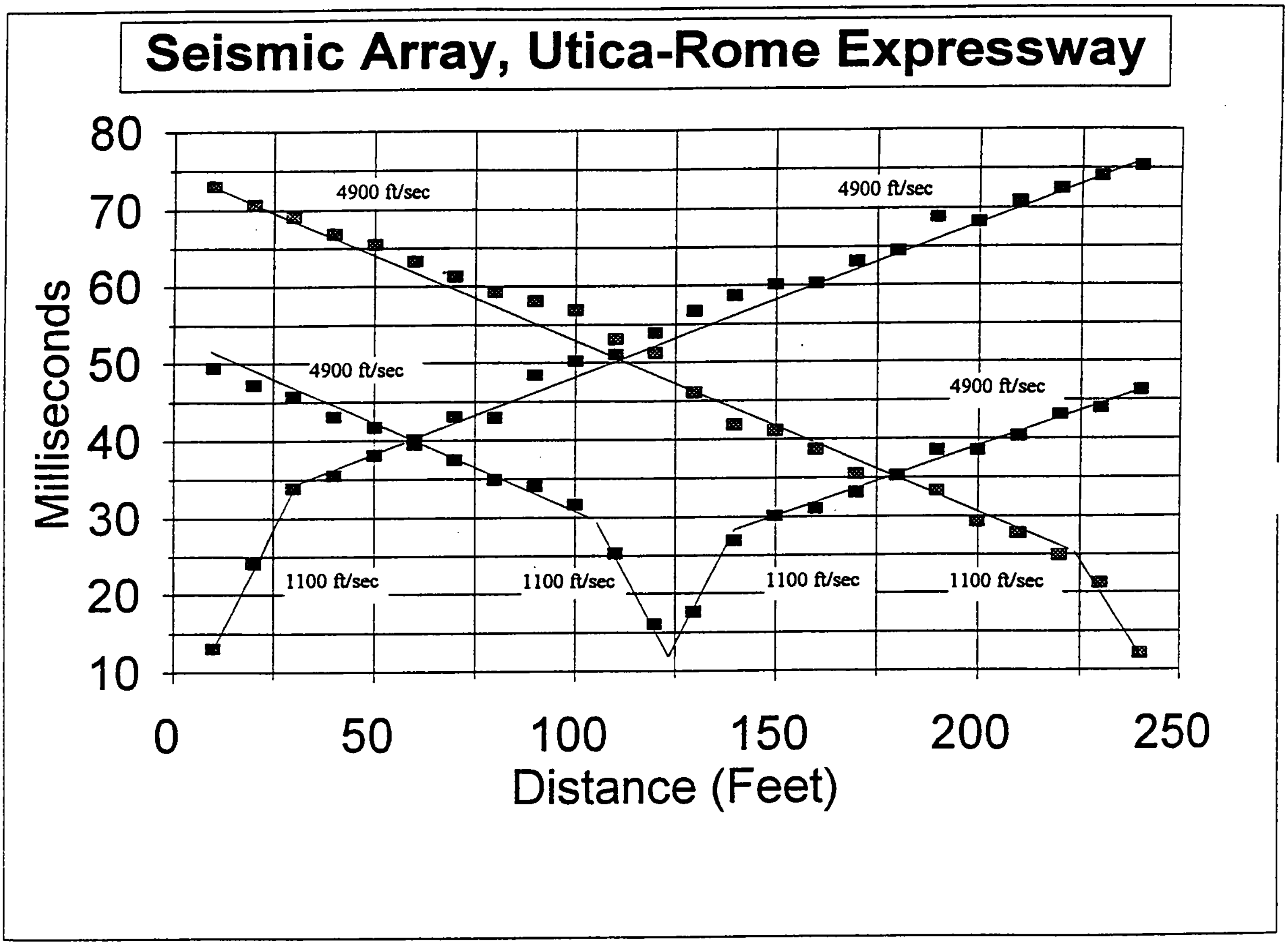

Seismic Array, Utica-Rome Expressway
Milliseconds
Distance (Feet)
80
70
60
50
40
30
20
10
0
50
100
150
200
250
4900 ft/sec
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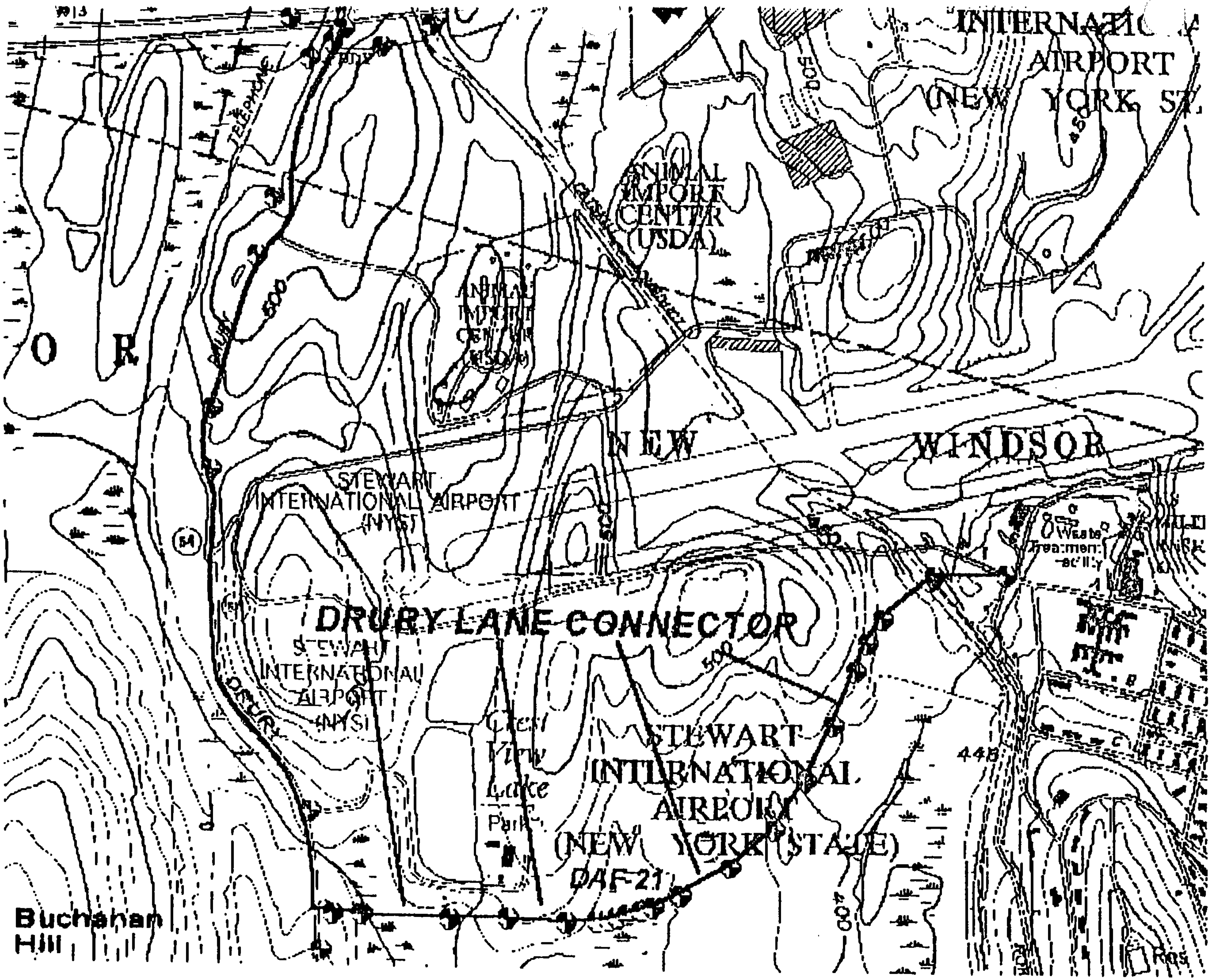
Figure 4



Druryl Lane Connector

Drill Hole Locations

Figure 5. Drury Lane Connector, Stewart International Airport

Stewart International Airport. The airport is located near Newburgh, New York, about 50 miles north of New York City. The proposed access road will begin at a new exit off I-84, go south along two miles of Drury Lane (County Rte. 54), and turn east to Stewart Airport (Fig. 5). The access road between Drury Lane and the airport is on a new alignment, approximately 1.5 miles long.

The geology of the project area consists of till drumlins, with areas of poorly drained thin till over shale bedrock between the drumlin hills. One of the new alignment segments, located between two drumlins, contains a pocket of thick peat and muck. A number of borings were progressed in the peat deposit, and showed the extent of the organics to be about 500 feet along the center line of the new alignment. The greatest thickness of peat was encountered by borehole DAF-21, which showed 20 feet of black peat over till. The peat has a moisture content of 500% to 750%, which indicates that it is a nearly pure deposit of decomposed plant matter with very little mineral soil. The present plant life consists of old forest hardwood trees, ferns, mosses, and skunk cabbages.

This peat is probably the result of a glacial kettle infilling which accumulated after the retreat of the ice, about 14,000 years ago, and is classified as a bog. A kettlehole bog typically originates in a glacially formed depression. If the bottom of the kettle is sealed by material having a low permeability, water and plant matter accumulate. The process of filling the kettlehole is very slow, and may take thousands of years (Stone et al., 1994).

Two seismic arrays were set up in the peat bog, perpendicular to the direction of the road alignment and tied to two of the boreholes. Unlike the Utica survey, however, the seismic data did not show a gas velocity in the organic layer; instead, the organics exhibited the velocity of water. This may be because the plant matter, which has been accumulating for such a long period of time, is nearly all decomposed, unlike the Recent alluvial deposits found at the Utica site. If this is the case, the biochemical reactions involved in the production of carbon dioxide would, for the most part, have gone to completion.

The seismic arrays at the Drury Lane Connector were double arrays, each having shots at each end and one at the center, for a total of six seismic points. Using the water velocity for the peat and the local glacial till velocity of 6800 feet per second, the depth of the organic layer was calculated. At two of the points, there was a velocity inversion, because there was a material having a velocity lower than water below the organic. This may have been the result of a pocket of gas trapped below the high-moisture muck. At these two locations, the depth of the organic layer could not be calculated. At the other four points, the calculated depth agreed with the depth from the boreholes, and additional information about the lateral extent of the organic deposit was acquired.

Conclusions

Seismic refraction has long been a valuable tool for subsurface exploration when finding depth to rock for design of transportation systems. Its use can be expanded to defining the thickness and extent of organic deposits. Using refraction for this purpose requires some knowledge of the nature of the organic deposit (for instance, organic silt versus peat or muck), so that one may know whether to expect an air velocity or a water velocity from the organic deposit. Seismic refraction as a method for determining the presence of organic deposits would be most useful when done as part of a terrain reconnaissance survey in the early stages of design, so that subsequent boreholes could be located to the best advantage.

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CURRENT AND POTENTIAL USES OF TIME DOMAIN REFLECTOMETRY FOR GEOTECHNICAL MONITORING

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ABSTRACT

The California Department of Transportation successfully used Time Domain Reflectometry (TDR) in a number of case studies to monitor the movement of landslides and embankment failures. This application of TDR technology uses a cable tester and a coaxial cable grouted in a borehole. TDR measures changes in cable impedance to determine the location of shearing, tension, or a break in the cable. The authors also deployed a remotely accessed TDR system (cellular phone, modem, data logger, and cable tester) to monitor landslide movement.

There are several advantages to TDR over the standard inclinometer technology. There is a cost advantage: \$0.16/foot (\$0.52/meter) for certain cables versus up to \$5.50/foot (\$18/meter) for inclinometer casing (the cost of the cable tester is comparable to the inclinometer probe and readout unit). The TDR reading is taken at the surface end of the cable, whereas the inclinometer probe is lowered down the casing and is occasionally lost. A TDR reading takes only a few minutes, regardless of length, compared to inclinometer readings which can be time consuming. If the monitoring location is inconvenient or unsafe, such as in a roadway, the cable end can be extended to a more convenient and safe "reading" location off the roadway, preferably behind a guardrail or other barrier. This also eliminates the traffic control which would be required for a inclinometer reading. Cables can be installed in smaller diameter boreholes, or in inclinometer casings that have deflected so much that the probe is blocked and readings can no longer be taken. Finally, readings can be taken on cables installed in borings at any inclination, horizontal through vertical.

The authors' research and experience suggest a number of additional geotechnical monitoring applications for TDR. For example, foam or air-filled coaxial cables can be used to monitor groundwater levels. The presence of water in the dielectric produces a characteristic change in cable impedance. Horizontal cables buried in shallow trenches can be used to monitor potentially unstable slopes and embankments. With the addition of some computer software, the remotely accessed TDR equipment could be used as a landslide warning system. Finally, rockfall barriers could be remotely monitored with TDR for rock impacts and damage.

INTRODUCTION

Landslide repair and mitigation require information that can be obtained readily. The direction of slide movement can be determined by mapping the surficial features of the landslide. The amount and rate of slide movement can be measured by establishing survey points and/or installing extensometers. The depth to the slide plane, slide direction and movement rate can be determined by installing an inclinometer.

Inclinometers are the most widely-used method of monitoring unstable slopes. They measure the offset, over time, of an oriented slotted pipe placed in a borehole. Readings are made with a probe that is lowered into the casing. The time required for data acquisition increases with hole depth and closeness of reading spacing. The data is then entered into a computer and analyzed to determine the pipe deflection. In most cases, the depth to an existing failure plane is the most important information derived from the inclinometer data. That data is used to determine the shear strength of the slide mass by back-analysis and to develop a repair strategy.

Recently, a time saving and less costly alternative to inclinometers for landslide monitoring has been developed and is in use by the California Department of Transportation (Aston, 1994; Dowding et. al., 1988; Kane and Beck, 1994; Kane et al., 1996)). This alternative monitoring system, consists of a coaxial cable inserted in a borehole or trench and a cable tester to determine deformations or breaks in the cable. Figure 1 is a diagrammatic representation of this method. It is called time domain reflectometry (TDR).

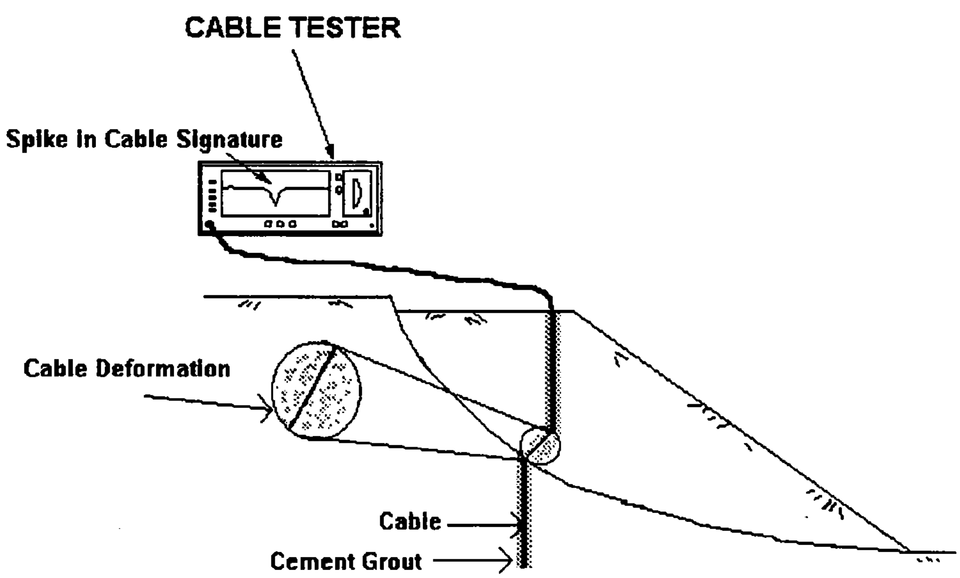

Figure 1. Cable Tester and Coaxial Cable for TDR Monitoring of Landslide Movement (Anderson et al., 1996)

TIME DOMAIN REFLECTOMETRY

TDR was originally developed by the power and communications industries to find breaks or damage in cables. In TDR, a voltage pulse is sent down a cable. If the pulse encounters a change in the characteristic impedance of the cable, it is reflected. This can be caused by a crimp, a kink, the presence of water, or a break in the cable. The returned pulse is compared with the emitted pulse and the reflection coefficient (in rho's or millirho's) is determined. If the reflected voltage equals the transmitted voltage, the reflection coefficient is +1 and the cable is broken. If the opposite occurs, and the cable is shorted, all the energy will be returned by way of the ground and the reflection coefficient will be -1. Deformation of the cable, usually shear or tension, will change the impedance and the reflection coefficient will be between -1 and +1.

Electrical energy travels at the speed of light in a vacuum. The speed at which it travels in a cable is less, depending on the impedance of the cable. This speed is known as the velocity of propagation and is a property of each cable. When the cable propagation velocity and time delay between transmitted and reflected pulses are known, the distance to any cable deformation can be determined.

Coaxial cables are composed of a center metallic conductor surrounded by an insulating material, a metallic outer conductor surrounding the insulation, and a protective jacket (See Figure 2). All cables have a characteristic impedance determined by the thickness and type of insulating material between the conductors. This insulating material is called the "dielectric" and may be made of almost any non-conducting material. Common dielectric materials are PVC-foam, Teflon, and air.

If the cable is deformed, the distance between the inner and outer conductors changes as does the impedance at that point. The TDR cable tester determines the location of this change.

TDR data consist of "signatures" which show cable impedance versus length (See Figure 3). Cable shearing results in an abrupt impedance spike and while tension results in a pronounced trough. The length and amplitude of the spike indicate the severity of the damage to the cable. Determining ground movement with TDR requires reading the cable signature at regular time intervals. Ground movement, such as slip along a failure zone, will deform the cable and result in a change in cable signature. This change can be used to determine the position of failure and the increase in impedance with time will correspond qualitatively to the rate of ground movement as shown in Figure 3.

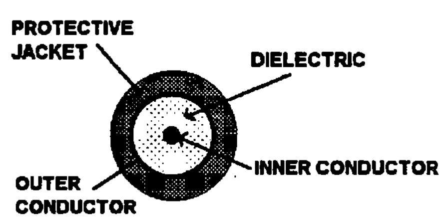

Figure 2. Coaxial Cable Cross-Section

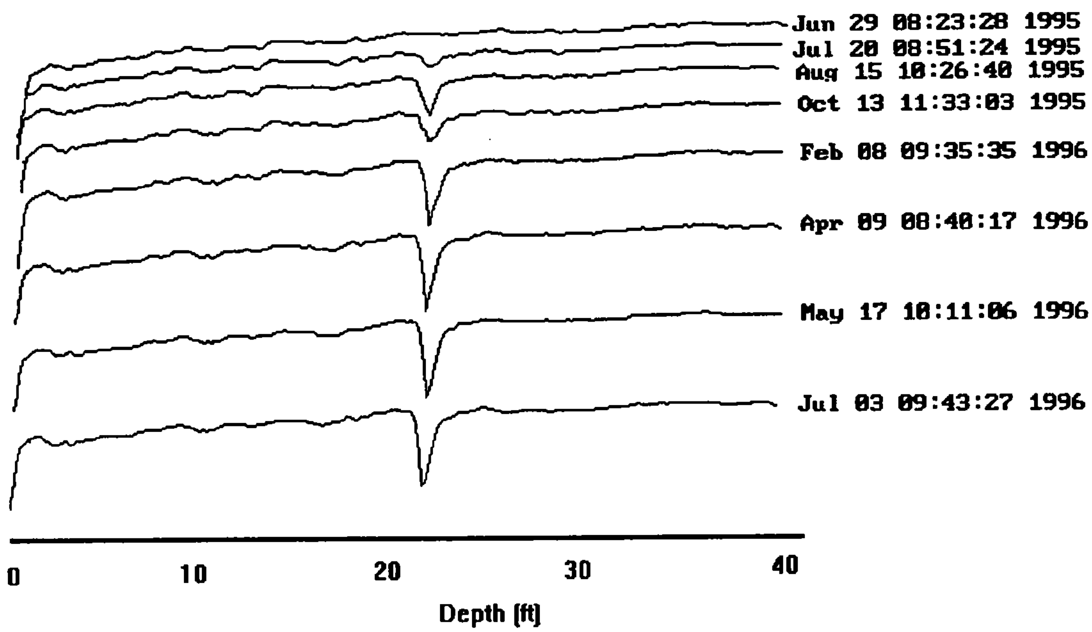

Figure 3. Typical Cable "Signatures" Show Growth of Spike with Continued Deformation (Anderson et al., 1996)

CALIFORNIA DEPARTMENT OF TRANSPORTATION TDR INSTALLATIONS

The California Department of Transportation (Caltrans) has begun using TDR in conjunction with inclinometers in their slope failure investigations (See Figure 4). Described below are a number of Caltrans TDR installations that highlight some unique features of using TDR for geotechnical monitoring.

Willits Landslide

A small landslide damaged State Highway 20 in Mendocino County just west of Willits (See Figure 4) during the winter of 1996. The failure also damaged a private road below the highway. Three inclinometers were installed in this slide. Coaxial cables were taped to the outside of the inclinometer casings.

Because a TDR measurement is taken at the surface, it has a distinct advantage over inclinometers . For example soon after installation at Willits, one inclinometer was offset so much that the inclinometer probe could no longer be lowered past that point. The cable attached to the casing was still intact and the TDR signature had a sharp spike at the depth where the casing was offset (See Figure 5). Although the inclinometer is no longer functional, the cable continues to provide information on the depth to failure plane and a qualitative feel for the rate of movement.

Figure 4. Locations Where Coaxial Cables Were Installed For TDR Measurements

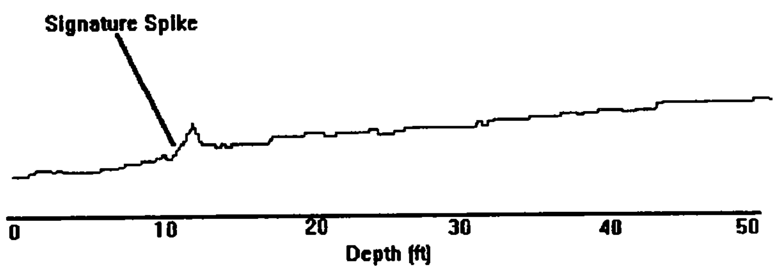

Figure 5. Cable Signature from Willits Landslide Showing Spike at Location of Inclinometer Failure

Redwood National Park Landslide

The Redwood National Park landslide is a failure of the cut slope adjacent to Highway 101 in Humboldt County just south of the town of Klamath (See Figure 4). In the winter of 1996, the debris from this slide blocked the northbound truck lane.

The terrain around the slide is steep. Drilling holes to obtain subsurface samples and install inclinometers will require pioneering a road into the slide area. Since this slide is in a sensitive region (national park) obtaining the necessary drilling permits is time consuming.

A large diameter borehole is not necessary to install a coaxial cable for TDR. Cables can be installed in a small diameter hole while doing routine soil investigations. In order to get some preliminary subsurface data, three one inch (25.4 mm) soil probe holes were driven into the slide with a "Wacker" - a hand-held, gasoline powered hammer. The hole depths ranged from 20 to 26 feet (6.1 to 7.9 meters) deep. In two of the holes, coaxial cables were installed through the center of the probe rod as it was withdrawn.

Devil's Slide

Devil's Slide is massive slope failure in San Mateo County south of the town of Pacifica (See Figure 4). The first historic record of this slide indicates that it was caused/reactivated by the 1906 San Francisco Earthquake. State Highway 1 was built across this slide in the early 1940's and movement of the slide has periodically damaged the highway since then. In the winter of 1995, sliding caused significant damage to the roadway which made it impassable for several months while repairs were made.

TDR can save time when monitoring deep holes or multiple locations. In the winter of 1996, as part of Caltrans' ongoing monitoring of the slide, an inclinometer 356 feet (111 meters) deep was installed in the slide. A coaxial cable was attached to the outside of the inclinometer casing. The inclinometer shows a shear zone from roughly 124 to 164 feet (38 to 50 meters) below the ground surface. The cable shows an impedance spike at a depth of roughly 144 to 152 feet (44 to 46 meters). A TDR reading of the cable takes about five minutes. In contrast, the inclinometer reading takes at least an hour.

South Willow Creek Landslide

This installation is a good example of the cost advantage of TDR. The South Willow Creek Landslide is a moderately sized failure of the sea cliff in Monterey County along the Big Sur coast (See Figure 4). This failure removed a portion of the southbound lane of State Highway 1. An inclinometer was installed in the slide to a depth of 56 feet (17 meters). A coaxial cable was attached to the outside of the casing. The coaxial cable purchased for this project was $0.16 per foot while the inclinometer casing cost $5.50 per foot, or about 35 times greater than the TDR cable.

Cuesta Grade Landslide

The Cuesta Grade Landslide is in San Luis Obispo County north of the city of San Luis Obispo (See Figure 4). This slide is an embankment failure that damages a section of the southbound lanes of Highway 101. This embankment has a 30 year history of instability.

The traffic is extremely heavy on this section of Highway 101. An inclinometer was installed in the roadway because of the narrow shoulder at this location. Reading the inclinometer requires traffic control and is hazardous for the technicians involved.

To increase technician safety by reducing the number of inclinometer readings needed to monitor this slide, a coaxial cable was installed during May 1996. The cable was attached to a 1.5 inch (38 millimeter) tremie tube grouted in a 94 foot (28.7 meters) deep borehole, and was extended off of the road behind a guard rail. In this instance, Caltrans was able to use TDR to increase safety while not jeopardizing data collection.

Grapevine Grade Landslide

The Grapevine Grade Landslide is in Kern County south of Bakersfield, California (See Figure 4). This slide is a failure of the cut slope next to Interstate Highway 5, a major transportation route between the Mexican border and Oregon. The toe of this slide daylights in the northbound truck lane. The head scarp was already visible in aerial photographs taken in April 1992. By April 1993, the toe had moved enough to rupture a buried oil pipeline.

In the winter of 1995, the California Department of Transportation installed geotechnical instrumentation (inclinometers and piezometers) to determine the slide depth and monitor groundwater levels. Coaxial cables were attached to the outside of the casings on two of the inclinometers and one of the piezometers.

The Grapevine location is in a steep and remote area. Two technicians have been assigned the task of reading the inclinometers. Driving from their office to the site requires two hours. At the site, a steep and difficult climb is required to access the instrumentation locations. All told, reading the instrumentation at this site requires a full day from both technicians.

A remote data acquisition system was installed to do the TDR readings at this site. The system consisted of a cable tester, data logger, multiplexer, cellular telephone, modem, solar panel and battery (See Figure 6). The system allows TDR readings to be taken by computer and modem from anywhere with telephone access.

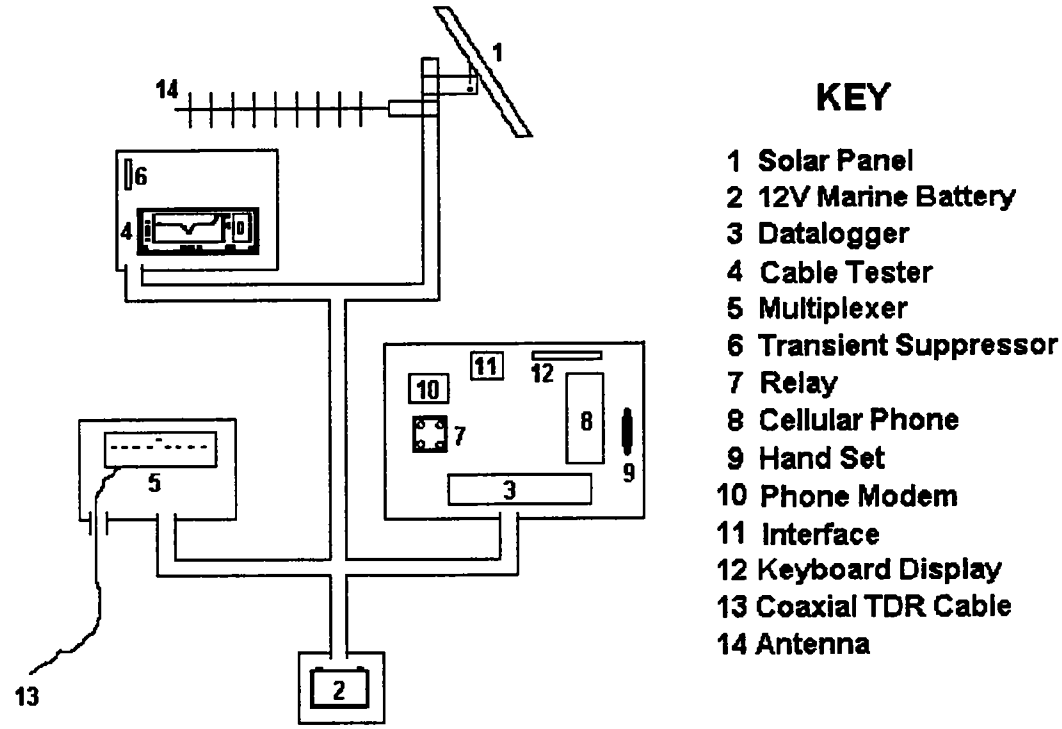

Figure 6. Remote TDR Data Acquisition System

POTENTIAL APPLICATIONS OF TDR TO GEOTECHNICAL MONITORING

Caltrans is currently considering additional applications of TDR in geotechnical monitoring. The following is a brief discussion of those applications.

When an inclinometer is installed in an active slide, the deflection of the casing eventually reaches a point where the probe will no longer will pass through it. If the casing is still open, a coaxial cable can be inserted past the bend to the bottom of the casing and grouted in place. In this way, the monitoring life of the borehole can be extended.

Due to the restricted access around many landslides, subsurface samples can only be obtained by drilling angle borings. Since drilling permits require most abandoned borings to be grouted anyway cables can be grouted in place and the holes used for monitoring. Gravity does not affect TDR readings. On the other hand inclinometer readings can have large errors if a boring deviates significantly from vertical (or horizontal for horizontal inclinometers).

The presence of water in the cable dielectric changes its impedance (Dowding and Huang, 1994). By selecting a cable whose dielectric allows rapid water infiltration and has a minimal capillary effect, TDR could be used to monitor groundwater levels. One such cable is currently being manufactured, it has an air-filled helical chamber that serves as the cable dielectric.

Frequent site visits are required where private property or public safety are threatened by continued slide activity. Examples are where the uphill migration of a head scarp would engulf a private residence or where continued sliding would remove part of the roadway. The number of site visits could be substantially reduced by installing a monitoring system consisting of a coaxial cable grouted in a trench and cable tester with remote data acquisition system. Reduced site visits will result in a significant time saving. A computer program could be written and combined with the remote data acquisition system to create a landslide warning system

Flexible wire rope barriers are often used to protect highways from rockfall. These barriers typically are in remote locations, and site visits are required to assure that they are not damaged or overloaded. The number of visits to the barriers could be reduced by attaching coaxial cables and a remotely-accessed TDR system

CONCLUSIONS

The reliability of TDR technology in determining landslide depths has been demonstrated by the six slopes described in California. TDR cables can be used to reduce or eliminate inclinometer installations in landslides.

TDR costs less and saves time when compared to inclinometer technology. Coaxial cable can be less than 5% of the cost of inclinometer casing. TDR readings require less than 10% of the time it takes to collect inclinometer readings. Remotely accessed TDR systems save additional time by eliminating site visits and the travel time required to collect geotechnical data.

Beside the economic advantages, the safety benefit of collecting TDR readings from the cable end justifies its use for landslide monitoring. In situations where monitoring locations only can be placed in the roadway, TDR can be extended off the roadway. This eliminates the need for traffic control and thereby increases worker safety.

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INSTRUMENTATION OF A SHREDDED TIRE FILL USED FOR A LANDSLIDE REPAIR, "THE BURNING ISSUE"

B. Boundy, P.G.
J. Dahill, P.G.

ABSTRACT

A project involving the use of shredded tires as lightweight fill material for control of an active landslide was completed in the summer of 1995. The Double Nickel Slide is located along State Highway 28, at milepost 55, approximately 20 miles south of Lander, Wyoming. The slide repair consisted of a shift in the centerline alignment and construction of a lightweight embankment using shredded tires. In light of the tire wall and shredded tire fill fires which developed in Colorado and Washington, two thermistor strings and three inclinometers were installed as part of a long term monitoring research program. Geology personnel will also test the air and water emanating from the underdrain that runs along the bottom of the shredded tire fill for by-products of an exothermic reaction.

The Double Nickel Slide has been active since 1987 when the highway was realigned and upgraded to meet primary roadway design standards and specifications. A contract was let in 1994 to remediate the landslide, utilizing several earth stabilization techniques. These included the use of heavy rock toe berms to increase resisting forces, an alignment shift to reduce the embankment height which in turn lowered the driving forces, along with the construction of a box culvert over the spring to facilitate drainage. Construction of an eleven foot high fill using 8,300 cubic yards of shredded tires further reduced driving forces. The lightweight tire shred fill, which was constructed with roughly 500,000 tires, weighed 70 percent (10,000 tons) less than a similar volume of soil.

The tire fill is the subject of a monitoring program by the Geology Program of WYDOT. Inclinometers were placed below the roadway to detect any possible movement along the original slide plane and also in the tire fill above the road to confirm that the upper portion of the slide has been stabilized. A surveying program has been established to monitor settlement of the tires. Additionally, thermistor strings, consisting of temperature probes, have been placed in the tire fill to monitor any rise in temperature indicative of an exothermic reaction. A 30 inch surface area drainage culvert passes beneath the roadway directly under the tires and is also the outlet for the underdrain pipe located at the base of the tires. Geology personnel plan to enter the culvert and proceed to the outlet of the underdrain to measure the temperature, gases, and pH of the water; all properties that influence or are influenced by exothermic reactions. This paper summarizes the research monitoring to-date and the current conditions of the Double Nickel Slide repair. A contrast will be made between the site conditions of the Washington fires, Colorado's Glenwood Canyon retaining wall fire and the Double Nickel tire embankment.

BACKGROUND

Since it was reconstructed in 1985, a section of State Highway 28, south of Lander, Wyoming, has caused more than a few headaches for the state DOT officials. In an act of bold geotechnical engineering, a large fill was constructed over colluvial deposits from which a naturally occurring spring discharges. The spring flows 300 G.P.M. and is a registered State water right. An underdrain system at the base of the fill was designed to allow unrestricted flow of the spring, however, the drainage system was constructed improperly and did not function as it was intended. Stability problems soon developed and the road and fill began to drop. A head scarp developed in the backslope above the road and a toe bulge formed below the spring, evidence that a slide was threatening the roadway and integrity of the spring. Accelerated movement in late 1992 prompted construction of three heavy rock toe trenches which provided additional drainage and created resisting forces to stabilize the slide. An abundant source of heavy iron ore rock for use in the toe trenches was readily available from the nearby U.S. Steel Pit.

In early 1994, movement started to occur immediately east of the toe trenches. To save the road and the spring, the roadway alignment was shifted into the hill, additional toe beams were constructed, and a portion of the embankment was constructed with lightweight shredded tire fill (see figure 1). The recently completed $1.76 million contract may have finally ended the struggle with the stubborn slope-presuming the fill does not spontaneously combust.

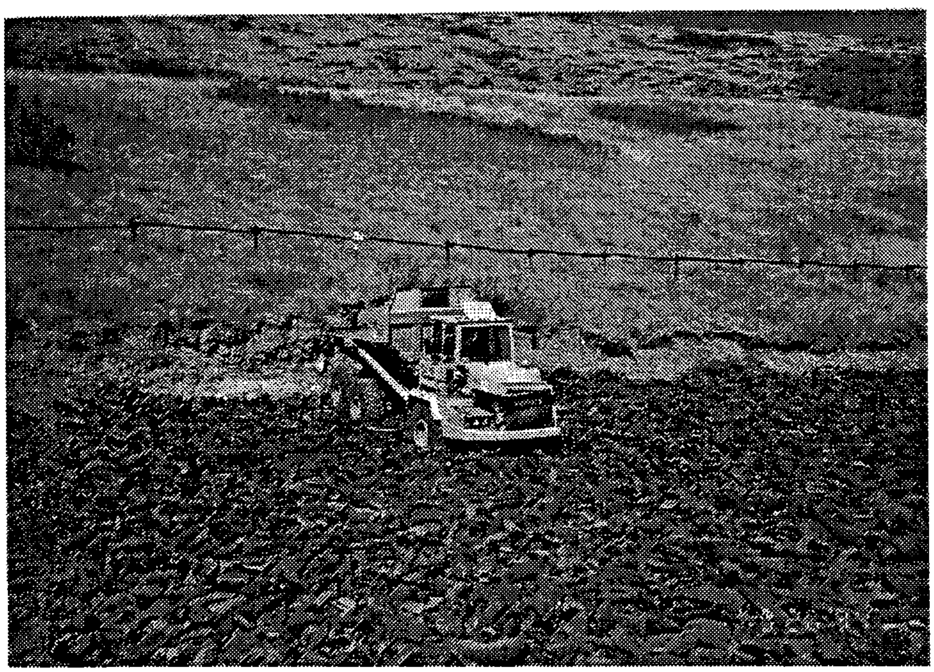


Figure 1: Placement of Tire Shreds

B. Boundy
J. Dahill

INTRODUCTION

Due to the experimental nature of the shredded tire fill, and reports of fires developing in similar shredded tire applications in Colorado and Washington, the Geology Program of WYDOT established a monitoring program. Two thermistor strings were installed in the shredded tires to monitor temperatures, and several surface survey points have been established to determine settlement above the shredded tire embankment. Three inclinometers were installed to monitor movement within the tires and in the underlying failure surface. Geology also plans to measure the pH of the water flowing from a 6 inch underdrain (unrelated to the spring drainage system) immediately beneath the shredded tires. Personnel will also test for gases indicative of exothermic reactions, and make additional temperature measurements of the shredded tires through drill holes in the 30 inch culvert which passes through the fill (see figure 2).

In 1994 (long before reports of oozing, smoldering chipped rubber fills), shredded tires appeared to be the answer to all our problems at the Double Nickel Slide. At a cost of $4.00 per cubic yard, shredded tires were more economical than other lightweight fill materials including wood chips ($8 to $15 per cubic yard) and expanded polystyrene blocks ($30 to $50 per cubic yard). Including the haul, the in-place cost of tires was $26.00 per cubic yard.

According to a detailed stability analysis, the section of shredded tires would need to be 15 feet thick to provide a 1.5 factor of safety when constructed in conjunction with additional toe trenches and an alignment shift into the hill. Although the alignment was shifted, the embankment still covered the spring, therefore a large, three sided concrete box (open at the front and the bottom) was designed to allow the spring to emerge from the ground under the box and flow unrestricted out from under the fill. At 10 feet high and 20 feet wide, the box was designed to be large enough to allow inspection of the spring. Shredded tires were placed directly over the rear of the box to reduce the overburden pressures on the box.

To allow for better compaction, a maximum 4 inch tire shred width was specified, but the tire shredding machine was unable to achieve this. Generally the pieces were over 4 inches wide, and in some cases tires were only sliced in halves or thirds. When the machine cut the tires to specifications, the shreds were comprised of very long strips derived from the sides of the tire (see figure 3). From an engineering standpoint, compaction was excellent with the ratio of unconsolidated rubber scrap to in-place volume estimated at nearly 3:1. A late spring storm dropped two inches of snow on the newly placed tire shreds and facilitated compaction.

With a total Wyoming population of under 500,000 people, an entire year's supply of discarded tires was consumed in the project. The shortage of available tires resulted in the thickness of the lightweight fill being reduced from 15 to 11 feet.

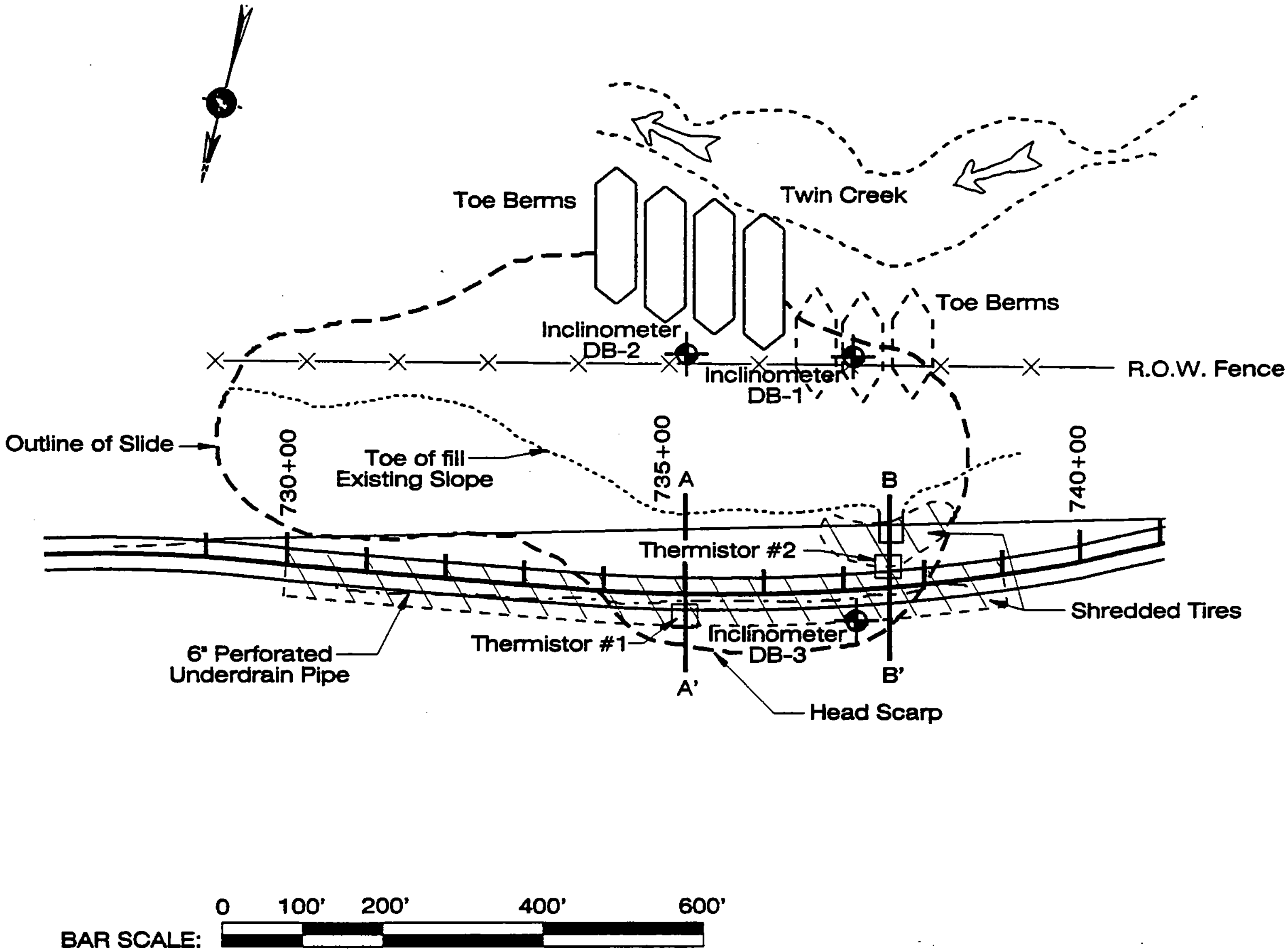


Figure 2. Plan View and Instrument Locations

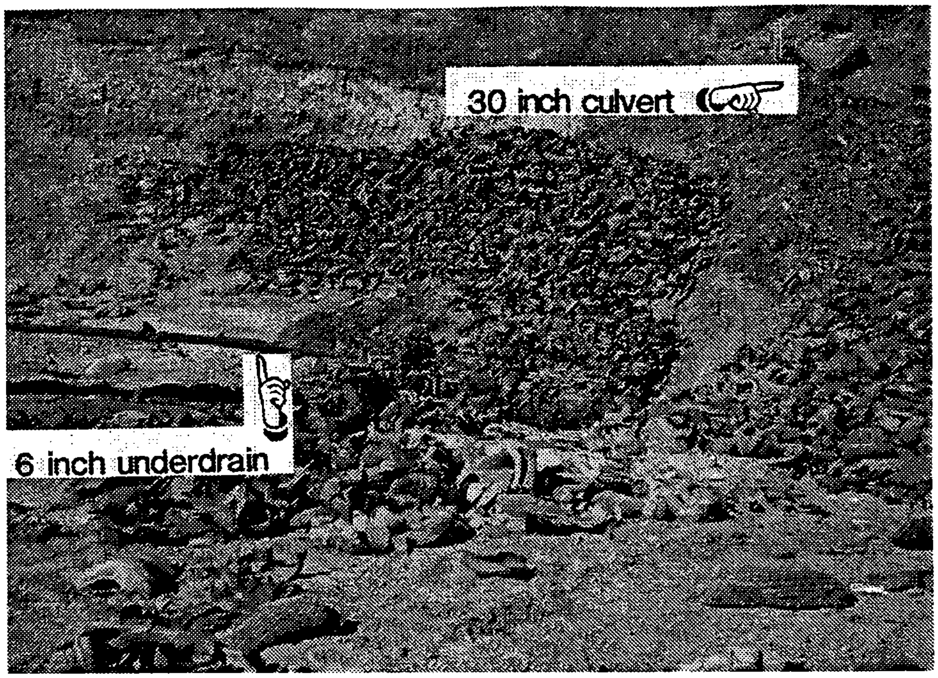


Figure 3: Tire Shreds

POSSIBLE MECHANICS OF SPONTANEOUS IGNITION

There are several possible exothermic reactions that are discussed in Dr. Dana N. Humphrey's report, "Investigation of Exothermic Reaction in Tire Shred Fill". The first, and probably most important, involves the oxidation of exposed steel wires. Every pound of steel that is oxidized produces approximately 2, 620 Btu's. By comparison, a typical piece of low sulphur Wyoming coal produces roughly 10,000 Btu/pound. Every cubic foot of tire shred fill contains about 5 lbs. of steel, however, only a small percentage of this is immediately accessible to oxygen. This reaction and subsequently, the heat liberated, is increased by environmental factors including acidic conditions, higher temperatures, the presence of organic materials and dissolved salts, and the tire cutting method which leave large amounts of exposed steel wire. (Humphery, 1996)

Oxidation of sulphur by bacteria is another type of exothermic reaction which produces up to 7,900 Btu/pound of sulphur. Most of the sulphur in tires is bonded with rubber, so this is probably not a major source of heat, however, sulphuric acid is a product of the reaction, which would lower the pH and increase the rate of oxidation of the steel. Other exothermic reactions include oxidation of rubber (unlikely below 250 degree Fahrenheit) and microbes consuming petroleum products (unlikely since free petroleum products are very diffuse in tier fills unless petroleum products were spilled during construction). (Humphrey, 1996) All of the above reactions will be considered with respect to the tire shreds placed at the Double Nickel Slide and compared to the recent Washington and Colorado fires.

Another potential mechanism for spontaneous ignition involves the minimum overall thickness of tires, as well as a maximum shred size. Above a certain thickness of tires, the heat loss is less than the rate of heat production by exothermic reaction. Smaller shred sizes can be compacted to a greater overall density resulting in excellent insulating properties as well as providing a higher percentage of steel belt ends. Results of our investigation indicate hot spots have only developed in shredded tire stockpiles more than 15 feet high, and only when the material was composed of 2 inch tire chips. Similar sized stockpiles consisting of 4 inch tire shreds have not been reported to combust. The two shredded tire fills in Washington which have spontaneously combusted were both over 25 feet thick and were subjected to several aggravating factors.

The introduction of organics into the fill may act as a significant catalyst in the spontaneous combustion of shredded tire fills. At one Washington tire fill, topsoil from a cranberry bog was placed on the upper part of the side slope directly on top of the tire shred fill. The topsoil was hydro seeded with a mixture of seed, fertilizer, and mulch. The zones of the fill that are experiencing the exothermic reaction appear to be beneath this area of topsoil. At the other Washington site, flood waters laden with organics, and possibly fertilizers from fields upstream, ponded against the fill shortly after it was constructed. It is probable that the copious amounts of water (i.e. floods, irrigation, or high rainfall) have leached the organics, phosphates, and nitrates into the tires. It is speculated that the organic material could enhance the oxidation of the steel belts. The nitrogen and phosphorous are inorganic nutrients necessary for the types of microbes that consume liquid petroleum products, another exothermic reaction. Although the percentage of free petroleum products in tires is probably too low to support significant microbial activity, any petroleum spills from equipment could be sources for hot spots to begin. (Humphrey, 1996) At the Colorado retaining wall site, a mixture of soil and compost was placed over the shredded rubber fill that was placed behind the pressed rubber facing blocks. It was specified that the mixture would have a minimum of 50% organics, 0.9% phosphorous, and 0.75% nitrogen with a moisture content of at least 20%. (Knott Laboratories, 1995)

By comparison, the topsoil in Wyoming is rarely organic. The topsoil at and above the Double Nickel realignment is no exception. It is predominately inorganic clayey silt and fine sand, distinguished from the underlying soil zones by the presence of sagebrush roots. There are no agricultural fields upstream and the topsoil was drill seeded, therefore, there is no major source of nitrates or phosphates.

Low pH values greatly increase the rate of the various possible exothermic reactions in a shredded tire fill. Soils in the area tend to be alkaline, however, the exact pH value will not be known until the upcoming air and water investigation is completed in August of 1996.

INSTRUMENTATION AND MONITORING

Two thermistor strings were installed in the tire fill. Each thermistor string consists of a cable which contains five separate temperature probes to give a profile of temperatures within the shredded tire fill and the soils above and below it. The cables were taped to PVC casing which allowed the probes to be placed at known depths. Covered, recessed readout boxes placed near the shoulder of the road allow all five channels to be read from the surface. One thermistor string was placed on the north side of the road, near the middle of the slide where the deepest layer of tires was placed (see figure 4).

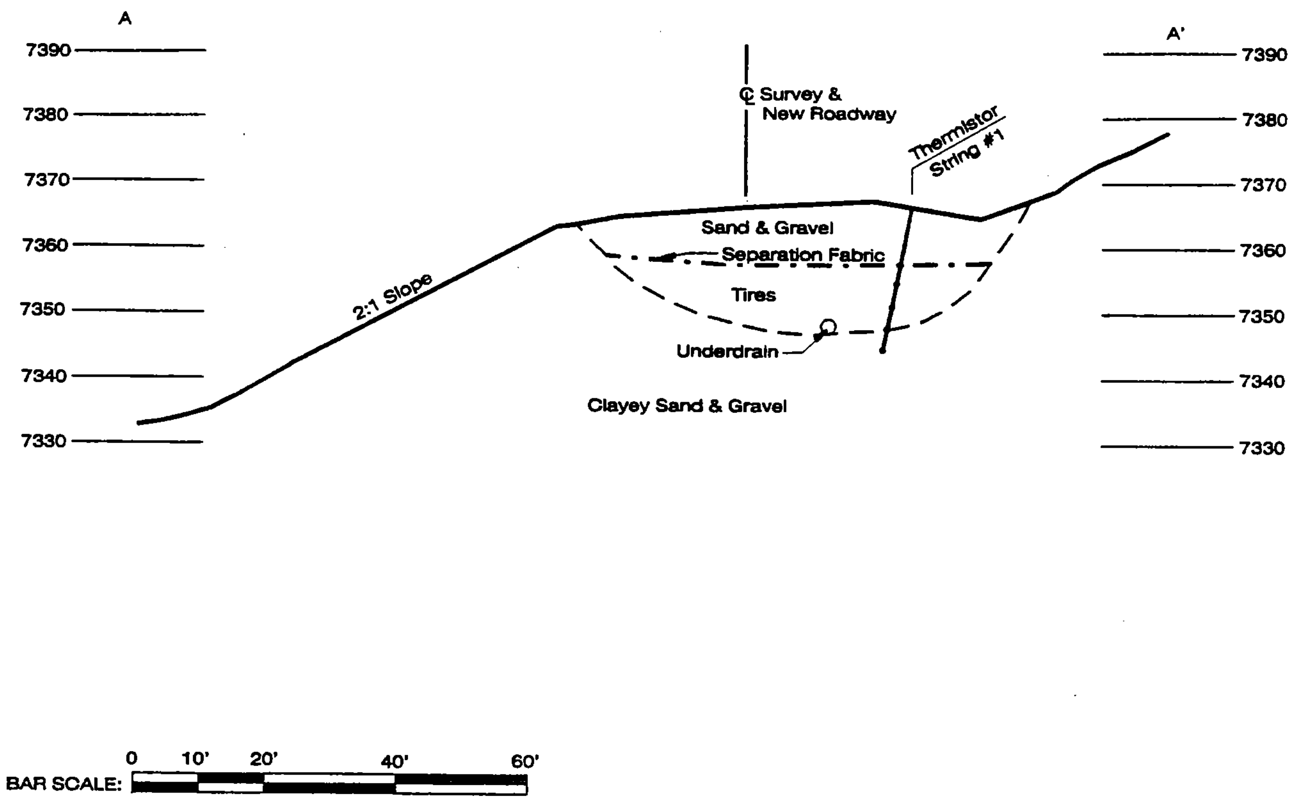


Figure 4. Cross Section Through Thermistor No. 1

The other was placed on the south side of the road, just behind the back wall of the box, which is a potential route for additional oxygen to enter the tires (see figure 5). The tires were found to be only slightly moist at thermistor site #1, but were moist to saturated at the site above the box. Eight and seven feet of tires were penetrated by thermistors 1 and 2, respectively. The initial readings showed temperatures as high as 160 degrees Fahrenheit within the tires, however, the high readings were caused by residual heat left from friction between the rubber and the augers during drilling. The temperatures were undoubtedly higher while drilling was proceeding.

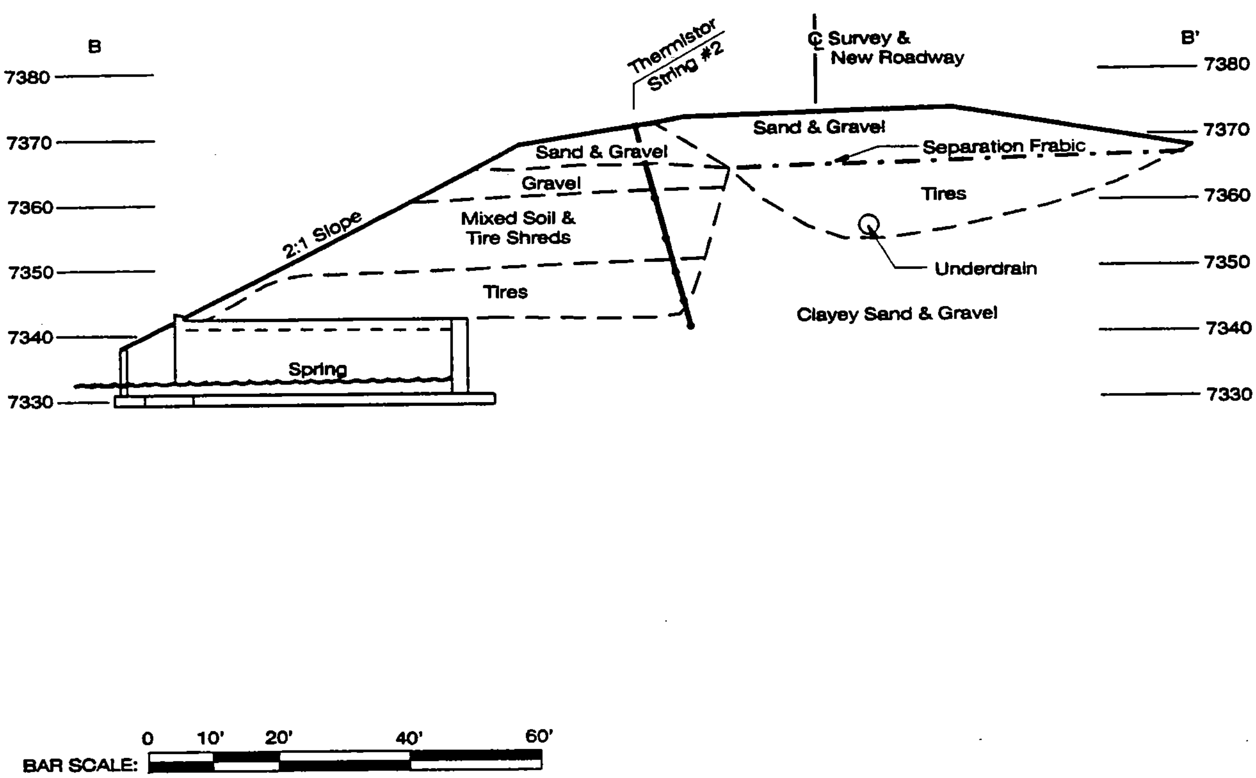


Figure 5. Cross Section Through Thermistor No. 2

Subsequent readings obtained once a month since December of 1995 have shown a steady decrease in temperature and are approaching the mean annual temperature of roughly 48 degrees, however, it is possible that scattered hot spots are present in the fill which are not penetrated by the two widely spaced thermistors. The attached graphs illustrate the temperature fluctuations of the thermistors at various depths to-date.

To search for evidence of a hot spot undetected by the thermistors, Geology personnel will test for heavier than air gas by-products of an exothermic reaction emerging from an underdrain that runs beneath the thickest section of tires along the length of the fill. Hydrogen sulfide, carbon monoxide, combustible hydrocarbons, and oxygen will be monitored for safety, as well as scientific reasons, since the outlet of the underdrain can only be accessed by entering a 30 inch culvert. If present, water emanating from beneath the tire fill will be measured for pH and compared to the pH of the natural runoff above the project. If the pH of water emerging from the tires is significantly lower than the pH going in, it is an indication that some reaction is occurring. The reaction could be either oxidation of sulphur or combustion of rubber, evidence that conditions exist to accelerate oxidation of the steel and further increase the heat output. We hope and expect that the pH's will be similar. Since the culvert passes through the shredded tires, it will also be checked for any evidence of heating. If possible, a temperature probe will be pushed up into the underdrain.

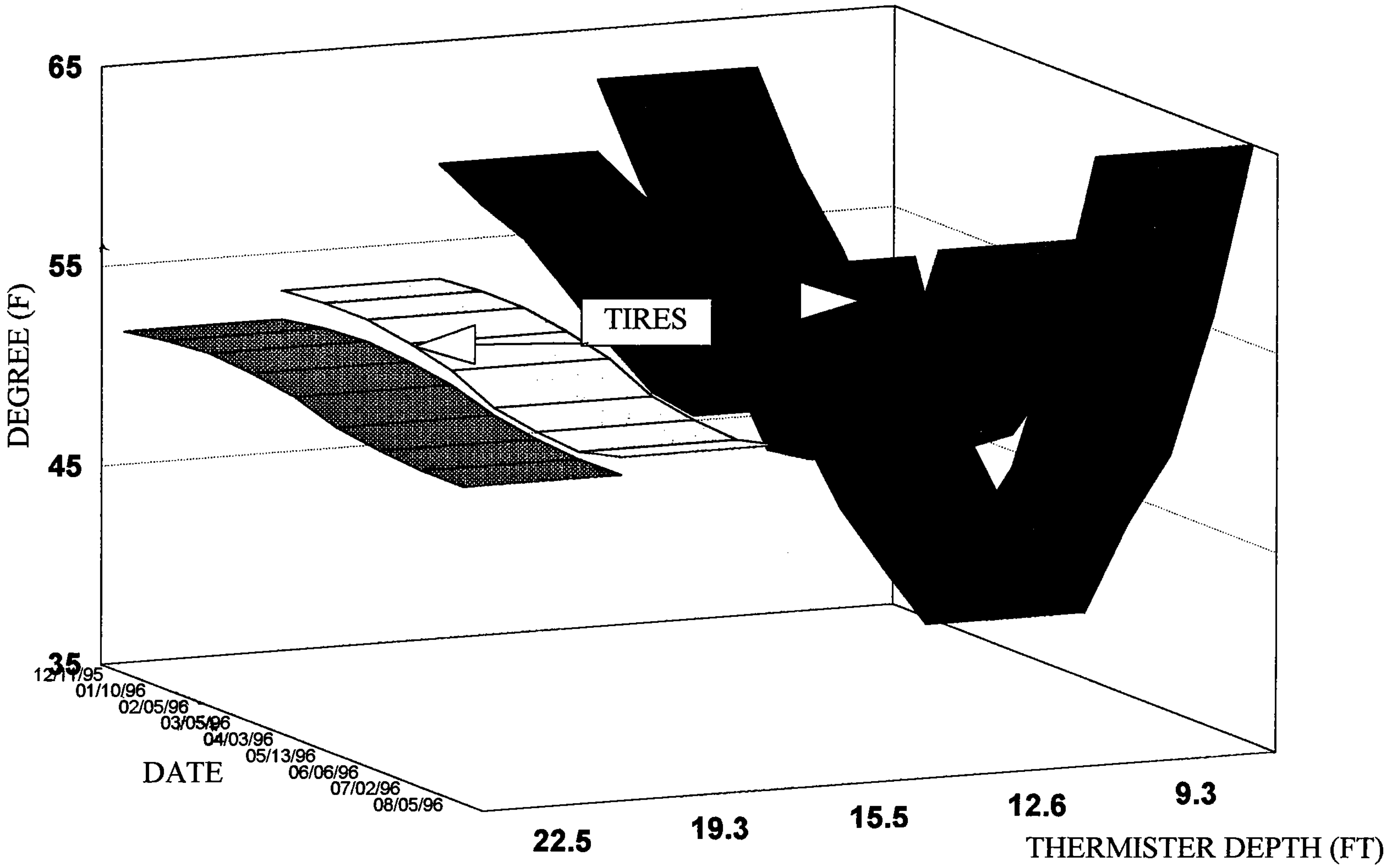
CONCLUSIONS

The realities of construction did not match the designed specifications, however, some of the shortcomings may have been beneficial. The use of larger tire pieces at the Double Nickel project may not have compacted as closely as the small chips that were used in the Washington and Colorado projects, resulting in a lower overall density and less rubber on rubber contact. The lower density will aid in heat dissipation produced by exothermic reactions within the fill, although it may allow more oxygen in the system. There is also less exposed steel than similarly cut but smaller pieces, which is an especially important factor since the edges of the cutting machine made ragged edges, fringed with long strands of wire. Finally, the thinner section of tires should allow better heat dissipation. Unless evidence of combustion is found in the upcoming air and water study, scheduled just prior to this year's conference in Cody, a correlation between the tires and thickness can be made.

We are cautiously optimistic that shredded tires less than 15 feet thick are still a good material for lightweight fills, and the insulating ability of the rubber in fills below this thickness will not be sufficient to allow significant heating by exothermic reactions. To date, there has been only minor settlement over the tire fill (less than a tenth of an inch), no discernable movement along the previous failure plane, and water continues to flow from the spring at over 300 G.P.M. Our goal of learning more about shredded tires as an alternative lightweight fill appears bright.

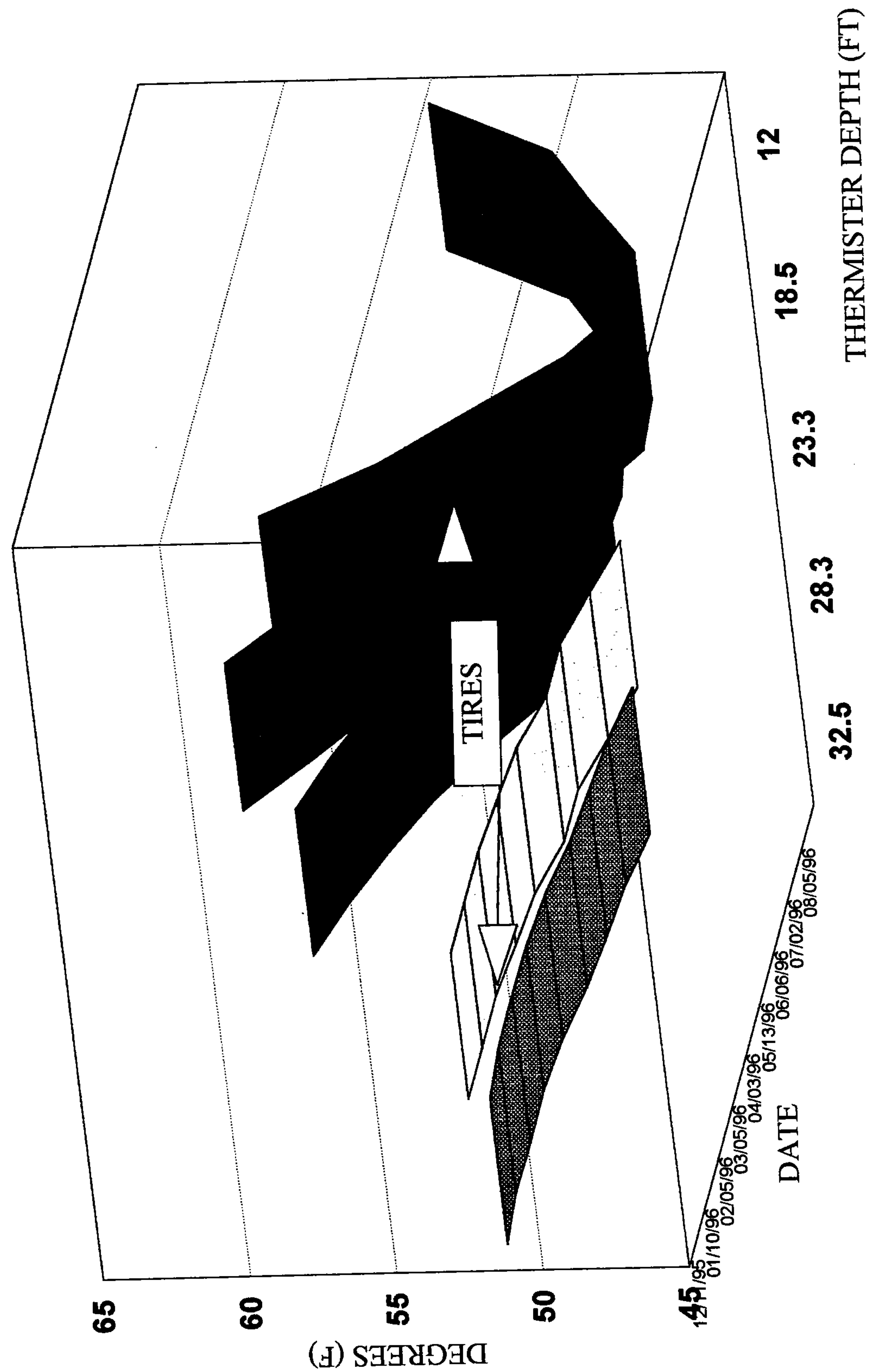
TEMP READINGS DOUBLE NICKLE

THERMISTER #1



TEMP READINGS DOUBLE NICKLE

THERMISTER #2



B. Boundy
J. Dahill

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INTEGRATED GEOHAZARD MANAGEMENT - A SYSTEMWIDE APPROACH

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and
Matthew McCahan, Pennsylvania Turnpike Commission

ABSTRACT

Rock slopes along the Pennsylvania Turnpike have been monitored for over ten years. During this period, the approach to the rockfall hazard management has been refined. Earliest inspections used a specially developed form, while later inspections used a modification of the Oregon rating system. The first inspections attempted to look at all slopes, now slopes are viewed on a prioritized basis. Initially, there was a sense of immediacy to performance of remediation. Now there is more monitoring and analysis prior to scheduled remediation. With growing assurance that rock slope hazards are manageable, it became apparent that other geotechnically-related hazards can also be similarly managed.

The first step in this process was definition and delineation of other geohazards: items of potential geotechnical concern. These hazards include karst problems, mining-related problems, embankment stability, erosion and scour. These areas were identified on database lists and straight-line diagrams. Maintenance histories and construction records in these areas of concern are being collected to complete the initial inventory.

With the inventory assembled, plans for inspection and monitoring are being developed. Preliminary considerations for each concern included how to assess risk, what specifically to inspect and monitor, and the frequency of monitoring. With the collection of periodic new data, it is possible to monitor changes in conditions and evaluate corresponding changes in risk. Priorities can then be established for remediation.

Corrective action can be taken in an integrated, planned manner to maximize the benefits (minimize the hazards) within the external constraints of schedules and budgets. Performance records can be maintained during future monitoring to provide feedback as to the apparent success of various remediation design options.

ACKNOWLEDGMENTS

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EVOLUTION OF ROCK CUT MANAGEMENT ALONG THE TURNPIKE

The first systemwide inspection of rock cuts along the Turnpike occurred in 1986 (Baker, 1987). At the time, the Turnpike consisted of approximately 359 miles of east-west roadway ("Mainline") and roughly 110 miles of north-south roadway ("Northeast Extension") (see Figure 1). Over 400 cuts were identified based on milepost number on both roadways. General characteristics such as cut height and length, slope angle, rock type, drop zone, and drainage were noted on an inspection form. Photographs and videotapes were taken to document conditions. The 1986 inspection used a numerical rating system, assigning values of 1, 2, 3, 4, or 5 to each of the rock cut slopes. The ratings were subjectively based, with 1 representing the "best" slopes and 5 representing the "worst." The following is a summary of the 1986 slope ratings:

1986 Rating	No. of Cuts
5	6
4	49
3	69
2	32
1	261

This first field inspection was performed by a solitary geotechnical professional. Review of the inspection ratings indicated that the ratings were subjectively lower for certain parts of the system. Further, while a certain few cuts obviously needed remediation and many other cuts were performing very satisfactorily, a relatively large number of cuts fell in the number 3 ("uncertain") category.

The product of the study was a multi-volume document with all completed inspection forms, copies of photographs, a computerized listing of rock cuts, and recommendations for additional investigations.

Subsequent to the 1986 evaluation, the "worst" rock cuts were further evaluated with in-depth inspections, and remedial designs were developed. Three of those six cuts received complete reconstruction, two received partial reconstruction, and one received a simple scaling of loose material.

In 1991 (Baker, 1992), Mainline and Northeast Extension rock cuts were reevaluated. Because of the large number of slopes with ratings of 3 and 4 in the earlier inspection and the recent development of the "Rockfall Hazard Rating System" by the Oregon State Highway Division (Oregon, 1990), the inspection procedure and form were re-evaluated. A new field inspection form was developed, modified from the Oregon form (see Figure 2). Rock cuts were field-rated based on cut geometry, geology, and evidence of rockfalls. Maintenance history, traffic and roadway geometry issues also were considered, but not assigned a rating value. The following is a summary of the 1991 slope ratings:

1991 Rating	No. of Cuts
>300	4
200-300	16
100-200	38
<100	200

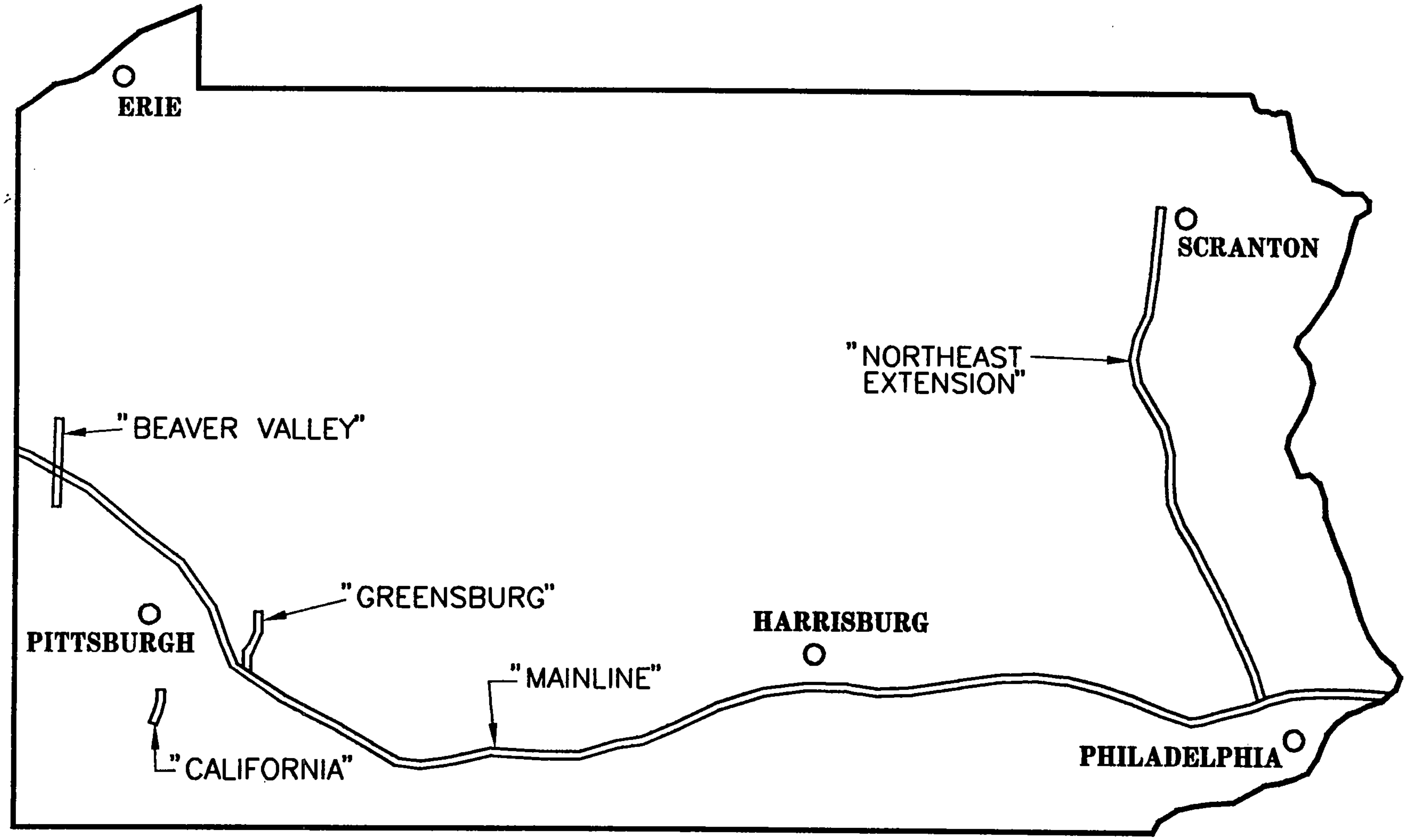

FIGURE 1

PENNSYLVANIA TURNPIKE STATEWIDE SYSTEM

Baker

PENNSYLVANIA TURNPIKE COMMISSION
Rock Cut Rating Form

Cut Location: MP ______________________ Date: ______________________

Geology: Formation, Rock Type, Fracturing, Structure ______________________

Rating	Points 0	Points 3	Points 9	Points 27	Points 81
Slope Angle	1 1/2:1 ☐	1:1 ☐	3/4:1 ☐	1/2:1 ☐	>1/2:1 ☐
Slope Height	0 to10 ft. ☐	10 to 25 ft. ☐	25 to 50 ft. ☐	50 to 100 ft. ☐	>100 ft. ☐
Slope Continuity	No Launch Features ☐	Possible Launch Features ☐	Some Minor Launch Features ☐	Many Launching Features ☐	Major Rock Launching Features ☐
Jointed/Structural Condition	Discontinuous Joints, Favorable Orientation, Dipping into Cut ☐	Discontinuous Joints, Favorable, Random ☐	Discontinuous Joints, Adverse, Toppling ☐	Continuous Joints, Adverse, Length <10 ft. ☐	Continuous Joints, Adverse Length >10 ft. ☐
Rock Joint Friction	Tight, Rough ☐	Rough, Irregular ☐	Undulating, Planer ☐	Clay InFilling, Slickensided ☐	Wet, Open ☐
Differential Erosion	No Differential Erosion Features ☐	Few Differential Erosion Features ☐	Occasional Erosion Features ☐	Many Erosion Features ☐	Major Erosion Features ☐
Difference in Erosion Rates	No Difference ☐	Small Difference (inches) ☐	Moderate Differences Overhang <3 ft. ☐	Large Difference Favorable Overhang >3 ft. Stable ☐	Large Difference Unfavorable Overhang >3 ft. Unstable ☐
Block Size / **Quantity of Rockfall/Event**	No Blocks / No Rockfall/Event ☐	<12 in. / <1 Cubic Yard ☐	1 to 2 ft. / 1 to 3 Cubic Yards ☐	2 to 5 ft. / 3 to 10 Cubic Yards ☐	>5 ft. / >10 Cubic Yards ☐
Rockfall History	No Falls ☐	Few Falls/Talus ☐	Falls/Raveling ☐	Falls/Mobilized ☐	Falls/ on Roadway ☐
Subtotal	____	____	____	____	____

TOTAL POINTS: ______________

COMMENTS: ____

____

____

INSPECTOR: ____

FIGURE 2
FIELD INSPECTION FORM

These field inspections were performed by solitary inspectors. Selected slopes were cross-checked to verify that the ratings were less subjective than the 1986 rating values. The use of a broader ratings scale eliminated the problem of a large "uncertain" midrange.

In addition to completed inspection forms and recommendations for additional study, the products of this study included straight-line diagrams of the system with slope ratings and an expanded geotechnical data base. The data base included not only the results from the two rounds of cut slope inspection, but also other items of geotechnical interest, such as recent geotechnical investigations, structures, and selected features from the annual physical inspection report (an annual systemwide inspection noting deficiencies and proposing corrective actions to restore proper operating conditions).

Since 1991, rock cut slopes have received remediation or treatment as part of other on-going projects. These projects have included bridge widening and rehabilitation work and pavement maintenance work.

In 1995, all rock cuts on Beaver Valley, Greensburg, and California sections of the Turnpike were rated and photographed for the first time. Mainline and Northeast Extension cuts which had a rating greater than 100 during the 1991 inspection were reevaluated. Photographs were obtained if the rating remained above 100. In addition, aprroximately one fifth of the remaining Mainline and Northeast Extension rock cuts were inspected regardless of prior ratings. The following is a summary of the number of slopes in various rating groups:

1995 Rating	No. of Cuts
>300	2
200-300	38
100-200	75
<100	152

The 1995 inspections were completed by two two-person teams. Selected slopes were rated by both teams for quality assurance purposes to verify procedures and results. As a result, a greater degree of comfort was reached in the use of the inspection form.

The straight-line diagrams and the geotechnical data base were expanded to include most recent data. In addition, an inventory file was established for all rock cut slopes that includes the rating forms, available photographs, and other relevant information. It is intended that this inventory file will grow with future inspections and design/construction activities.

THE PRESENT MANAGEMENT APPROACH

The present approach to cut slope management is summarized in the flow chart below (Figure 3). An inventory has been established based on historical data, which is referenced to the Turnpike's milepost locations. This inventory is a tool for assessment of slope performance and identification of areas where some action is required. All rock cut slopes are on a scheduled monitoring program, with maintenance, rehabilitation and remediation options available as necessary.

Annual inspections are to be performed on all cut slopes rated 100 or greater using the standard rating form. Photographs should be obtained every five years or upon change of conditions. Cut slopes rated at less than 100 should be reinspected on a five year cycle. Newly constructed cut slopes or slopes modified by construction or maintenance activities are to be inspected and photographed. Information derived from these inspections is to be added to the inventory.

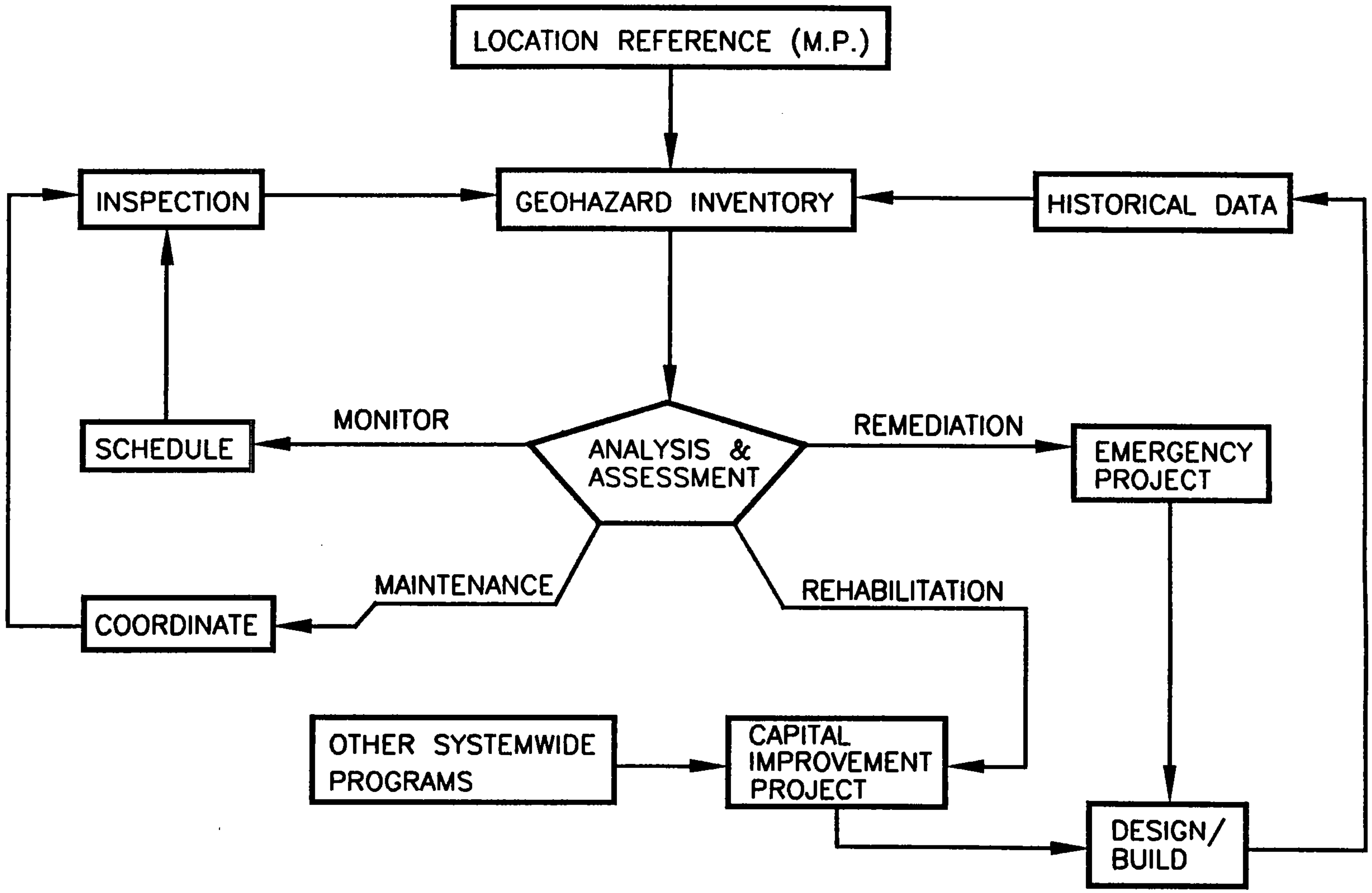

FIGURE 3

MANAGEMENT APPROACH

Maintenance activities related to rock cut slopes should include annual programs (such as cleaning behind rockfall fences and barriers) as well as on-call debris clean-up and installation of hazard signs for active areas. Communications have been established between the geotechnical staff and the maintenance forces and periodic rockfall activity reports are solicited and received. This information is also added to the inventory.

Rehabilitation activities refer to those activities which can typically be incorporated with other planned or scheduled construction, such as repaving or bridge rehabilitation. They apply to slopes which are less than desirable, but do not pose imminent risk to operators on the system. Typically, rehabilitation design activities can incorporate detailed slope evaluations. The evaluations consider flattening the cut slopes to 1-1/2:1 (H:V) and several other alternatives. There frequently is a substantial cost savings for slope work performed as a part of other construction projects. Information collected during the design and construction process is filed with the inventory.

When imminent risk to operators is identified, a slope can require its own remediation design/construction project. Then, a streamlined design is performed based on detailed cut slope reconnaissance, mapping, and analysis. Justification for the higher costs typically associated with a special slope remediation project can be supported by risk assessment. Historical documentation within the inventory provides a basis for this.

APPLICATION OF THE APPROACH TO OTHER GEOHAZARDS

The same approach used for rock cut management can be applied to other geohazards. Three hazards, coal mining, karst features, and embankment failures, illustrate how this approach can lead to more integrated treatment.

Coal Mining

Coal mining has been a hazard since the Turnpike was first constructed, and has continued to be an issue as the Turnpike has expanded. Risks associated with coal mining are primarily related to subsidence, although acid mine drainage is also a potential concern. Initial treatment techniques included undercutting, coal removal and controlled backfill where seams were below roadway grade and simple backfilling where voids were encountered above grade. Economic and environmental factors, along with newer technologies, have led to more involved treatments such as complex grouting programs. Criteria for determining the treatment need have also evolved over the past few decades.

Regional data on coal and coal mining is readily available as published and open-file maps. This serves as the basis for a coal inventory. Additional data on coal mining and treatment have been collected where applicable for both expansion and rehabilitation projects. There are places along the system where different techniques are side-by-side, and where treated facilities are adjacent to untreated facilities. This project-specific coal information can be systematically added to the inventory.

The regional data can also be used to efficiently monitor the risks associated with coal mining. A prioritized inspection schedule can be developed after an initial field inspection verifies the regional data, identifies the portions of the Turnpike impacted by coal mining, and provides a preliminary ranking of the severity of the impacts. At the present time, two forms are envisioned to aid the inspections: one for potential subsidence impacts and one for potential acid mine drainage impacts. Both forms would identify the Turnpike facilities within the mined area; coal thickness, depth and dip; visible surface features including subsidence cracks, openings or entries, and waste/spoil areas; and other relevant information.

A monitored coal hazard inventory would facilitate rational minimization of potential impacts as part of other systemwide programs. It would also allow analysis of the various treatment techniques and needs criteria employed in previous construction. Even just the readily accessible regional information can provide valuable baseline information in the event of an emergency situation requiring subsidence remediation.

Karst Features

Karst features also often make their presence known by catastrophic subsidence events. In the past, karst activity has been addressed by maintenance forces, in rehabilitation projects, and by emergency project repairs.

Available topographic and geologic maps, air photos, and past construction/repair records provide the starting point for a karst inventory. An initial field inspection of potential karst areas, with an appropriate form, will allow a preliminary ranking of the potential hazard of karst subsidence. The form used with these inspections would identify the Turnpike facilities within each specific karst-prone area, visible karst features, local drainage characteristics, and other secondary indicators of karst activity. Project-specific karst information can be added to the inventory. Then, a schedule of future inspections can be developed to monitor karst conditions. Changes in karst conditions can also be documented when maintenance forces identify and remediate sinkholes.

The karst inventory can be used to ensure that karst considerations are incorporated with design and construction projects in areas of identified risk. It can also allow evaluation of alternative remediation techniques.

Embankment Failures

Embankment failures have occurred periodically over the more than fifty years of the Turnpike's existence. Typically, they are repaired by either maintenance forces or workers under an emergency contract. Occasionally embankment failure repairs are able to be incorporated with other planned construction, and sometimes a decision is made to simply monitor movement.

Identifying factors which are commonly associated with embankment failure can allow establishment of an embankment hazard inventory. Initially, the locations of these factors can be determined from available regional data, including especially topographic and geologic maps and air photos. Historical data, especially from past construction or repair records, can be added to the inventory as it becomes available. Representative field inspections can verify the regional data and provide site-specific information. Again, the use of a form will facilitate the inspections. This form will be very similar to the cut slope form; addressing geometry, embankment materials and foundation conditions, visible surface features such as tension cracks, and other items of potential impact to stability like erosion and scour. After an initial sampling of embankment slope inspections, a schedule of periodic inspections can be developed. Maintenance activities repairing slides can be monitored and included with the inventory.

With an embankment hazard inventory, areas requiring rehabilitation can be identified for incorporation with other systemwide construction programs. In emergency response situations, historical data on the effectiveness of remediation techniques on similar and nearby failures will be available to expedite the repair.

Current Status

At the present time, these three hazard inventories are in their infancy. However, their development is easier as a result of the experience gained with the rock cut slope inventory. In all cases, available background and historical data was assembled first. Relatively easy-to-use forms are next to be developed, to collect relevant data which will lead to a broad-based numerical rating. The forms will be field tested, and a monitoring schedule will be developed. With this in place, geotechnical personnel will be better equipped to handle maintenance, rehabilitation, and remediation of the hazards.

FROM GEOHAZARD MANAGEMENT TO GEOTECHNICAL MANAGEMENT

While today the geohazard management approach involves inventory files, straight-line diagrams, and data bases, the information is compatible with future plans to implement GIS. In order to assure this compatibility, three basic criteria were met: the inventories are tied to geographic locations, the inventories are maintainable, and the inventories are unique for each hazard.

The milepost system of location is used for practically all surface features along the Turnpike. Therefore, if another location system, such as GPS, is adopted in the future, a common conversion can be used for all information presently related to mileposts or shown on straight-line diagrams.

Once the inventories are established, they are easy to update. Use of standard, relatively simple forms facilitates field inspections. Establishing a priority sequence for inspections minimizes the number which need to be performed in any year. Data entry is also simple and straight-forward, using standard computer programs.

At first, one all-encompassing geotechnical database was attempted. Then it was realized that many small inventories are preferable from the perspective of data handling. This also allows the creation of databases for other geotechnical items of interest and concern; including preservation of basic geologic data and monitoring the performance of experimental products and techniques.

Ultimately, perhaps an integrated, systemwide approach can be applied to most, if not all, geotechnical activities along the Turnpike.

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SGH FORM: FORMAT FOR EARLY COLLECTION OF ESSENTIAL GEOTECHNICAL DATA

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and
Christopher A. Ruppen, P.G., Michael Baker Jr., Inc.,

ABSTRACT

A geotechnical investigation for a new roadway routinely begins with an office study: the review of available information from reports, publications, maps and other materials at hand. It seems that everyone recognizes the importance of this, as it is considered sound practice to start with what can be easily known about the project site. However, the level of diligence put forth in performance of this relatively simple task varies greatly and often falls far short of a complete investigation.

To encourage a consistent and more complete effort, a form was developed for use during initial office studies. The form was first developed for use on projects performed by the Allegheny County Department of Engineering and Construction, then within District 11-0 of the Pennsylvania Department of Transportation, and recently for general consultant use.

This form allows easy definition of the soil, geologic, and hydrologic setting of a project site, and is commonly referred to as the "SGH form." It directs the user to appropriate agency resources, such as USGS topographic maps, USDA soil surveys, and other published information. It forces a systematic collation of information within, but not limited to, the project boundary. Information on description of soils, structure contours, outcrops, stratigraphy, landslides, rockfalls, mining and mine subsidence, karst, wetlands and hydric soils, flood and hydrologic data and indicators for potential hazardous waste are compiled.

Upon completion of this form, the user has a concise document including maps, a stratigraphic column, and related text which serves as a basis for the following steps in the design process. Completion of this form combined with a geotechnical site reconnaissance is an appropriate milestone to allow proper development of the test boring program.

When this form is not followed, time is lost due to repetitive work, inappropriate field investigations, unconstructible designs, failed designs or unnecessary environmental impacts. These in turn can be traced to oversight of basic soils, geologic, or hydrologic site conditions.

ACKNOWLEDGMENTS

The authors appreciate the support provided by Michael Baker Jr., and the Pennsylvania Department of Transportation in the preparation and presentation of this paper. However, the views and

conclusions contained herein are those of the authors and should not be interpreted as necessarily representing the official policies or recommendations of either Michael Baker Jr., Inc. or the Pennsylvania Department of Transportation.

I. INTRODUCTION

In the late seventies and early eighties, the senior author routinely was involved in reviewing geotechnical reports prepared by others for the Allegheny County Department of Engineering and Construction (ACDE&C) and at the same time performing geologic investigations for projects being designed in-house. Too often these reports were found to be lacking in critical information such as landslide or mine subsidence potential. This was a deficiency that frequently could be traced to an inadequate office investigation. Also, as staff levels shrank and workload increased, it became difficult to insure all critical soils, geologic, and hydrologic data necessary for proper geotechnical investigations were being researched even on the projects being designed in-house. It became apparent that, both to review the completeness of work being performed by others and to adequately perform in-house investigations, a format or system was needed to insure that a complete office study was conducted. It was at this time, to meet the need for a method of insuring a thorough office study of site geologic conditions, that the Soils, Geologic, and Hydrologic Setting Form (SGH) was first developed.

Since most of the work being performed at the ACDE&C were reviews and the staff was fairly experienced, the form consisted mainly of just a brief listings of critical topics (such as, rock type, mine subsidence potential, landslide potential, etc.). However, when the senior author moved to the Pennsylvania Department of Transportation in the late eighties, the level of staff training and experience was less and more work was being done in-house. It quickly became apparent that a more formal format was necessary and the SGH Form much as it is presented later in this paper was created. Along with the SGH Form, a list of references was started to guide the geologist or engineer in completing the Form.

Most recently in the past 5 years or so, private consultants (such as Michael Baker Jr., Inc.) have seen the value in using such a format and are regularly using it not just on projects for the Pennsylvania Department of Transportation but also for private clients.

The SGH Form enables the geologist or engineer to perform the typical office study quickly and thoroughly. The SGH walks the inexperienced geologist or engineer through the various physical features of a site that can be of concern from steepness of slopes to depth to minable coal seams. The associated reference list directs the individuals to the sources of the information. Even the experienced professional, when faced with a heavy workload, may forget to investigate all the necessary resources. The SGH helps to prevent such omissions, thereby encouraging a more consistent effort.

Upon completion of the form, the user has a concise document including maps, a stratigraphic column, and related text which serves as a basis for the next steps in the design process. The role of the SGH in the design process is discussed in the next section.

II. DESIGN PROCESS

The SGH is recommended to be completed as early as practical in the design process. The information collated in an SGH can be invaluable to selecting a corridor or refining an alignment. For larger projects, an SGH should be completed prior to conducting a field reconnaissance to aid in "tuning in" the reconnaissance to specific problems. On small projects, it's recommended to start an SGH, combine it with a geologic or geotechnical field view or reconnaissance, then finalize the SGH. This is recommended to minimize the time required to complete an SGH on smaller budget projects, while also maximizing the information gained by combining reference information and field information.

Once an SGH has been used as an aid to establish an alignment of the transportation project, it becomes a very useful tool and reference for follow-on geotechnical work. Two closely associated design process activities are completion of a detailed geotechnical reconnaissance and implementation of a test boring program. It is recommended to include completion of an SGH as a project milestone prior to completing a reconnaissance or test boring program.

Surface evidence from the geotechnical reconnaissance can aid in confirming or refuting problem areas identified during the SGH process. The test boring program is often considered the means to provide subsurface information to allow for design of cuts, embankments and structures, but it is also the common choice to investigate problem areas such as these commonly identified through the SGH process. A well constructed test boring plan and program is one that considers information gained from the SGH and the geotechnical reconnaissance to maximize the subsurface information obtained. This does not mean that the cost of drilling programs needs to increase to investigate every potential problem, but rather that a more efficient program can be implemented. Generally, this is significantly more cost effective then the often used but somewhat mindless grid patterns.

The SGH is an invaluable tool to aid in the development and mentoring of younger staff. Completion of the form is a good exercise for emerging geologists and geotechnical engineers to learn:

- What references are available
- Where to find and how to use appropriate references
- What soils, geologic, and hydrologic information is important to a project
- How to piece together reference material and field information to gain a full understanding of the project
- How collation of this information early can aid in problem avoidance and solving
- How to optimize follow-on work such as a geotechnical reconnaissance or test boring program
- A better understanding of the design process

This work can be completed under the guidance and mentoring of more seasoned or senior staff to keep the effort focused and build a sense of accomplishment. After completion of the SGH, the form should be utilized by the project team to apply the information to the design and development of the project.

III. DESCRIPTION OF THE FORM

The SGH is concise and formats a regimented and thorough effort towards researching the soils, geology, and hydrology of a project. This section is intended to briefly summarize the technical elements of the SGH.

Figure 1 depicts Page 1 of the form. Page 1 includes biographical and geographical information including name, location, and description of the project. Section 1 utilizes United States Geological Survey Topographic Quadrangles to establish project boundaries and elevation limits of the project. Maps collated for this section become the basis of a project location map which typically becomes an attachment to an SGH.

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SOIL/GEOLOGICAL/HYDROLOGICAL SETTING FORM

Project Name Route 18 Relocation **S.O. #** 22020-000-0000 00003

General Location BEAVER FALLS, BEAVER COUNTY PENNSYLVANIA

Specific Location THE Four Block AREA NEAR GENEVA College bound by Route 18 to the EAST, and Fourth AVENUE To the WEST, From the intersection of 9th Ave & RT. 18 To 4th Ave & Rt 18.

Project Description RELOCATION OF ROUTE 18 TO by pass College Avenue.

State Pennsylvania **County** BEAVER **Municipality(ies)** BEAVER FALLS

Prepared By I.J. ZELESNAK **Date** 11-3-95 **Checked By** J.D. LASKO **Date** 11-7-95

1. Surface Elevation From U.S.G.S. Topographic Map(s) REFER TO FIGURE 1

U.S.G.S. 7.5/15 Minute Quadrangle(s) and Date Revised

BEAVER FALLS, PA, 1957, Photorevised 1990

		EST. SURFACE ELEV.
Project Beginning or Boundary	Intersection of 9th Avenue and Route 18	790.0
Immediate Point(s)	30th Street and Route 18	845.0
	31st Street and Route 18	866.0
Project Ending or Boundary	Intersection of 4th Avenue And Route 18	910.0

Note: Define location and provide elevation

2. Description(s) of Soil(s) REFER TO FIGURE 2

There was one soil series identified with two types of slope ranges within the project area. Which is the Urban Land-Conotton complex, 0 to 8 percent slopes; and the Urban Land-Conotton complex, 8 to 25 percent slopes. The soils of the Conotton series are loamy-skeletal, mixed, mesic typic Hapludalfs. These soils are deep, well drained and somewhat excessively well drained soils on outwash plains, terraces, kames, and eskers formed in glacial outwash plains. The slope ranges from 0 to 50 percent, but the slope is dominantly 8 to 25 percent. The Urban Land in the Conotton

Sht. 1 refer to sht. 8

Figure 1

Section 2 provides for a description of the project soils. United States Department of Agriculture, Soil Conservation Service Soil Surveys are often the source of information for this section. However, as with most sections, the SGH Form is structured to allow for input from multiple sources.

Because of the regional nature of most of the sources, it is not unusual for conflicting information to be researched.

3. Approximate elevation of stratigraphic unit and bedrock inclination from structure contour map. REFER TO FIGURE 3 and 4, 5, and 6 Cont'd on Sheet 8

Structure contour on the base/top of Middle Kittanning coal **(stratigraphic unit)**

	Location	Elevation of Structure Contour	Surface Elevation	Difference in Elevation	Stratigraphic Unit or FORMATION or GROUP
Project Beginning	9th AVE. & RT. 18	978	790	-188	Pottsville Formation
Immediate Point(s)	30th ST. & RT. 18	997	845	-152	Pottsville Formation
	31ST ST. & RT. 18	997	866	-131	Allegheny Group
					(Close to contact)
Project Ending	4th Ave & Route 18	1000	910	-90	ALLEGheny Group.

Approximate bedrock inclination: Southeast **Direction** ∠1°- **°Dip**

4. Approximate elevation of stratigraphic unit from outcrop/contacts map(s).
REFER TO FIGURE 3 and 4 & 5, 6

Outcrop of the Brookville coal **[stratigraphic unit(s)]**
NOTE: The Contact between the Allegheny and Pottsville Group exist within Project Area.

	Location	ESTIMATED Elevation of Outcrop/ Contact	Surface Elevation	Difference in Elevation	Stratigraphic Unit or Formation or Group
Project Beginning	9th AVE & RT. 18	880	790	-90	Mercer COAL Group
Immediate Point(s)	30th ST. & RT. 18	885	845	-40	Homewood sandstone
	31ST ST. & RT. 18	885	866	-19	Homewood sandstone
					(Close to Allegheny Pottsville contact.
Project Ending	4th Ave & Rt. 18	905	910	+5	Clarion sandstone

5. Geologic Definition of Project Area REFER TO FIGURE 7

	Location	Stratigraphic Unit	Formation	Group
Project Beginning	9th AVE & RT. 18	Mercer COAL Group	Pottsville	Pottsville
Immediate Point(s)	30th ST. & RT. 18	Homewood sandstone	Pottsville	Pottsville
	31ST ST. & RT. 18	Homewood sandstone	Pottsville	Pottsville
			(close to contact)	
Project Ending	4th AVE & RT. 18	Clarion sandstone	Clarion	Allegheny

SHT. 2

Figure 2

Figure 2 is Page 2 of the SGH. Page 2 concentrates on the geologic definition of the project including items such as stratigraphy, geologic structure and geologic units, formations and groups which may be intersected by the project. Section 3 utilizes structure contours on marker beds to identify stratigraphic units which may be present near the ground surface at the beginning, intermediate and ending point of the project. This information can be obtained by combining resources from structure contour maps (typically on coal seams and limestones in our area) with generalized geologic sections which may be available for the area. In Western Pennsylvania, coal units such as the Pittsburgh or Freeport are typically utilized for this activity. However, older studies also used such persistent limestone units as the Ames limestone.

Section 4 utilizes outcrop or contact maps to further define the project stratigraphy.

Locally, this may be using a significant marker bed such as the Ames limestone to fine tune the stratigraphic limits of the project.

Section 5 combines Sections 3 and 4 to create a geologic definition of the project. Investigators are encouraged to show the range of possible units when combining the previous information into Section 5. This demonstrates the uncertainty of the information while keeping the investigator's mind open to all possibilities. Utilizing structure contour information, marker beds, outcrop and contact maps and local geologic sections, the anticipated geologic units which may be present at various parts of the project can be identified. Collation of this information also provides a stratigraphic column for the project which is typically an attachment to the SGH.

6. Landslides, Rockfalls and Related Features REFER TO FIGURE 8

NO EVIDENCE OF susceptibility TO LANDsliding or rockfalls WAS found during Literature review. HOWEVER, man-made fills LOCATED TO the south west of the project BEGINNING HAS exhibited signs OF SLOPE movement, AS observed during field reconnaissance.

7. Mining and Mine Subsidence REFER TO FIGURE 9

There are several mineable coals seams which outcrop in the Valley walls above the project area. According to the reference # 5 the project limits are located in an area that correlates with extremely rare damaging mine-- subsidence events. Additional, the project is located in an area where the coals have been removed by erosion or are less than 14 inches thick.

8. Karst Features

There is no evidence of Karst or Karst related features within project area. The rock types anticipated to be encountered beneath the project area are sandstones, shales, and coals which are not conducive to Karst development. However, The Mercer limestones may exist below the project area, which are typically 1 to 4 foot thick intermittent Limestones units.

9. Wetlands and Hydric Soils REFER TO FIGURE 10

According to the reference # 14 there are NOT wetlands within the project area. However, According to reference # 9 the soils identified in the project area have hydric soils components which may exist in swales and depressions.

10. Flood and Hydrologic Data

Beaver Falls annual precipitation is approximately 35 inches, which is the source of a majority of the ground water supply. The Homewood sandstone unit is a significant water bearing unit, but may be high in iron concentration. The Pre-Wisconsin gravels beneath high terraces along rivers are not typically water bearing. Although, generally the valley fill deposits are a source of groundwater. refer to Sht. 9

Sht. 3

Figure 3

Figure 3 depicts Page 3 of the form. Page 3 begins to collate specific information on potential geologic hazards as well as wetland information. Section 6 provides information related to landslides, rock falls, and related features. Depending on the area of study, sources for this information may be U.S. Geological Survey maps on landslides or landslide susceptibility, or local reports which may have been prepared for related projects.

Section 7 researches information related to mining and mine subsidence potential. Although coal is the most common mineral resource in southwestern Pennsylvania, other mineral resources need to be considered. Earlier sections of the SGH, especially Section 3-5 help to identify whether there is a potential for mineral resources and mining. In areas where mining has or is occurring, there typically are numerous references or sources of information related to mining ranging from the actual mining company to state agencies to the Federal Office of Surface Mining.

In Pennsylvania, some agencies will also lend assistance in determining potential for subsidence in connection with subsidence insurance.

In areas where there is a potential for karst development, Section 8 provides for further defining the karst hazard and the likelihood of impact to the roadway. State survey and U.S. Geological Survey references are typically available for karst areas to aid in developing an understanding of the initial magnitude of the hazard.

Section 9 refers to the presence of wetlands and/or hydric soils which may impact the roadway alignment. Depending on the area, wetland maps are commonly available from various local, state or federal agencies and the United States Department of Agriculture, Soil Conservation Service produces a listing of Hydric Soils by state. This Section also represents one of several links between the geologic/geotechnical study and the environmental study of the site. For example, while the identification of a wetland may mean a soft and wet foundation for the designer, it also means a loss of a natural resource that may need to be mitigated.

Section 10 allows for information related to flood and hydrologic data utilizing local and federal studies. Information can be obtained from U.S. Geological Survey Maps of Flood Prone Areas, Flood Insurance Rate Maps, as well as Federal Emergency Management Agency references. On some projects, groundwater studies and precipitation data are also very useful.

11. Summary of Aerial Photography Review REFER TO FIGURES 11, 12, 13, 14, 15

Aerial Photography WAS AVAILABLE for the project Area AS follows: 1955, 1963, 1975 (TWO Flight Lines), and 1988. These AERIAL photographs were reviewed for pertinent geotechnical information and the following is A summary of the findings:

refer to sheet 9

12. Indicators for Potential Hazardous Waste

The project Area is LOCATED within AN URBAN AND INDUSTRIAL AREA (Old Factory site located NEAR southern LIMITS). This environment would indicate A moderate to high potential for encountering hazards waste. An Area off of 29th STREET near the southern Limit of the project WAS observed to have A few rusted OUT 55 gallon drums present, as observed during field reconnaissance.

13. Other refer TO Figures 16 and 17

Reference NO 6, 13, AND 17, indicate that there Are no oil or gas wells in the specific project Area, Although several dry wells & gas wells were indicate near the Northern and Southern Limits. Additionally, A shallow gas reservior is shown near the Southern limits of the project Area. Two Wells on shown in the Geneva College CAMPUS Area near route 18, one dry one GAS WELL; Northeast Area OF project, Three wells were also shown in the Vacanted Factory Area.

14. Remarks/Summary

In summary, The main geotechnical and related concerns for the project Area Are:

1. Potential to encounter buried building foundations And related structures that may be poorly compacted and backfilled. 32 Buildings were demolished And backfilled with project Limits.

Refer to sheet 11

Sht. 4

Figure 4

Page 4 of the SGH is depicted in Figure 4. Page 4 continues with specific information related to aerial photographs and hazardous wastes.

Section 11 summaries the findings of review of any aerial photographs that may be available for the project. Ideally, this may include recent photographs which may have been flown for development of the project mapping and any historic photographs that can be obtained. These can be studied to identify past geologic hazards such as landslides, earlier land uses, past mining activities and structural items such as lineaments and fault traces.

Section 12 is included to summarize potential for hazardous wastes as identified through researching the project. This is not intended to be the main environmental investigative technique for the project, since this work is typically covered under a separate study and by other disciplines. However, if environmental items are identified during researching the Soils, Geology and Hydrology through items such as older aerial photographs, this should be noted on the SGH to create a comprehensive understanding of the project and should be communicated to those performing the hazardous waste investigations. This creates another necessary link between the geologist and the environmental disciplines working on the roadway project.

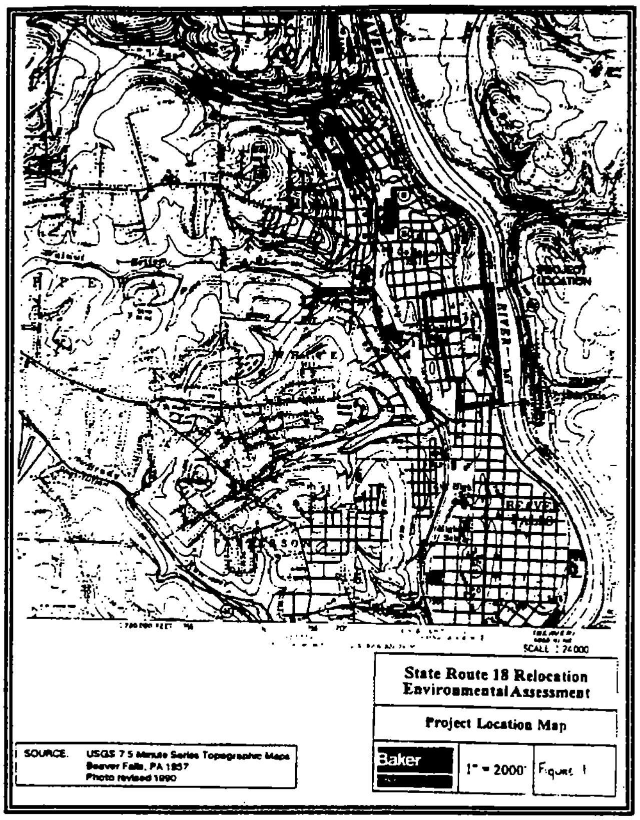
Since this is an expandable form, the remainder of the form is for other items (supplemental information) as deemed necessary for the particular roadway project, references and attachments. All sources of information used for completion of the SGH should be collated and attached as a reference. It is imperative that every source be referenced properly and completely. Attachments to the SGH typically include references, project location map, a geologic or stratigraphic section and copies of any additional maps which were referenced as part of the SGH. These may include, but are not limited to, structure contour maps, outcrop maps, mine maps, karst maps, flood maps, etc. A sampling of various typical figures from this example SGH are depicted as Figures 5, 6, and 7.

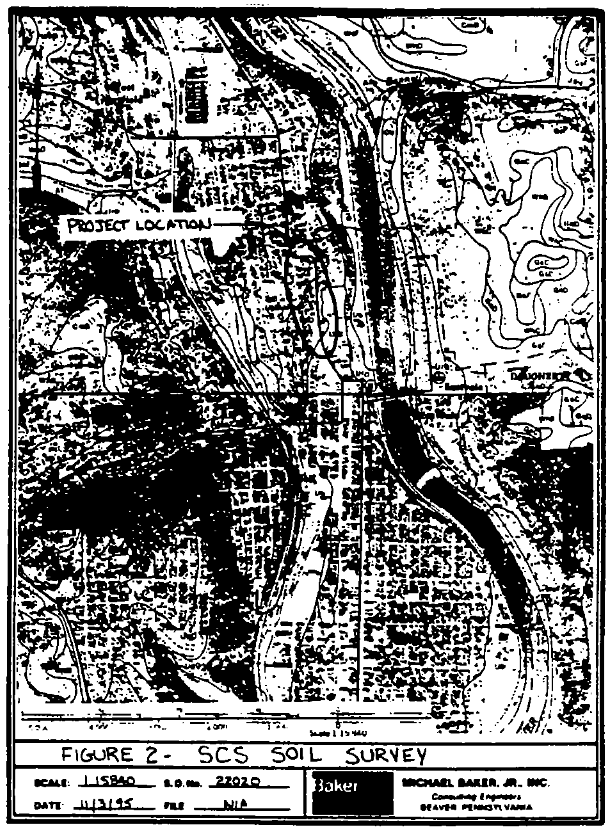
IV. POTENTIAL FORM ADAPTATION

The current SGH had generally been developed for use in the Tri-State area of Pennsylvania, Ohio and West Virginia, although it may be applicable for many parts of the country. Since it is now available as an electronic file, it is easily modifiable to suit the Soils, Geology, Hydrology of other parts of the country. Consideration should be given to customizing the SGH for a particular state or series of states with similar geologic conditions. Potential adaptation may include but not be limited to:

- Volcanics
- Earthquake Prone (Activities) and Faults
- Liquefaction
- Coastal Impacts
- Debris Flows & Avalanches

During development of this paper, the authors recognized the need to add groundwater conditions, oil and gas (wells) and streams/riverbank sections to the form, since all have common impacts on transportation projects in the Tri-State area.


PROJECT LOCATION
SCALE 1:24000
State Route 18 Relocation
Environmental Assessment
Project Location Map
SOURCE: USGS 7.5 Minute Series Topographic Maps
Beaver Falls, PA 1957
Photo revised 1990
Baker
1" = 2000'
Figure 1


PROJECT LOCATION
FIGURE 2 - SCS SOIL SURVEY
SCALE: 1:15840 S.O. No. 22020
DATE: 11/3/95 FILE: N/A
Baker
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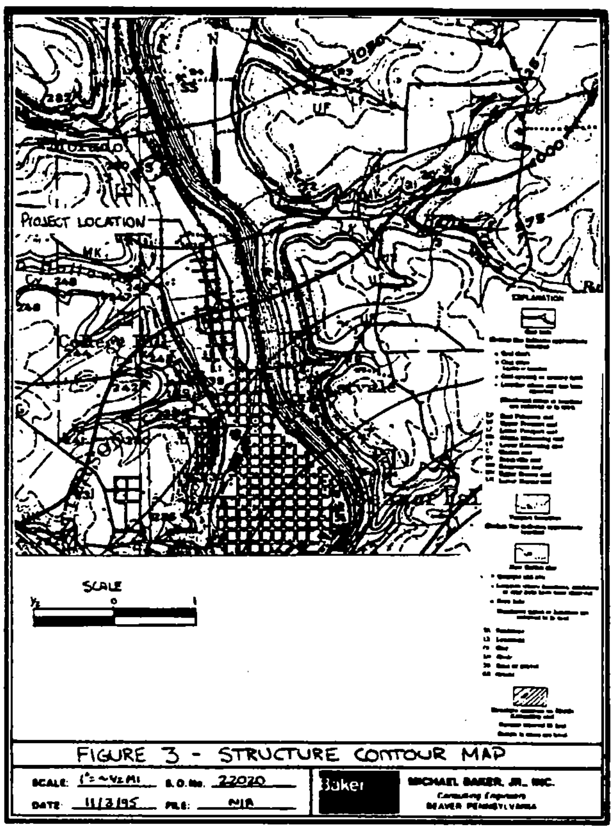
PROJECT LOCATION
EXPLANATION
SCALE
FIGURE 3 - STRUCTURE CONTOUR MAP
SCALE: 1"=~½ MI S.O. No. 22020
DATE: 11/3/95 FILE: N/A
Baker
MICHAEL BAKER, JR., INC.
Consulting Engineers
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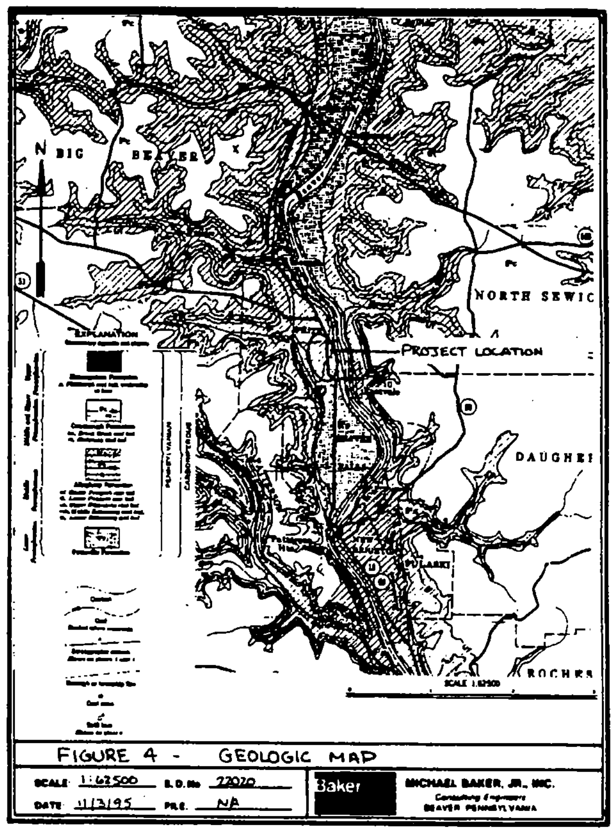
BIG BEAVER
NORTH SEWIC
PROJECT LOCATION
EXPLANATION
DAUGHE
ROCHES
SCALE 1:62500
FIGURE 4 - GEOLOGIC MAP
SCALE: 1:62500 S.O. No. 22020
DATE: 11/3/95 FILE: NA
Baker
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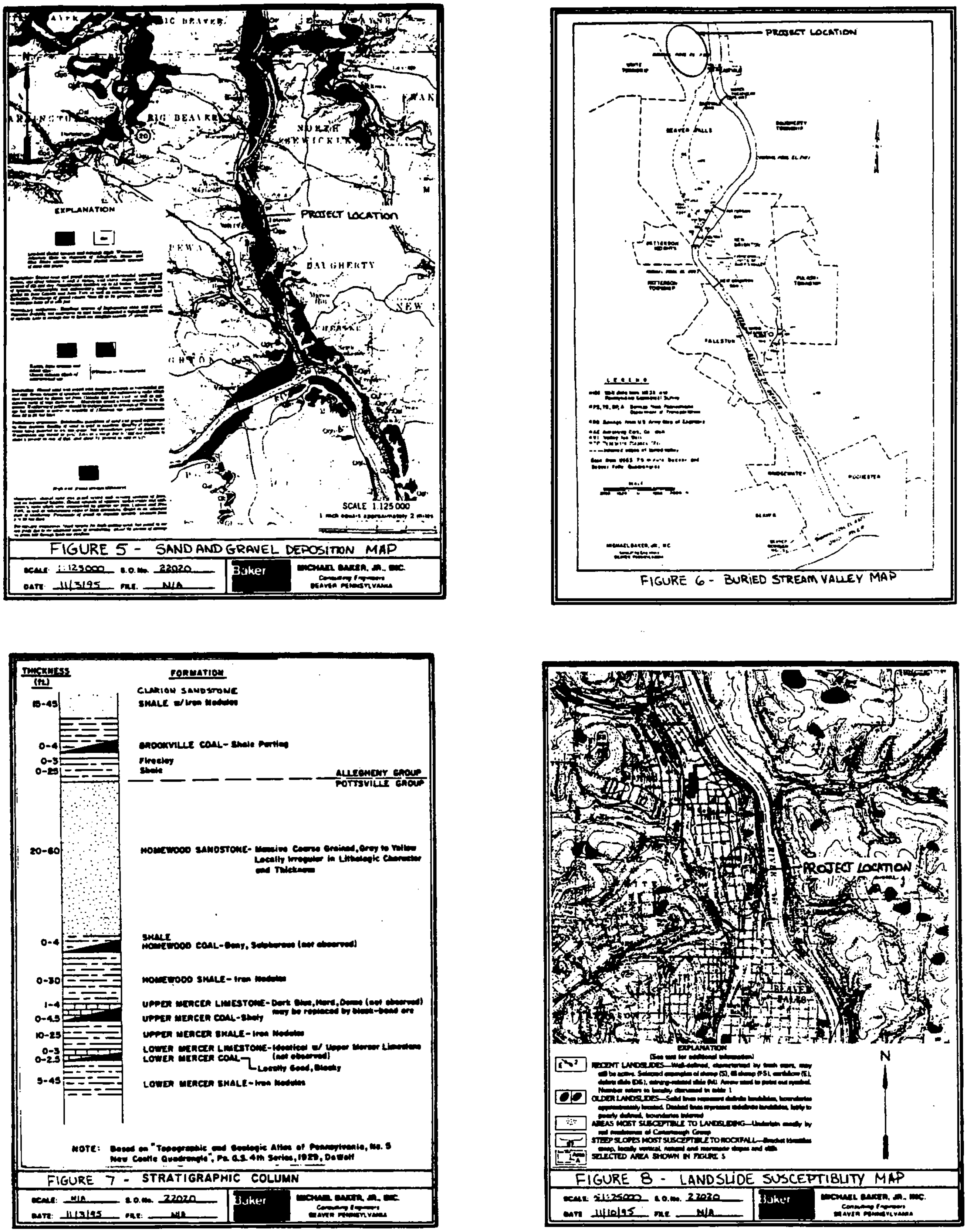
FIGURE 5



EXPLANATION
PROJECT LOCATION
BIG BEAVER
NORTH SEWICKLEY
DAUGHERTY
SCALE 1:125 000
FIGURE 5 - SAND AND GRAVEL DEPOSITION MAP
SCALE 1:125000 S.O. No. 22020
DATE 11/3/95 FILE N/A
Baker
MICHAEL BAKER, JR., INC.
PROJECT LOCATION
LEGEND
FIGURE 6 - BURIED STREAM VALLEY MAP
THICKNESS (ft.)
FORMATION
CLARION SANDSTONE
15-45 SHALE w/ Iron Nodules
0-4 BROOKVILLE COAL - Shale Parting
0-5 Fireclay
0-25 Shale
ALLEGHENY GROUP
POTTSVILLE GROUP
20-60 HOMEWOOD SANDSTONE - Massive Coarse Grained, Grey to Yellow Locally Irregular in Lithologic Character and Thickness
SHALE
0-4 HOMEWOOD COAL - Bony, Sulphurous (not observed)
0-30 HOMEWOOD SHALE - Iron Nodules
1-4 UPPER MERCER LIMESTONE - Dark Blue, Hard, Dense (not observed) may be replaced by black-band ore
0-4.5 UPPER MERCER COAL - Shaly
10-25 UPPER MERCER SHALE - Iron Nodules
0-3 LOWER MERCER LIMESTONE - Identical w/ Upper Mercer Limestone
0-2.5 LOWER MERCER COAL (not observed) Locally Good, Blocky
5-45 LOWER MERCER SHALE - Iron Nodules
NOTE: Based on "Topographic and Geologic Atlas of Pennsylvania, No. 5 New Castle Quadrangle", Pa. G.S. 4th Series, 1929, DeWolf
FIGURE 7 - STRATIGRAPHIC COLUMN
SCALE N/A S.O. No. 22020
DATE 11/3/95 FILE N/A
Baker
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PROJECT LOCATION
BEAVER FALLS
EXPLANATION
RECENT LANDSLIDES
OLDER LANDSLIDES
AREAS MOST SUSCEPTIBLE TO LANDSLIDING
STEEP SLOPES MOST SUSCEPTIBLE TO ROCKFALL
SELECTED AREA SHOWN IN FIGURE 5
N
FIGURE 8 - LANDSLIDE SUSCEPTIBILITY MAP
SCALE 1:25000 S.O. No. 22020
DATE 11/10/95 FILE N/A
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FIGURE 6

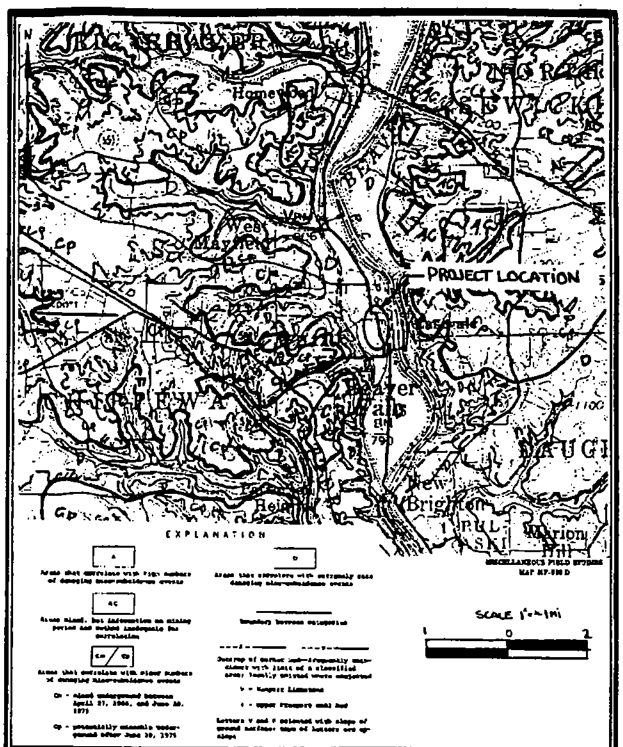

FIGURE 9 - MINE SUBSIDENCE MAP

SCALE: 1" = 1 mile S.O. No. 22020 DATE: 11/3/95 FILE: N/A

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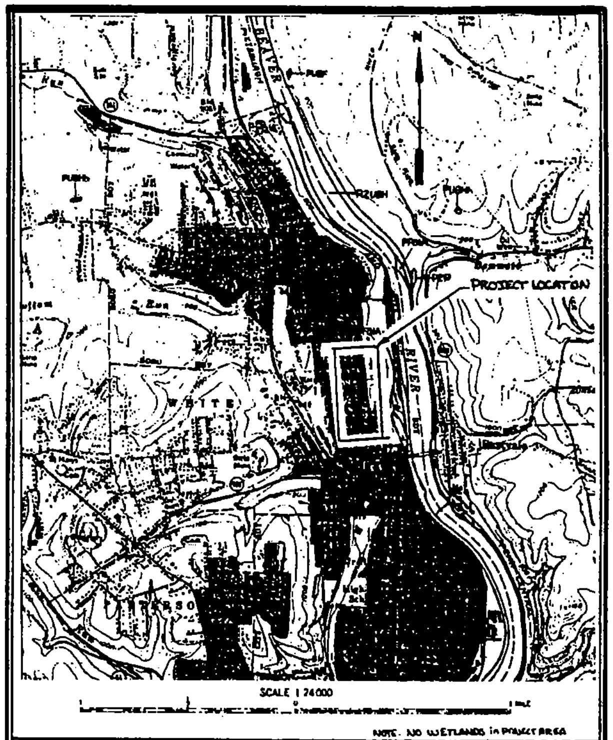

FIGURE 10 - WETLANDS MAP

SCALE: 1"=24000 S.O. No. 22020 DATE: 11/3/95 FILE: NA

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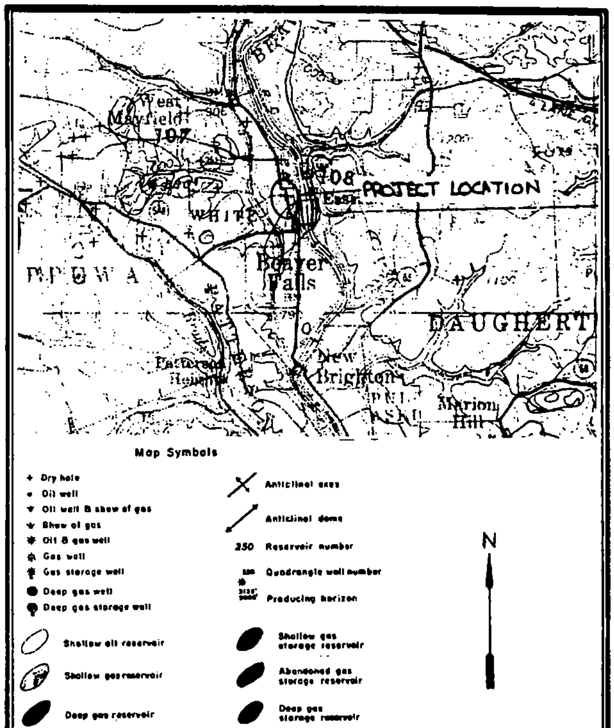

FIGURE 16 - OIL AND GAS RESERVOIR MAP

SCALE: ~1:62500 S.O. No. 22020 DATE: 11/10/95 FILE: N/A

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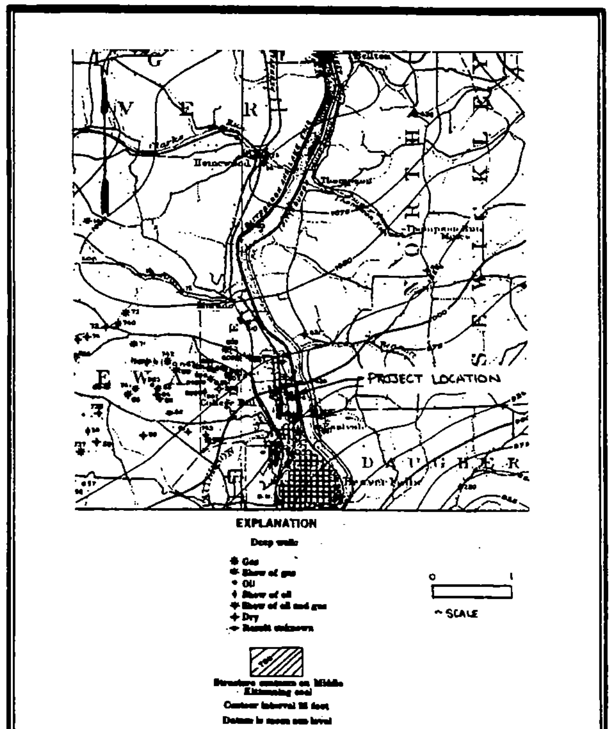

FIGURE 17 - GAS AND OIL WELL LOCATION MAP

SCALE: ~1" = 1 mile S.O. No. 22020 DATE: 11/3/95 FILE: NA

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FIGURE 7

V. FINDINGS OF THE SGH

Collection of data to complete the SGH is only one aspect of the process. If an SGH is completed only to satisfy a scope requirement or a project milestone, then its true value would have been lost. The real value of an SGH is interpreting and evaluating the findings to determine what it means to the design and development of the roadway. Depending on the stage of the project at which the SGH is developed, identification of soils, geologic or hydrologic problems or hazards aids in selection of the corridor, allows for an alignment to be refined, or an appropriate remedial design to be developed to account for the hazard without moving the roadway. In any event, identifying the problem early lets these types of decisions be made. It also allows for cost estimates to be developed which more appropriately represent the roadway which will be constructed.

On Pages 1 and 3 of the form, soils information helps to identify potential problems related to settlement and possibly even stability. Identification of thick compressible (organic) soils would be a red flag for settlement and identification of thick colluvial soils (especially derived from claystones) on slopes would raise a red flag for stability. Identification of wetlands or hydric soils may be the first indication of impact to this environmental resource.

Information from Section 6 on Page 3 should provide an understanding of the likelihood for landsliding or rockfalls along the new roadway. Interpretation from the distribution of known landslides and rockfalls may help target what could be expected for new roadway cut slopes and may even help make some preliminary determinations of proposed new slope angles.

Section 7 on Page 3 of the form should aid in a determination of the extent of mining, surface mining versus deep mining, and room and pillar versus longwall. It may also help with identification of related hazards or problems such as subsidence, mine fires, thick accumulations of mine spoil, or acid mine drainage. Research in this section may be the first information collected regarding the impact to coal reserves. All of these items can have major impacts on locating and designing a roadway.

Identifying the presence of a lineament of sinkholes, as noted in Section 8 on Page 3, would not only be very important to the long term performance of the roadway but, also to the proper positioning of the structures. Often, identification of karst features does not occur until the bridge substructure locations are established. At that point, there may be reluctancy to move the bridge or particular substructure unit and more of a push to design for the problem. This adds project costs and often schedule delays that were probably avoidable by good early detection of the karst problem.

Early identification of environmental impacts as noted in Section 9 on page 3 and Section 12 on Page 4 can be very important to a project. This can be either environmental impacts related to designated wetlands and similar wet areas or environmental impacts related to hazardous wastes. Avoidance of these areas during corridor or alignment studies is typically easier, cheaper and more expedient than trying to design through them later. Both of these areas are typically covered under other work tasks by different disciplines; however, it is important that this information is communicated among the geologist or geotechnical engineer and the other disciplines.

Not all of these problems or hazards impact every roadway job, but it is very common to have one or more significant soils, geologic or hydrologic factors influencing either the positioning of the roadway during the corridor or alignment study phases of the project or the design after the alignment is fixed. Not enough can be said about the critical role in detection of these items early in the design process. Not identifying early that a mine fire is under a roadway or in a proposed highway cut, a sinkhole is positioned under a bridge pier, a thick compressible soil layer is present under a large highway embankment, a thick colluvial soil mass is located in a cut section with limited right-of-way, can make the difference between a bad project that ends up over budget and behind schedule and a roadway or highway that all parties involved feel good about. The SGH Form is a simple format, which when properly applied, will improve the process which should yield better projects.

VI. FORM REFERENCES

Most agencies and consultants have collated a wealth of soils, geologic, and hydrologic information for the particular geographic areas in which we work. The authors highly recommend that a Geotechnical Site Background Resource Guide be established to create a guide for where to find various resources to complete an SGH. This can greatly streamline the SGH process to maximize the benefit yet minimize the effort, meaning keeping the costs down! This guide can be as simple as a folder or binder. The authors' guide is in a three ring binder and has a Table of Contents as follows:

TABLE OF CONTENTS

1. TOPOGRAPHY
2. SOILS
3. GEOLOGY
   - STRATIGRAPHY
   - STRUCTURE
4. LANDSLIDES/ROCKFALLS
5. MINING/MINE SUBSIDENCE
6. KARST
7. WETLANDS
8. FLOODS
9. AERIAL PHOTOGRAPHS

APPENDIX

A. COMMONLY USED REFERENCES
B. WHERE TO FIND IN-HOUSE REFERENCES

This is not meant to complicate the process rather to simplify by identifying what resources or references are really useful and where can they be found. The authors have opted for the three ring binder but this collation could be as elaborate as a CADD or GIS model to systematically organize the data.

Once the in-house sources of references have been organized, an AEG publication (C.H. Trautmann and F.H. Kulhawy, 1983) should be reviewed to discover additional opportunities for resource information. This paper presents a description of the most widely used sources of geologic and related data for use primarily by engineering geologists in the United States. Addresses and telephone numbers are provided for many of the resources.

VII. SUMMARY AND CONCLUSIONS

As with any good geotechnical investigation, the geotechnical investigations for transportation facilities should routinely start with a thorough office study. The SGH Form and its associated Geotechnical Site Background Resource Guide as described above provide a way to obtain such a study and to provide a consistency otherwise frequently lacking.

The SGH Form provides guidance to the less experienced geologist or engineer. It takes the young professional systematically through all the soil, geologic, and hydrologic elements of a site and, when appropriate, leads them to help identify potential problems for the design, construction, and maintenance of a transportation facility.

It is not uncommon when using different references to identify conflicting information about a particular aspect of the project. This may be as simple as finding two different stratigraphic columns which may identify different intervals of the geologic section. This often occurs because of the age of the references as well as the wide variety of sources from U.S. Geological Survey Publications to past design reports from adjacent projects. In any event, when conflicting information is identified, it should be documented to inhibit jumping to specific conclusions about the soils, geologic and hydrologic conditions of a project. This aids the investigator in keeping an open mind for follow-on aspects of the design process.

Typically, engineering studies tend to focus too narrowly on the project site boundaries. The format of the SGH allows for incorporation of information beyond the project's boundaries which frequently plays a significant role in evaluating a site. It allows the investigator to see the big picture. While being of use when designing any site, this aspect also is useful when the inevitable design changes cause realignment of the facility and additional areas of impact result.

In today's environment of "do more with less" the use of the SGH Form offers many areas of cost savings including a much needed link between the environmental studies and the geotechnical studies. The traditional environmental studies dealing with wetland impacts, resource reserves, etc. and the more specialized environmental studies dealing with residual and hazardous waste both require input by a geologist and much of the same information on soils, ground water, etc. to be researched. The Form has been developed to accommodate such overlap and to enable one professional or one group of professionals to supply certain information to all specialized disciplines.

As with any office study, the use of the SGH Form is just one piece that is used in putting together the site puzzle. However, with the SGH Form, potential problems related to the project soils, geology and hydrology can more easily appear as red flags during the investigation. After viewing the site of some of the design/construction failures in our region, it is quite easy to believe that many of them would not have occurred if the project geologist or geotechnical engineer had used the SGH Form and properly identified the site conditions.

REFERENCES

Trautmann, C.H. and Kulhawy, 1983. Data Sources for Engineering Geologic Studies. Bulletin of the Association of Engineering Geologists, Vol. XX, No. 4, pp. 439-454.

Permanent and Temporary Earth Anchoring For Highway Applications

By Kevin Heinert
7/26/96

Abstract

This paper addresses various earth anchoring systems that are used for solving highway related geological problems. A general overview is given exploring: applications, earth anchor types, design and installation. The anchor types presented here are mechanical and bonded systems utilizing steel bars. Also included, is a discussion on the basic differences between these systems, and guidance for selecting the best anchor for a specific application. Factors that influence anchor selection are: soil and rock classification, corrosion protection, working load, anchor permanency and economics. Rock and soil anchors are used extensively for stabilizing slopes that present a hazard to the highways or structures beneath them. Other applications are soil nailing, foundation tie-downs, tie-backs for retaining structures and stabilization of underground excavations. Anchor installation procedures and equipment are discussed for both mechanical and bonded anchorage systems. When Choosing the proper anchor bolt for a specific project, there are several categories that must be investigated. All of them interact importantly together. The designer must consider many factors in specifying the most appropriate anchor. Numerous anchoring systems are available because no single system is best for all situations.

Anchor Applications

Prestressed earth anchor systems have been used for a variety of applications since the 1930's. Over the years engineers, scholars, contractors and manufacturers have worked together to improve the quality and to broaden the scope of earth anchor technology. Organizations such as PTI, ADSC, FIP and AASHTO help provide today's designers with the latest in earth anchorage recommendations and expertise. Today, earth retainment utilizing anchorage systems is more common then ever, and as designers become more aware of the systems available and their applications, earth anchoring will become even more popular. Some of the most common applications for earth anchorages are:

1. Slope stability
2. Tieback walls
3. Soil nail walls
4. Tie downs
5. Tunnel bolting

Slope Stability-Earth anchorages used for slope stability have often been seen where falling rocks or land slides jeopardize the safety of people or the structures above and/or below the slope. Engineers are constantly faced with situations where a slope failure would damage a bridge below or a highway above. If an earth wedge failure or slope failure is anticipated, earth anchors are used to anchor the wedge past the predicted failure plane. If the slope yields potential rock fall hazards, earth anchors often are used to reinforce the rock face and sometimes are used in conjunction with wire, mesh-netting to control falling debris. (Fig. 1)

Tie Back Walls- Tie backs are used to anchor earth past a predicted failure plane, hence allowing the wall to resist lateral earth pressure that would otherwise cause failure. They are often used when a project requires a cut in a slope to accommodate a new highway or bridge abutment. The earth is tied back with the anchors and a wall system, with the construction starting from the top moving down. The Tie Backs, once installed, are pre-stressed to a desired test load and then "locked-off". Pre-stressing is typically done with the use of a center-hole test jack or can be accomplished by using Torque-tension methods with certain thread forms (Fig. 2)

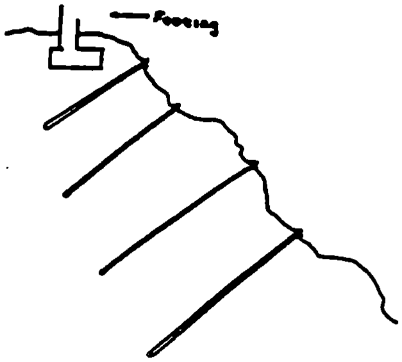
Soil Nail Walls- Soil nailing is a technique used to strengthen or reinforce a particular soil mass. The concept behind soil nailing is that soil (poor in tension) can be reinforced by closely spacing grouted steel tendons within the soil mass. These nails would be a passive system (not pre-stressed) and will be put into tension by the natural lateral forces that the soil mass will exert as a result of support loss due to continued excavation. Like Tiebacks soil nailing is a "Top down" construction procedure. They are commonly used for slope retention for roadway widening and for bridge abutment work. (Fig. 3)

Tie downs- Tie down anchorages are often used to anchor footings allowing them to resist overturning or uplift forces. Footings for retaining walls, piers and bridge abutments commonly are designed with rock and soil anchor systems. (Fig. 4)

Roof Bolting- Roof bolting is a means of strengthening and preserving the natural stresses which exist in rock strata and of guarding against failure due to undermining, tunneling, or other excavation. Rock failures are prevented through the tensile loading of a pattern of bolts, thereby placing the strata in compression, and preventing the development of tension in weak rock strata. The result is a reinforced, "laminated" beam, that will resist sagging, cracking, falling and/or lateral movement among it's components. (Fig 5.)

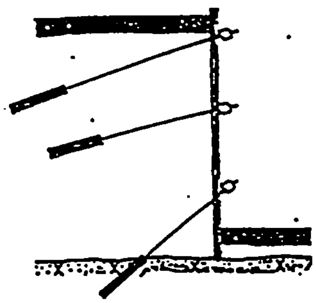
Factors Affecting Anchor Selection and Design-

Several factors influence an engineer's selection of an earth anchor system. Before an engineer specifies an anchor, one of the first questions that the engineer has to answer is if earth anchors are the most effective and economical solution to the problem. There are many cases when anchoring isn't the most economical or practical solution



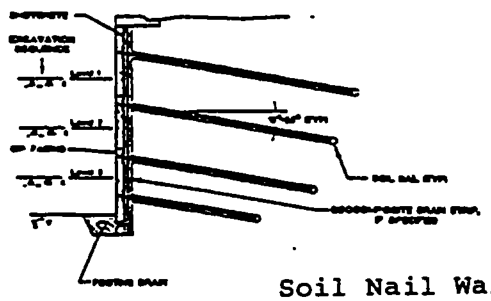
Slope Stabilization

(Fig 1.)



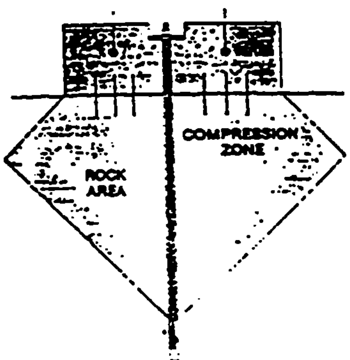
Tie Back Wall

(Fig 2.)

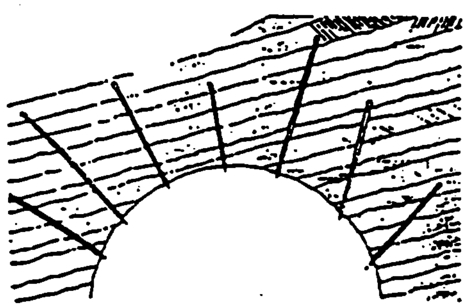


Soil Nail Wall

(Fig 3.)



(Fig 4.)
Tie Down Anchor



(Fig 5.)
Roof Bolting

for retaining earth or resisting uplift. However, in the many cases where they are the best solution, the engineer is then faced with the task of selecting and designing the best anchor for the job. It is important that the designer/engineer investigates the following categories:

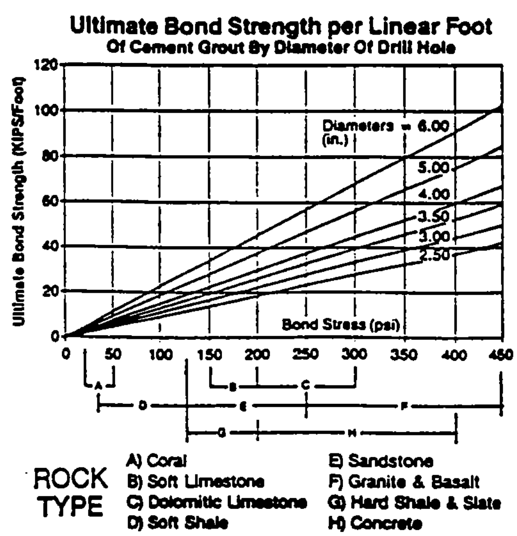
1. Soil or rock exploration (core samples & testing)
2. Corrosion protection
3. Working load
4. Permanency
5. Economics

<u>**Soil or rock exploration-**</u> Prior to design, a geologic study should be performed by a competent specialist to determine anchor suitability.

Since the anchor selection and design is primarily a function of the material that is being anchored into, the more information known about the geology of the site the easier the selection will be for the engineer. A thorough site investigation should include the following steps:

(a) Field survey and site reconnaissance; This should be conducted to achieve an overall understanding of the job site relating to site topography, geology, groundwater conditions, and history. These are all important factors when determining the orientation, practicality and limitations of the anchor design.

(b) A sufficient geotechnical field investigation and test program; The anchor capacity, length, diameter and level of corrosion protection are all factors primarily determined by soil or rock properties. When anchoring into rock or soil with a bonded anchor, it is important to know what kind of load can be transferred, per foot of drill length in relation to the drill hole diameter. The following are soil characteristics that need to be investigated: (1) unit weight, (2) angle of internal friction, (3) cohesion, (4) particle size, (5) liquid and plastic limits, and (6) unconfined compressive strength. Once these properties are known and studied they allow the engineer to estimate the anchor geometry based on the capabilities of the soil. When anchoring into rock the following properties are relevant to design (1) modulus of elasticity; (2) uniaxial compressive strength; (3) rock seam locations; (4) RQD index; and (5) water presence. These rock properties will help in the assessment of the rock mass stability and bond stress values of the material. The following graph (fig 6.) demonstrates how the bond stress value of rock is related to anchor length. Information such as this is available from PTI (Post Tension Institute), and also from documents by AASHTO which relate soil classifications to estimated bond capacity per unit of anchor length.



(Fig 6.)

(c) Investigations during construction. Because soils and rock can vary tremendously from one sample site to the next, unexpected

geological conditions are frequently encountered during anchor installation. This is why it is important that data be collected during anchor hole drilling for possible design changes. Test anchors should be required if initial site investigations reveal that ground conditions may be unpredictable.

__Corrosion Protection-__ Ground anchors placed into an environment classified as "aggressive" need to be protected against corrosion of the steel tendon. Virtually all ground anchors used today have some degree of corrosion protection, unless they are being used in a temporary application or the environment has been declared " non-aggressive". The aggressivity of the ground is based on several characteristics, some being:

1. ph level
2. Chloride content
3. Sulfate content
4. Resistivity
5. Water seepage

Soils with resistivity levels greater than 5000 ohms/cm are generally considered mildly corrosive or noncorrosive. When the resistivity levels are less then 2000 ohms/cm the soil is generally considered corrosive or very corrosive. The chemistry of the groundwater present is very important when classifying a soils level of aggressivity. Some of the parameters are listed below in table 1.

	Aggressivity		
Test	Weak	Strong	Very Strong
1. pH value	6.5–5.5	5.5–4.5	<4.5
2. Lime-dissolving carbonic acid (CO_2) in mg/liter determined by Heyer's marble test	15–30	30–60	>60
3. Ammonium (NH_4^+) in mg/liter	15–30	30–60	>60
4. Magnesium (Mg^{2+}) in mg/liter	100–300	300–1500	>1500
5. Sulfate (SO_4^{2-}) in mg/liter	200–600	600–3000	>3000

(Table 1.)

Several options are available to the engineer when designing anchors for corrosion resistance. Anchors can be designed with simple corrosion protection to the more complex MCP systems ((M)ultiple (C)orrosion (P)rotection). Each system has it's function.

Simple Corrosion protection: Cement grout filling the annulus of the hole around the bar (ASTM C-845-76)

Coatings: Hot dip galvanizing ASTM -A153
Epoxy coating ASTM-A775-85
Grease ASTM B-117 and D-1743

Coverings: Poly Corrugated Tubing ASTM-3350
Smooth PVC tubing SCH40, ASTM D-1785

Working Load- Once an engineer has determined how much lateral earth pressure needs to be resisted, or how much uplift on a structure needs to be resisted, he or she can then begin to look at anchor quantities and working loads. Several steels can be used for anchoring, but the most common for prestressed (bar) earth anchorages are all-thread grade 150 ksi steel (ASTM 722) and grade 75 all thread bar (ASTM 615). Shown below are tables 2 and 3 that show the structural properties and bar diameters for each.

Grade 150 ASTM 722

NOMINAL BAR DIAMETER	ULTIMATE STRESS (ƒ pu-ksi)	*CROSS SECTION AREA A (in^2 - mm^2)	ULTIMATE STRENGTH (ƒpu A) (kips / kN)	PRE-STRESSING FORCE - kips / kN			NOMINAL WEIGHT (lbs./ft. - Kg/M)	APPROX. THREAD MAJOR DIA.
				0.80ƒ pu A	0.70ƒ pu A	0.60ƒ pu A		
1" (26mm)	150	0.85 / (549)	127.5 / 567.1	102.0 / 453.6	89.3 / 397.2	76.5 / 340.3	3.09 / 4.6	1-1/8"
1-1/4" (32mm)	150	1.25 / (807)	187.5 / 834	150.0 / 667.2	131.3 / 584.0	112.5 / 500.4	4.51 / 6.71	1-7/16"
1-3/8" (36mm)	150	1.58 / (1019)	237.0 / 1054.2	189.6 / 843.4	165.9 / 737.9	142.2 / 632.5	5.71 / 8.50	1-9/16"
1-3/4" (45mm)	150	2.66 / (1716)	400.0 / 1779.2	320.0 / 1423.4	280.0 / 1245.4	240.0 / 1067.5	9.06 / 13.48	2"
2-1/2" (64mm)	150	5.19 / (3350)	778.0 / 3457.0	622.4 / 2765.8	544.6 / 2422.4	466.8 / 2074.3	18.2 / 27.1	2-3/4"

*Effective cross sectional areas shown are as required by ASTM A 722-88. Actual areas may exceed these values.

Note: Williams All-Thread-Bar may be stressed to the allowable limits of AC: 318-77. The maximum jacking stress (temporary) may not exceed 0.80 fpu and the transfer stress (lockoff) may no exceed 0.70 fpu.

(Table 2.)

Grade 75 ASTM 615

R61 GRADE 75 REBAR ASTM A615			
REBAR DESIGNATION	MAX. O.D.	YIELD STRENGTH	MINIMUM ULTIMATE STRENGTH
#6	7/8"	33,000 Lbs./146.8 kN	44,000 Lbs./195.7 kN
#7	1"	45,000 Lbs /200.2 kN	60,000 Lbs./266.9 kN
#8	1-1/8"	59,300 Lbs./263.8 kN	79,000 Lbs./351.4 kN
#10	1-3/8"	95,300 Lbs./423.9kN	127,000 lbs./ 564.9kN
#11	1-1/2"	117,000 Lbs./520.5 kN	156,000 Lbs./693.9 kN
#14	1-7/8"	168,700 Lbs./750 kN	225,000 Lbs./1000.8 kN
#18	2-7/16"	300,000 Lbs./1334 kN	400,000 Lbs./1779.2 kN

(Table 3.)

Permanency- Temporary anchors are any prestressed or passive rock or soil anchor with generally less then a 24 month service life. Anchors with longer service lives are usually considered Permanent. A permanent anchor has to fulfill it's function for an extended period of time, sometimes throughout the service life of the structure, and thus requires special design and supervision. Two characteristics that often differ between temporary and permanent anchors are the level of corrosion protection and whether the anchor is pre-stressed. In most cases corrosion protection is not as crucial with a temporary system as it is with a permanent system. Several permanent anchors will require 2 or 3 levels of corrosion protection to reach their design life. This is not to say that temporary anchors do not require high levels of corrosion protection, however, generally they do not.

Most permanent anchors are pre-stressed where as most temporary anchors often serve as rock or soil dowels. When an anchor is said to be pre-stressed, it simply means that the anchor was put into tension during the installation by the use of a tension jack or torque wrench. For example, if a grout bond anchor is being installed, after the grout reaches an adequate compressive strength, the installer will connect a jack to the threaded end of the anchor. The jack will directly place a specific load on the anchor, stretching the steel and thereby lifting the nut slightly off the anchor plate. By tightening the nut back down against the plate and then releasing the load from the jack, a transfer of load from the jack to the anchor has been accomplished. Once the anchor is put into service the anchor will only be subjected to additional steel elongation if the pre-stressing load is surpassed. Therefore, by pre-stressing the anchor, the anchor load has been "locked off" and secured. Some of the advantages of pre-stressing are:

1. Pre-tested: Assurance that each bolt will hold the design load prior to final construction.

2. Full bolt elongation: By pre-stressing the anchor, the steel elongation is locked off prior to applying the working load. Therefore, no additional steel elongation occurs during the anchor's service life.

3. Negligible stress relief: Grouting the bolt after locking in the tensile load allows the grout to harden around the deformations of the bolt and/or bond to the bolt to prevent stress relief in the bolt.

4. Eliminate uplift: Prestressing can eliminate a "floating" condition of a foundation due to uplift forces and overturning moments produced by hydraulic pressures, wind, and live loads.

5. Minimize fatigue failure: Since an external tensile load must exceed the prestressed load of the bolt to cause additional elongation, the periodic stretching and relaxing that causes fatigue failure (often in dowels) is eliminated.

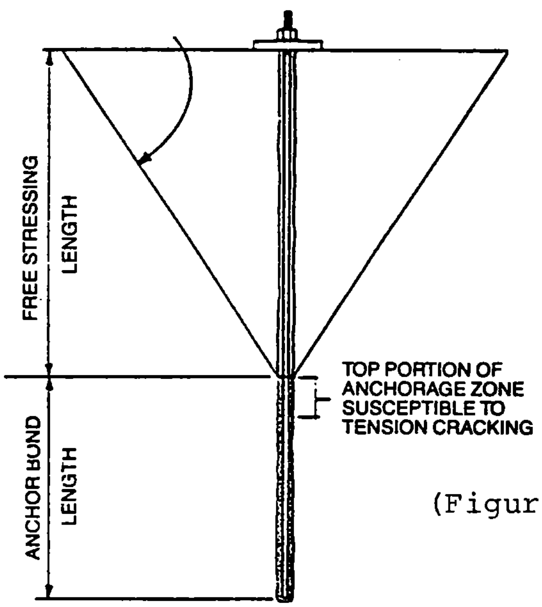
6. Corrosion Protection: A prestressed anchor bolt will not elongate through the grout when resisting loads within the pre-stress range. A non stressed anchor (dowel) will elongate during loading which breaks down and cracks the grout, opening the door to air and water exposure, at that point corrosion begins and can cause eventual failure of the anchor.

Earth Anchor Types

Numerous anchoring systems are available because no single system works in all situations. The anchor types presented here all utilize steel bar. Today, the three most common anchors used in earth anchoring are:

1. Cement grout bonded anchors
2. Resin grout bonded anchors
3. Mechanical anchors

Cement grout bonded anchors- Anchors that rely on the bond between the rock or soil interface and the cement grout that surrounds the annulus of the anchor rod, are classified as cement grout bonded anchors. The anchor length is divided into 2 major sections. The first section is the free-stressing length which runs down the length of the anchor starting from the plate bearing surface. This length is recommended to be a minimum of 10 feet by PTI (Post Tension Institute) for pre-stressed rock and soil anchorages. The second section is called the Bond length which is located from the end of the free-stress length to the bottom of the anchor. This length is dependent on the geotechnical characteristics of the anchoring regime and the desired working load (Fig 7.). The load is transferred from the anchor to the earth by the bond generated along this bond length. A typical Cement grout bonded earth anchor may be 25 feet long and contain a PVC or polyethylene sleeve that is placed around the anchors free-stressing length. The anchor will then be placed in the hole and centered by the use of centralizers (fig 7a). Centralizers should be used to ensure good grout cover (approx. 25 mm). Centralizers are often placed every ten feet on center starting five feet from the back of the anchor. Once the anchor is placed in the drill hole, grout is pumped into the hole until the anchor is encapsulated or in some situations the hole is grouted and then the anchor is immediately installed. After several days the grout hardens and the pre-stressing of the anchor can be accomplished.

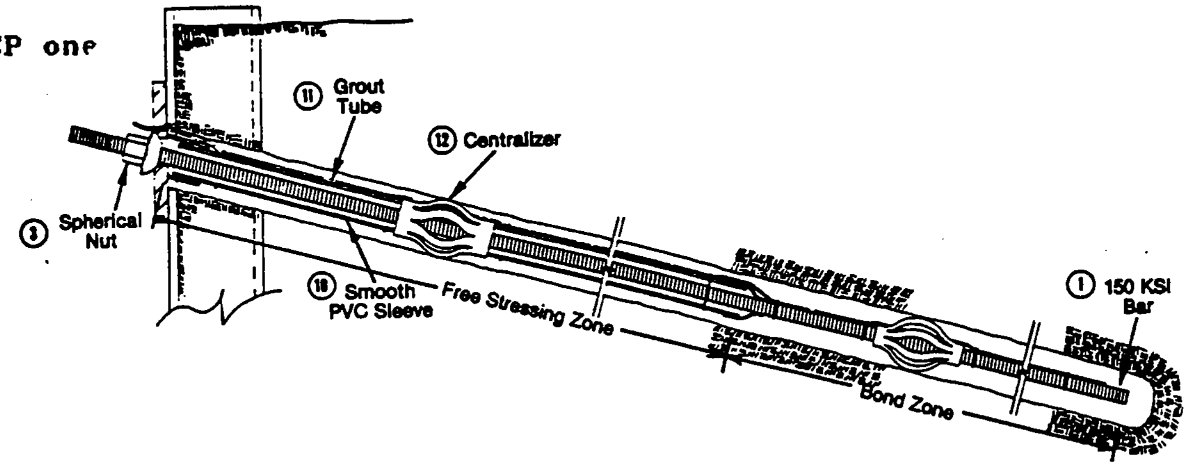


(Figure 7.)

(Figure 7a.)

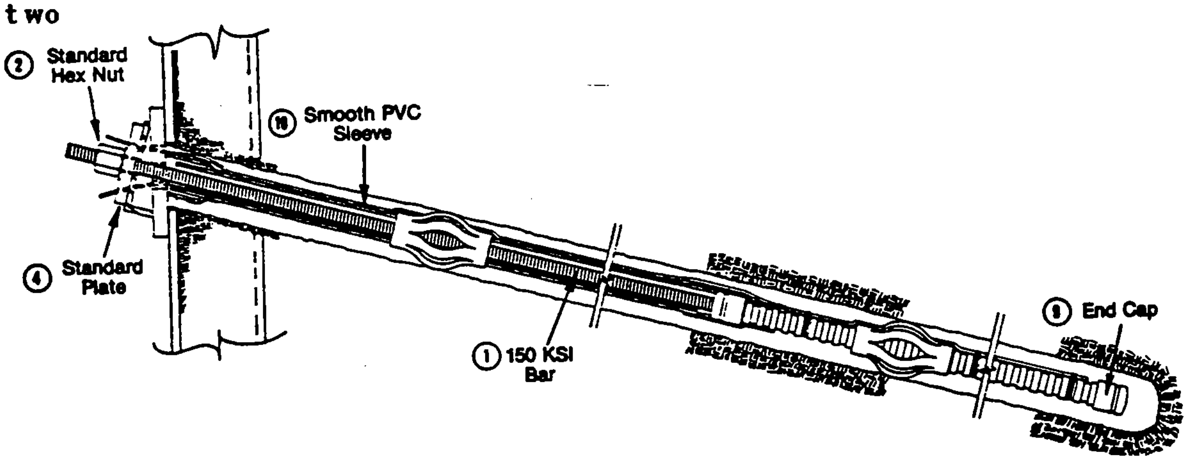
The PVC or Polyethylene tubing along the free-stressing length serves as a bond breaker between the steel and the grout. This allows the steel to elongate during pre-stressing and transfer the load directly to the bond length of the anchor. Pre-stressed cement grout bonded anchors usually are designed with all-thread (ASTM 722) or (ASTM 615) steels. Grout bonded systems generally are classified by their degree of corrosion protection. The degree of corrosion protection should be matched against the aggressivity of the environment and the expected life of the anchorage systems. Multiple Corrosion Protection (MCP) systems offer increasing barriers against corrosion attack.

MCP one



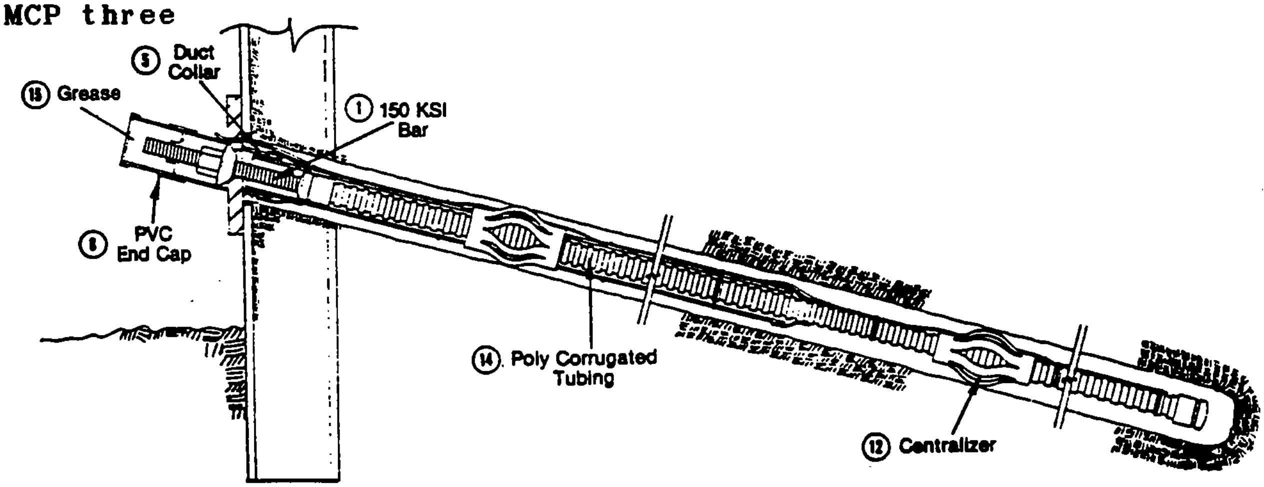
MCP one systems offer two barriers of protection around the bar in the free-stressing length plus the drill hole grout. This system contains a plain, epoxy coated or galvanized bar, a sleeve over the bar in the free-stressing zone and grout inside the sleeve after the tensioning of the anchor.

MCP two



MCP two systems are identical to level one, however, the bond length of the anchor is engulfed in a pre-grouted poly corrugated tube.

MCP three



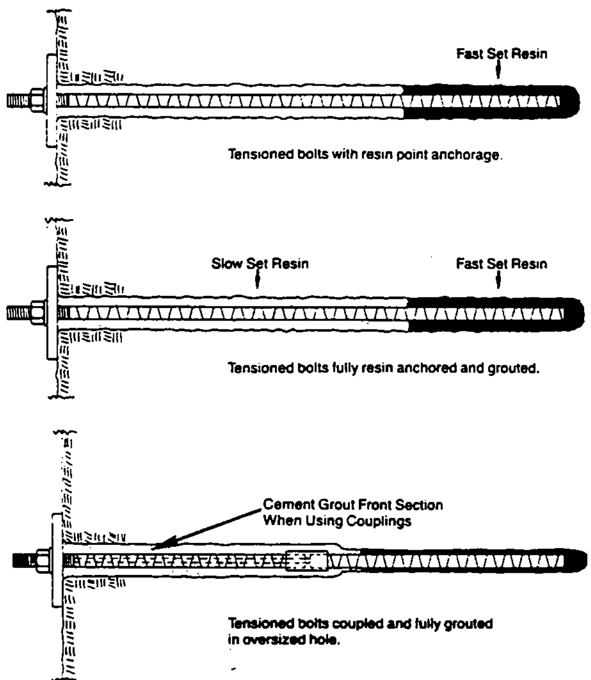
MCP three systems have three barriers around the bar in the free-stress zone and two barriers of protection in the bond zone. The entire bar is encased in a pre-grouted poly corrugated tube. The plate of the anchor is galvanized and a protective end cap is placed over the nut and washer.

Resin grout bonded anchors- Resin rock anchor systems are used for both temporary and permanent applications but predominantly for temporary anchoring. The resin grout is kept inside a plastic cartridge which contains two separate compartments. One compartment holds the polyester resin grout and the second compartment holds it's catalyst. After the hole is drilled the sausage-shaped cartridges are inserted. When rotating the rod through the cartridges, cartridges are torn and the two components mix to create the high compressive strength resin. Although resin systems offer a quick and economical approach to a fully grouted rock anchor, they do have several limitations. Anchors with lengths over 30 feet are difficult to install because of the drag of the bar through the resin cartridges during insertion. Sometimes the resin sets up to quickly before the anchor can be spun to it's full depth. Another point to keep in mind is that the resin anchor system is a pre-measured system. This means that the amount of resin put in the hole is dependent on the number of cartridges inserted and often does not allow for resin loss into fissures and voids along the drill hole length. Essentially this can leave several gaps along the length of the anchor, reducing the safety factor and opening the door for corrosion. Caution should be taken when using resin anchors in moist or wet hole conditions, Moisture, in some cases, can affect the holding capacity of the resin, essentially reducing the safety factor of the bolt.

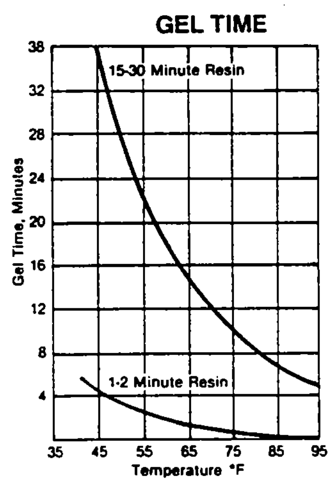
Pre-stressing a resin grouted rock bolt is typically done by one of three ways. One method is to use a fast set resin in the bond length while leaving the free-stress length open. However, this would be strictly for a temporary application. If a resin anchor is used for a permanent application sometimes a fast set resin is used in the bond length and a slow set resin in the free-stress length. This provides some corrosion protection and allows for immediate tensioning and still enables the anchor to elongate in the free-stress length due to the slower setting resin. The last method is to use a fast set resin in the bond length and a cement grout in the free-stress length. Although this is more labor intensive it yields a more complete anchor system. This is because the cement grout will fill all fissures and voids along the free-stress length providing better corrosion protection around the annulus of the anchor. Shown in figure 8 is a diagram of the three methods mentioned above. Also, figure 9 shows resin gel times for both the fast and slow set resin grouts related to temperature. Temperatures below 40 degrees F will significantly slow down set times.

Mechanical anchors- Unlike cement grout or resin anchors, mechanical anchors generate anchorage by mechanical means. They do not rely on a grout bond along an anchor bond length for immediate anchorage. However, mechanical anchors may use grout bond as an additional strength medium in the final installation stage. Mechanical rock anchors typically use a threaded cone and expansion shell located at the end of the anchor rod to produce a frictional grab. Soil mechanical anchors generally utilize a pivoting plate at the end of the anchor which flips open when the anchor rod is tensioned or helical shaped attachments along the length of the anchor rod to transfer load. Mechanical earth anchors tend to be slightly more expensive then grout bonded systems in terms of material costs, however thier installation efficiency often results in labor savings.

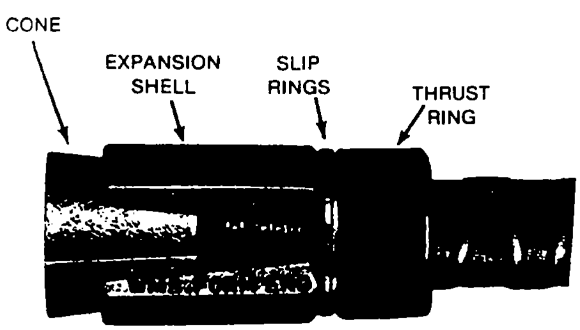
Mechanical rock anchors- When anchoring into rock, a mechanical anchor can be used if the rock is of good quality. For "Spin-lock" type head assemblies, the compressive strength of the rock should be upwards of 2500 psi and the RQD value above 70% for best results. With a serrated type head assembly, anchorage can sometimes be better accomplished in slippery rocks then the "Spin-Lock", However thier load capacities are smaller then the "Spin-locks". Most mechanical pre-stressed rock anchor systems consist of an expansion shell and cone head assembly (fig 10 & 11).



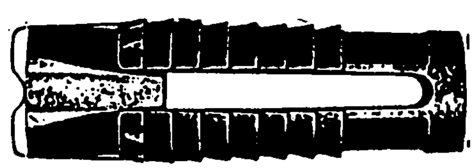
(Figure 8.)



(Figure 9.)

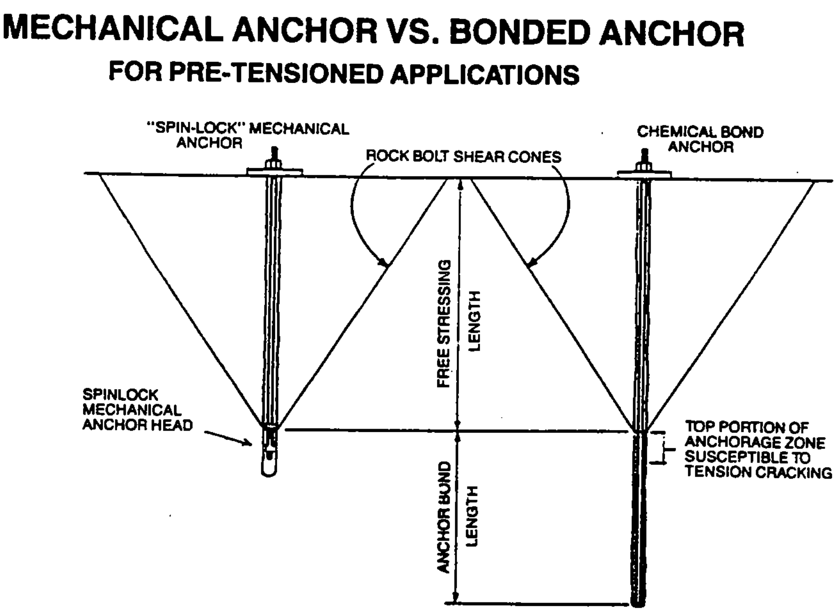


(Figure 10.)

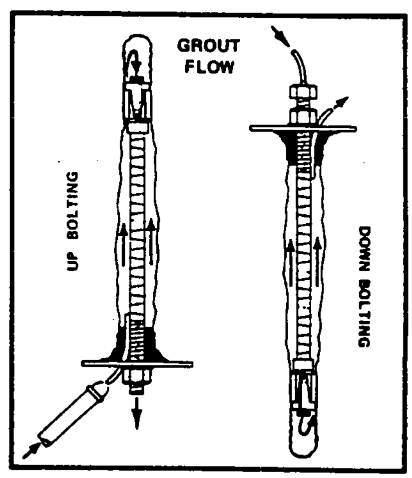


(Figure 11.)

Fig 10. represents a "Spin-Lock" type head assembly , which enables the installer to develop immediate anchorage. A grout bonded anchor would take three or four days to install because the pre-stressing can not be preformed until the grout has reached an acceptable compressive strength. The mechanical anchor rod is initially torqued to begin the expansion process, and once a required torque has been achieved, the anchor can immediately be pre-stressed to the desired working load. Because the anchorage is generated at the mechanical head, a shorter embedment depth can be used in comparison to a grout bonded anchor system (fig 12). Also, because grouting is done after the anchor is pre-stressed, grout cracking caused by prestressing is eliminated. After the anchor has been stressed and tested, the installer can finish the installation by grouting up the hole with a cementitious grout. The grout is there for corrosion protection but it also helps lock in the pres-stress load applied to the anchor. Anchors such as the "Spin-lock" have an oversized head assembly compared to the anchor rod. This allows for adequate grout cover around the anchor bar. Some mechanical type anchors are equipt with a hollow bar, which enables the anchor to be grouted through the center of the rod. This is especially useful and advantageous when installing long rock anchors with couplings or in underwater applications. Grouting through the hollow bar enables the installer to grout from the lowest gravitational point in both up and down bolting applications, ensuring that grout reaches the back of the drill hole (figure 13). Applications such as large scale slope stabilization projects are one application where mechanical rock anchors are ideal. Slopes that are over 100 feet often require a crane for installation purposes. Workers will use a mounted skip (platform) to work from as they install the bolt. The immediate anchorage that is given from the mechanical head allows the workers to complete the bolt installation and not have to return to that spot a second time for grouting or tensioning. This would not be the case with a grout bonded system, resulting in remounting of the skip to the rock face increasing labor costs dramatically.

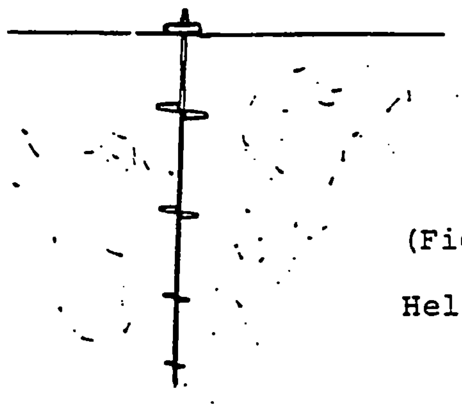


(Figure 12.)



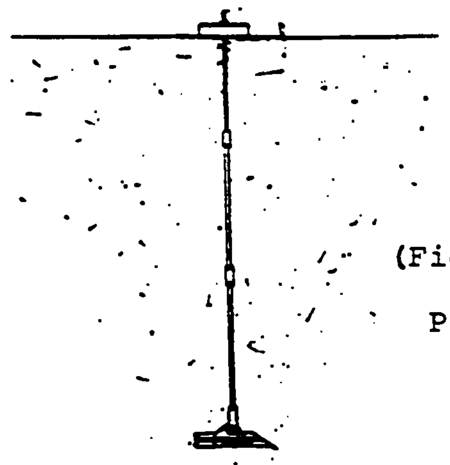
(Figure 13.)

Mechanical soil anchors- These systems develop anchorage by the use of helical shaped plates around the anchor rod which is screwed in the ground, or by a pivoting plate at the end of the rod, which flips open when the rod is pulled back (figures 14 and 15).



(Figure 14.)

Helical Type



(Figure 15.)

Pivot type

Anchor load generated by these systems is very much soil dependent. Correlations between holding capacity and Standard Penetration Counts (N) have been used for the pivoting plate anchor (Manta Ray) to help determine holding capacity given a particular soil. This anchor is pulled back with a load locking device to flip the soil bearing plate open, generating anchorage. Helical anchors are screwed into the soil and often tensioned with a stressing jack. These systems can be galvanized for corrosion protection based on the soils aggressivity. Their main advantage over grout bond systems are ease of installation. However, they can not generate the load that some of the grout bonded systems can because, they utilize smaller bars.

Anchor Installation

Installing pre-stressed earth anchors is based on two to five steps depending on the type of system that is being used. These steps are:

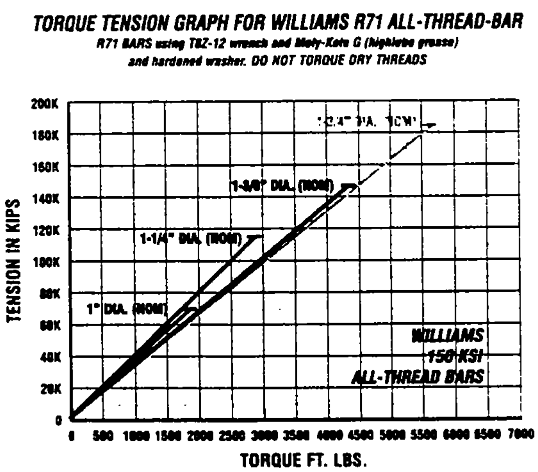
1. Hole drilling
2. Anchor placement
3. Anchor setting
4. Pre-stressing
5. Grouting

Hole drilling- Cement and resin grout anchors along with mechanical rock anchors require a drilled hole for anchor placement. Systems such as the mechanical soil anchors are driven or screwed into the soil and a drilled hole is only necessary if a pilot hole is required. Some of the drilling methods used for anchor installation are rotary, percussion, auger type and core drilling.

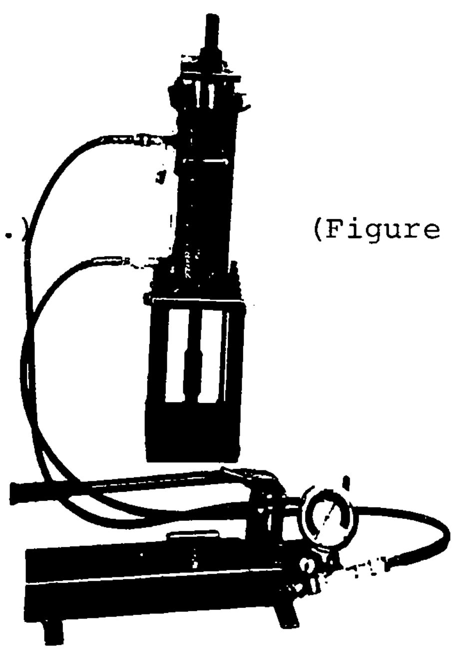
Anchor placement- Smaller diameter anchors are often placed in the drill hole by two or more workers. For larger anchors a crane is often used to lift the anchors as the workers guide it into the hole. In situations where there are overhead restrictions such as in tunnel bolting, anchors are often used with couplings every 5 to 10 feet and installed in sections. When using a mechanical anchor at very long lengths (50 feet or more), sometimes it is necessary to step drill the anchor hole for ease of anchor placement. By drilling the hole larger towards the top, then tapering it down as the depth approaches the area where the head assembly will be, installation is simplified. For resin grout systems the necessary resin cartridges are placed into the drill hole and the bolt is pushed into the hole and rotated by a track drill.

Anchor setting- Setting an anchor is a step that is performed for mechanical anchors only. For expansion types (the "spin-lock") a specified torque on the anchor rod must be achieved to assure that the head assembly has expanded far enough to hold the initial load during prestressing. Setting is usually done with an impact tool which is coupled to the rod with an adapter. The impact tool applies a quick torque to the anchor rod which then can be measured with a torque wrench.

Pre-stressing- Pre-stressing can be accomplished by two ways. The most common is by the use of a hydraulic stressing jack. The jack is coupled to the end of the anchor rod and then activated by either a hand, air or electric pump putting the anchor directly into tension. The jack consists of a ram, pump, gauge, hoses, jack stand, high strength coupling, high strength test rod, plate and hex nut (figure 15). Anchors can also be pre-stressed by torque tensioning methods. However, only certain thread bars will allow this procedure. This is accomplished by torquing the nut to a specific amount which translates to a tension value on the anchor. Torque tension charts as the one shown below (figure 16) are used in determining the relationships. This approach eliminates the use of a heavy hydraulic jack and is especially useful in production bolting with lighter loads on the anchors.



(Figure 16.)



(Figure 17.)

Grouting- Cement grout is used in earth anchoring for devloping bond and for corrosion protection. The grout is delivered into the hole by pumping the grout through a plastic grout tube that runs alongside the anchor rod. Some earth anchors are designed with a hollow bar which enables the installer to grout through the bar ensuring that the grout reaches the back of the anchor. Both systems are often considered acceptable, however grouting through a hollow bar, is more effective.

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MAKING A SOIL NAIL WALL LOOK LIKE ROCK

By Claus Ludwig, Schnabel Foundation Company

Abstract

A permanent soil nail wall was completed in April, 1996 as an emergency slide repair contract for Caltrans (California Department of Transportation). Schnabel Foundation Company was the prime contractor selected by Caltrans to perform this work. This soil nailing project, located near San Rafael, California, was unique for the following reasons:

1. Access was very difficult. The benches and ramps normally required for economical construction of soil nail walls were not available. Therefore, much of the work was performed using equipment and man-baskets suspended by a 65 ton crane, and by having workers use ropes to tie off to safety cables.
2. The permanent shotcrete wall facing was sculpted and stained to give the wall the appearance of adjacent natural rock formations and soil.
3. The project was an emergency contract.

The work consisted of constructing approximately 3000 square feet of soil nail wall and approximately 3000 square feet of shotcrete slope protection below the soil nail wall. The height and length of the affected area were approximately 60-70 ft and 100 lineal feet, respectively. The work was performed during the rainy season and required two months to complete, working long hours six and seven days a week.

Introduction

On January 18, 1996, a landslide triggered by heavy rains forced the closure of two of three southbound lanes on Highway 101 near San Rafael, located approximately 15 miles north of San Francisco. Emergency crews were able to clear the slide debris from the highway that night in time to allow all lanes to re-open for traffic for the morning commute.

Caltrans decided that given the history of slide problems at this site, some means of permanent slope stabilization was required. Highway 101 is used by auto commuters living north of San Francisco who cross the Golden Gate Bridge every morning to reach San Francisco. With about another two months remaining until the end of the rainy season, Caltrans was concerned that if it followed normal contracting procedures, another slide with more serious consequences could occur before permanent slope stabilization could be performed. In addition to the concern for the safety of motorists traveling on Highway 101 and on the county road (Tiburon Boulevard) above the slide, Caltrans was well aware that a major slide could cause serious long-term traffic problems in the area.

Caltrans opted to award an emergency contract to Schnabel Foundation Company so that the stabilization work using approximately 3000 square feet of soil nail wall and 3000 square feet of shotcrete slope protection could proceed immediately. The slide area covers a length of approximately 100 lineal feet. The slide area is shown in Figure 1.

FIGURE 1: MARIN COUNTY SLIDE

Soil Nailing Background

The first soil nail walls were constructed in France in 1972 and the first reported use of soil nailing in the United States was in 1976 (Byrne et al, 1993). Design and installation methods have been developed primarily by specialty contractors. While its primary application has been for temporary excavation support, soil nailing has become increasingly popular for permanent wall construction. Soil nailing is a top-down method of construction and is used to reinforce the in situ strength of soil. Soil nails are passive anchors which rely on lateral deformation of the excavated soil mass to mobilize the forces in them. Soil nail walls are generally designed using limit equilibrium methods which consider the nail to act as a tension element.

The three major operations in soil nailing are:

1. Excavation
2. Installation of the soil nails.
3. Installation of drain material, reinforcement and shotcrete facing. If the wall is a permanent wall, the exposed facing is most commonly cast-in-place, with shotcrete and precast panels being used less frequently.

   Step 2 is normally performed before Step 3.

Excavation proceeds downward in lifts not to exceed approximately five to six feet. In almost all applications, temporary benches on the order of 20-30 ft wide are provided at each lift. One essential requirement for economical soil nail construction is that the soil should have sufficient short term strength to allow a vertical or near vertical cut to stand open for eight hours or more. If this short term stability cannot be achieved, other alternative earth retention methods should be considered.

The spacing of soil nails is typically on the order of five feet in each direction. Design lengths vary and depend on factors such as the soil unit weight and strength, the soil nail grout to soil adhesion, surcharge loading, sloping above or below the wall, the presence of water and right of way constraints.

Soil nail holes are almost without exception drilled in as opposed to driven. The drill holes are advanced using rotary or percussion drilling techniques. The drill hole diameters typically range from four to twelve inches in diameter. In general, for soil nailing to be economical, the drill holes should remain open approximately a minimum of four to eight hours. If cased methods of drilling are anticipated, alternative retention methods should be considered.

Soil nail reinforcement is typically .75 to 1.375 inch diameter steel bar having a minimum yield strength of 60,000 psi. For most permanent applications within the United States, corrosion protection ranges from the epoxy coating of the bar reinforcement to a double corrosion protection system in which the bar reinforcement is encapsulated with grout-filled corrugated sheathing. The soil nail reinforcement is centered in the drill holes using plastic centralizers. The drill holes are grouted with either a sand-cement-water mixture or a cement-water mixture.

A percentage (5 % is typical) of the nails are proof tested to verify assumptions made in the design. If the nail test loads do not satisfy the test requirements, the original design needs to be re-evaluated. This typically entails performing additional analysis and design by modifying soil-to-grout adhesion, nail lengths, spacing and drill hole diameters as required to achieve an adequate safety factor.

The facing thickness can vary from three to four inches with nominal mesh and bar reinforcement for temporary applications, to six to twelve inch thick facings with one to two mats of reinforcement for permanent applications. Drainage is typically provided using geocomposite drain installed in vertical strips one ft wide with weep drains.

Design

The emergency slope clearing had left a steep backslope of 1/2:1 to 1/4:1 and steeper in the highly weathered siltstone, which was overlying sheared shale and graywacke sandstone. Caltrans' design required a battered soil nail wall approximately 35 ft high and 100 ft long with a permanent shotcrete facing, and permanent shotcrete slope protection for an approximate slope height of 30 to 40 ft below the wall. Horizontal drains 100 ft long were to be installed at the toe of the slope. The important design features are shown in cross-section in Figure 2.

Other anchored wall methods besides soil nailing, such as tieback walls with or without soldier beams, are also used for slope stabilization. As a design-build contractor specializing in earth retention systems, we are very familiar with the relative merits of the different anchored wall systems. From our point of view, the choice by Caltrans to construct a soil nail wall was the correct one.

Firstly, the soil conditions satisfied two of the main factors required for economical soil nail construction, namely that:

1. The excavated face could stand vertical or near vertical for a height of five to six feet for a minimum time period of eight hours.
2. The soil nails could be installed using open hole drilling techniques.

Battering the wall gave a more pleasing appearance than a vertical wall would have by minimizing the difference between the existing slope outside of the slide limits and the existing slope within the slide limits. Caltrans has designed and constructed many permanent soil nail walls in the state of California. The use of a shotcreted permanent facing for this soil nail wall was a deviation from Caltrans' standard practice of using cast-in-place facings. Part of the resistance by Caltrans of using permanent shotcreted facings has been their concern about the aesthetics of the exposed facing.

The soil nails were designed with lengths ranging from 35 ft at the top of the wall to 20 ft at the base of the wall. The drill hole diameter required was four inches. The soil nail spacing was 5 ft vertically and horizontally. The nail bar reinforcement was epoxy-coated #8 (1 inch diameter)

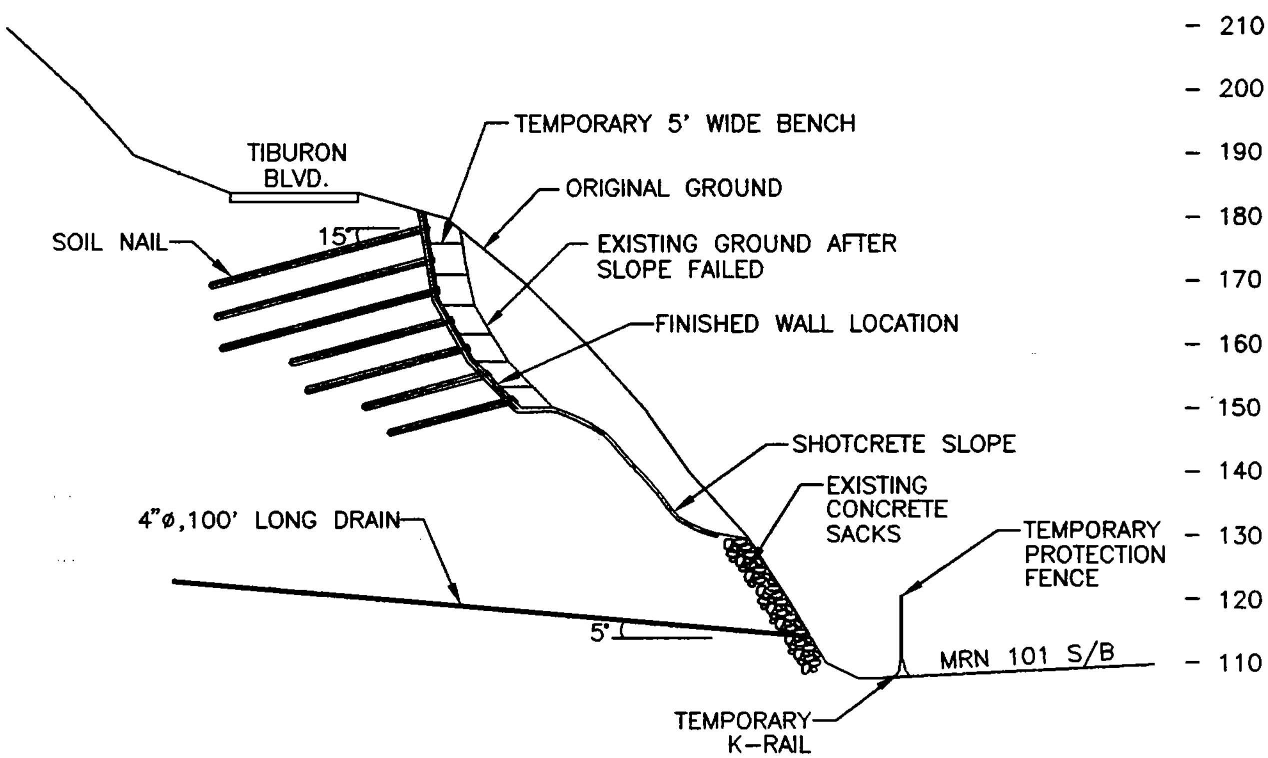


FIGURE 2. MARIN EMERGENCY SLIDE REPAIR
TYPICAL CROSS SECTION
1/20"=1'-0"

Grade 60 (i.e., yield strength = 60,000 psi). The soil nail grout was a mixture of cement and water mixed at the job site.

The facing thickness was about 11 inches applied in approximately two equal applications. Each application was reinforced with a single mat of bar and mesh reinforcement. The 10 inch square bearing plates were placed against the initial application of shotcrete. Horizontal shear reinforcement was designed to tie the two applications of shotcrete together. The final thickness included two inches more than what was required structurally. The additional two inches allowed the wall to be sculpted to give it the appearance of rock.

The main differences in design between this soil nail wall and a more conventionally designed wall can be summarized as follows:
1. This wall was designed to be battered.
2. This wall was designed with a permanent shotcrete facing using two applications of permanent shotcrete, as opposed to the conventional design which would have used a wasted temporary shotcrete application overlaid by a permanent cast-in-place facing.
3. The final wall appearance was sculpted shotcrete and colored staining made to look like rock.

The slope protection was nominally 6 inches thick with one mat of mesh reinforcement and no soil nail reinforcement.

Construction

Due to the fact that the project was an emergency one, we did not make our first visit to the job site until we had already agreed to perform the work. Our first viewpoint was from Highway 101 below the slide. This viewpoint is similar to the one shown in Figure 1. The view was ominous to say the least. We had difficulty imagining being able to perform this work from below. The reach to the slide area was significant, but the greater concern was the stream of traffic along Highway 101. Working from below would have required a significant disruption to traffic on the highway because of our equipment requirements. Cranes required to perform the work would have necessitated closure of the shoulder plus a minimum of one traffic lane.

When we viewed the project site from the county road (Tiburon Boulevard) located above the slide, the project looked only slightly less imposing. We decided to perform the work from the county road above the slide, instead of from the highway below. Working from below would have required highway lane closures and night work. Caltrans was able to obtain permission from Marin County to close Tiburon Boulevard above for the duration of our work (just over two months). Working from above had its disadvantages, however:
1. A power line was located over the roadway. Since this overhead power line was to remain active for the duration of the project, this required us to be extremely careful when using cranes and excavating equipment on the roadway. The overhead line and our setup on Tiburon Boulevard can be seen in Figure 3.
2. The equipment operators had to rely on observing hand signals because their view below the wall was obstructed.

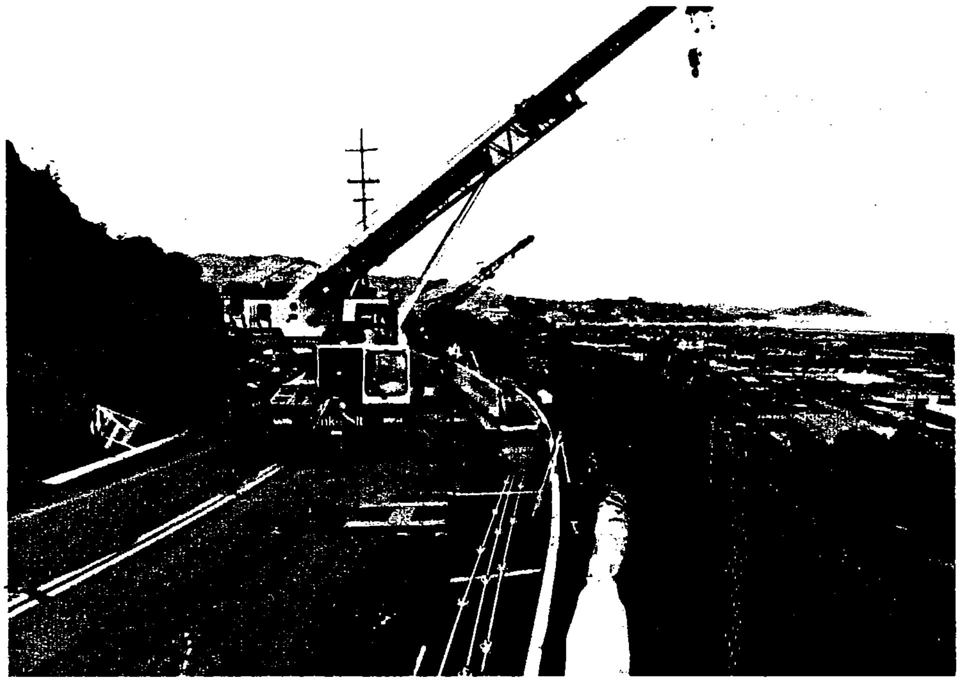


FIGURE 3: SET-UP ON TIBURON BLVD.

3. The equipment and materials placed on the roadway above exerted additional surcharge on the distressed soil mass.

The lead time of one to two weeks required for the fabrication and delivery of the permanent soil nail anchors gave the project participants limited time to plan the work.

Due to the emergency nature of the work, the design was still being performed even after construction was underway. This actually gave the entire project team, consisting of Caltrans, Schnabel Foundation Company and our subcontractors, better flexibility to respond to actual field conditions. This flexibility was important because it was an unusual project for everyone involved. Caltrans modified the design as time progressed and was very receptive to many suggestions we had regarding design and construction. Likewise, we also benefited from the input of Caltrans' field representatives in performing our work.

One significant change which was agreed to prior to construction is a perfect example of teamwork that was employed to complete the project. Caltrans' original design required that the soil nail wall be installed against the failed slope (1/2 to 1/4 horizontal to 1 vertical) with only minor scaling of the slope to be performed. We felt we could work more productively and safely if we could excavate a 5 ft wide bench for every 5 ft height of wall we worked on. Economical soil nail wall construction is predicated on having temporary benches on the order of 25-30 ft for optimum production because this allows sufficient width for drill rigs, concrete trucks, forklifts and other equipment to operate on the working bench. While the 25 to 30 ft bench was clearly not available here, we felt that a 5 ft wide bench would at least be wide enough to allow workers to stand on, which would give them much more maneuverability in performing their work. Without these small benches, all of the work would have had to be performed out of man-baskets supported by cranes. This would have been much less productive and certainly more unsafe. The 5 ft wide bench provided the additional benefit of allowing the wall to be constructed against more intact soil (i.e., as compared to the soil at the already exposed face). These benefits offset the additional cost of excavation.

Work on the project began on January 30, less than two weeks after the slide had occurred. The first items of work included closing Tiburon Boulevard, installation of a protective fence attached to the k-rail on the shoulder of the highway below, and clearing and grubbing at the top of the wall. The fence was installed to prevent debris from falling down the slope on to the highway below.

The construction of the 35 ft high soil nail wall followed, in general terms, the following procedure:

1. Excavate, using a combination of excavator, or clamshell where the excavator could not reach, and hand excavation, a 5 ft high lift, while providing a 5 ft wide bench. The bench, where provided, was wide enough to enable the work crews to stand on the bench. Because of the unevenness of the slope, we, in fact, at times had places where no bench was available and where all of the work had to be done out of a man-basket. The approximately 100 lineal feet of wall at each lift required 1-2 days to excavate.
2. Install drain material, bar and mesh reinforcement and install an initial 5.5-6 inch thick

application of shotcrete. Apply a curing compound when directed to do so by Caltrans. This required approximately 1 day to complete per lift.

3. Drill and grout 4 inch diameter soil nails whose lengths varied from 35 ft at the top to 20 ft at the bottom of the wall. For drilling, the mast from an air track drill rig was chained to bar reinforcement protruding from the shotcrete and supported by a 65 ton hydraulic crane which was positioned on Tiburon Boulevard. The face was shotcreted first (Step 2) because the drill mast had to be anchored to the wall to resist the reaction forces caused by the drilling. The drilling and grouting of approximately 20 holes per lift required 2 days on average to complete.
4. Step 1 was repeated for the second 5 ft high lift.
5. Step 2 was repeated for the second 5 ft high lift.
6. Step 3 was repeated for the second 5 ft high lift.
7. The 10 ft high section of wall was sandblasted to ensure that there would be a good bond between the two applications of shotcrete. A second mat of reinforcement and the final sculpted 5.5 inch thick application of shotcrete were installed. The sculptors carved fractured lines and relief to simulate natural cliff formations in the area. Synchronization of the shotcrete nozzleman and the sculptors was crucial because once the wet shotcrete was applied to the wall, the sculptors only had about one hour to perform their work before the shotcrete would set (see Figure 7). The sculptured texture and fractured pattern of the wall were designed as the wall construction proceeded downward. The free sculpting technique and design of the sculptured wall were the result of the collaborative efforts between Caltrans' landscape architect and the specialty sculpting subcontractor. Step 7 required 1-2 days to complete.
8. After the 10 ft high section of wall was completed, steps 1 through 7 were repeated until the wall was completed.

Steps 1 through 3 are shown schematically in Figure 4, Steps 4 through 6 in Figure 5 and Step 7 in Figure 6.

After the wall was completed, work began on the slope below the wall. Since Caltrans required the simulated rock look for the slope as well, the slope was cleaned of loose dirt, drain material, mesh and bar were installed, irrigation lines for landscaping were installed, blockouts for landscaping were placed, and a minimum 6" thick application of shotcrete was placed. The work on the slope was very difficult because no benches were available and all the work was done from man-baskets or by standing on the slope while being tied off using safety harnesses and lanyards to safety cables secured to the shotcrete wall. The work on the slope required another 10 days to complete. We could have avoided placing workers on the slope by using a remotely controlled nozzle to apply fiber-reinforced shotcrete. However, then we would not have been able to sculpt the slope.

The wall and slope were pressure washed and readied for the staining process. The colors, selected to match the surrounding rock formations and soil, were applied with a hydrochloric acid stain which reacts with and impregnates itself into the shotcrete to develop a permanent color tone. The stains were applied with multiple coats which were overlapped to create the natural mottled appearance. The natural aging process will actually improve the appearance of the wall. The final wall appearance is shown in Figure 8. This photo was taken just after the horizontal drains had been installed and before the dust generated by the horizontal drain drilling was washed away. The final

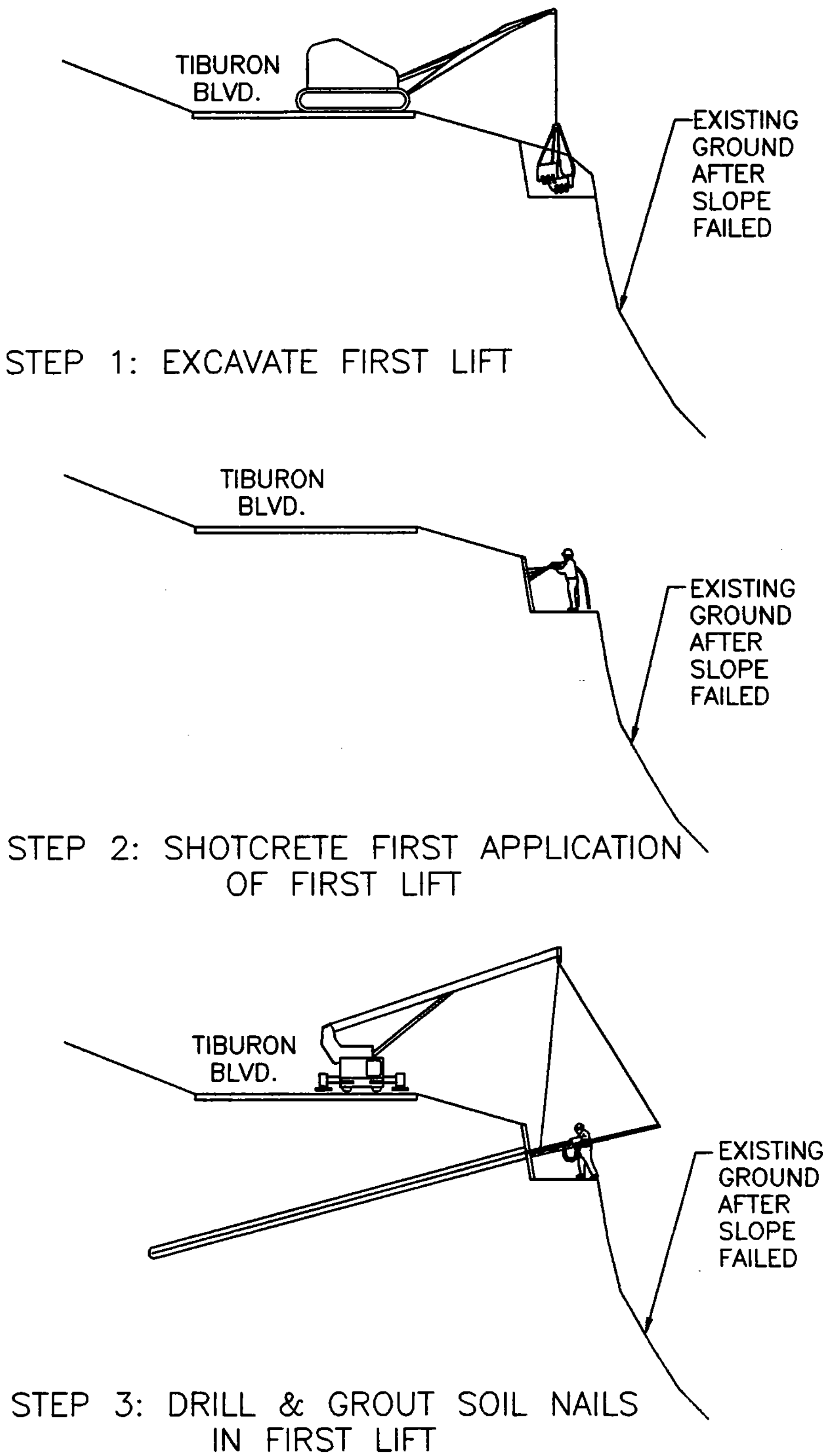
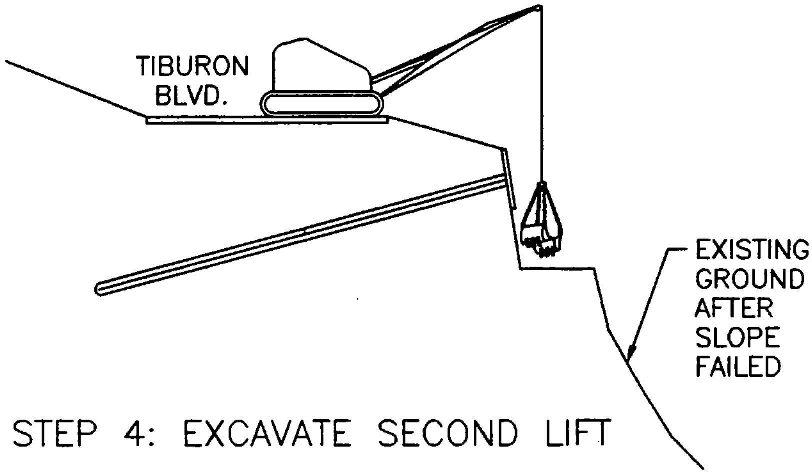
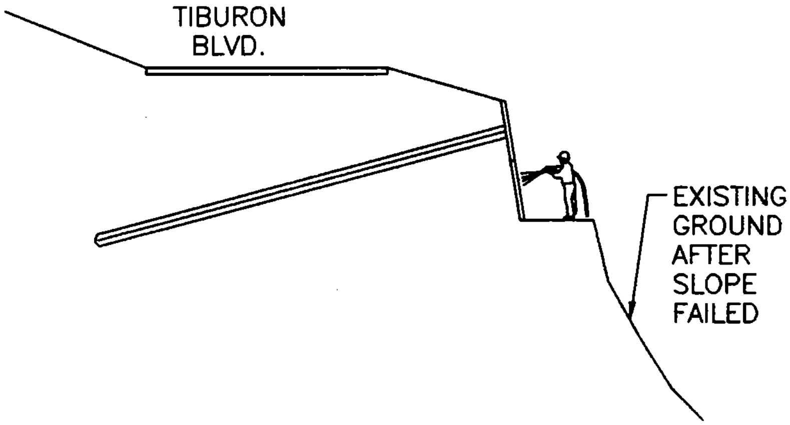


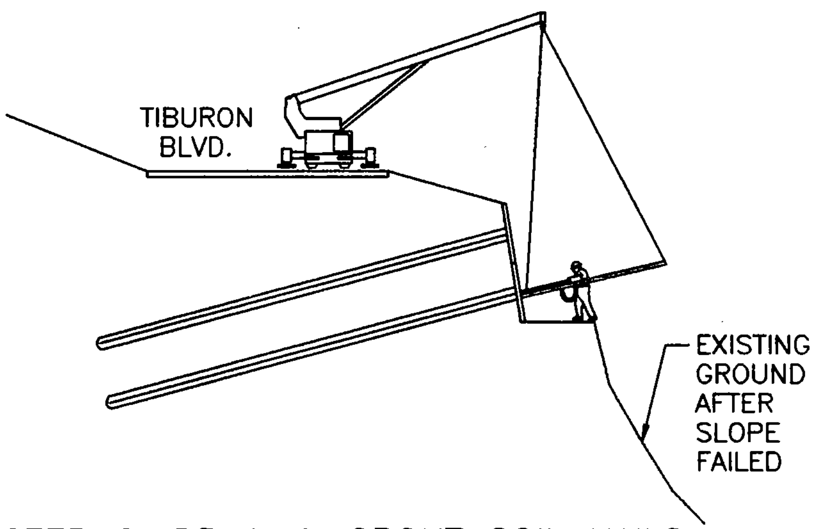
FIGURE 4. STEPS 1, 2, 3 OF SOIL NAIL WALL CONSTRUCTION



STEP 4: EXCAVATE SECOND LIFT



STEP 5: SHOTCRETE FIRST APPLICATION
OF SECOND LIFT



STEP 6: DRILL & GROUT SOIL NAILS
IN SECOND LIFT

FIGURE 5. STEPS 4, 5, & 6 OF SOIL NAIL WALL CONSTRUCTION

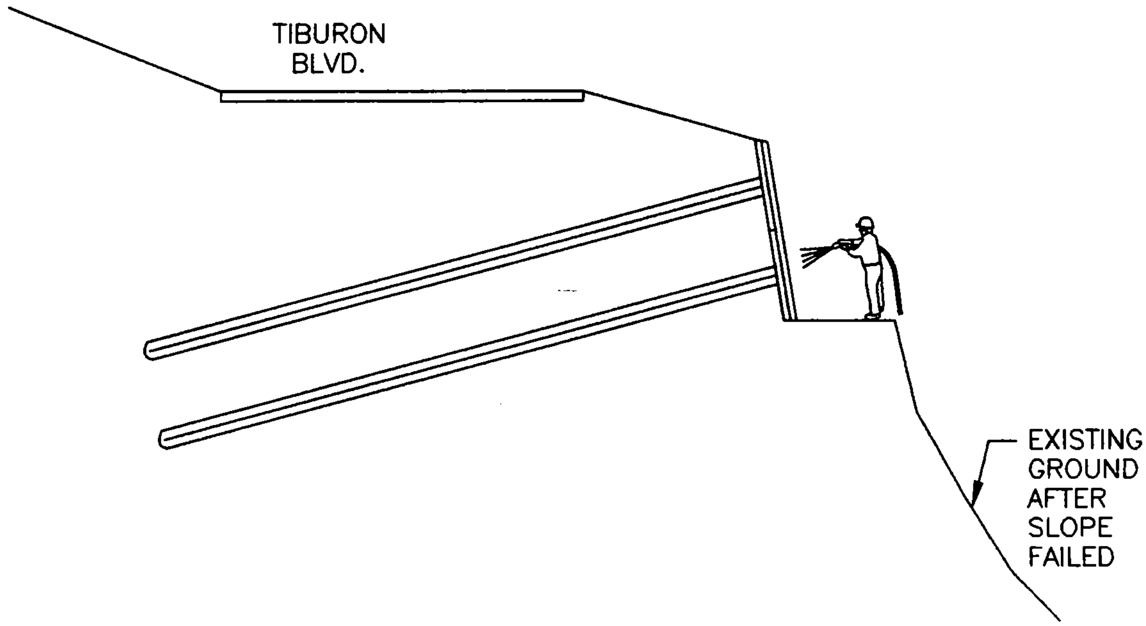


FIGURE 6. APPLY FINAL FINISHED APPLICATION OF SHOTCRETE FOR TWO LIFTS

FIGURE 7: SHOTCRETING AND SCULPTING PERMANENT FACING

FIGURE 8: FINAL WALL APPEARANCE

main item of work was the installation of five 100 ft long 4" diameter horizontal drains which were drilled at night from the highway below. The drains were drilled with a track-mounted Krupp drill rig and down-the-hole hammer.

We had serious concerns about safety on the project. There was concern for the workers and Caltrans' employees. All workers were given training in fall protection and crane safety. We were also concerned about the safety of motorists traveling on Highway 101 below. We had to be careful not to allow anything to fall which might break through the debris fence and on to the traveled highway. Fortunately, the work was completed without any injuries or accidents.

Conclusions

This was a challenging soil nail wall project because it was an emergency, difficult access project performed in rainy weather. It was a success because the final product is an aesthetically pleasing, quality wall which blends in very well with the environment. The quality appearance was achieved by giving the wall the appearance of rock rather than that of a wall. Further testimony to the success of the project is the fact that Caltrans has already designed another wall similar to this one which is scheduled to be advertised for construction soon.

This emergency contract was successful largely as a result of the open communication between the designer (Caltrans) and the contractor (Schnabel Foundation Company and its subcontractors). This project was, in effect, a pseudo-design-build project because the design evolved from direct communication between the designer and contractor. A project such as this can certainly benefit from a design-build approach. This approach maximizes the communication between the designer and builder because they are one and the same. For projects such as this one, where the schedule is critical and the access is difficult, the designer can communicate directly with the builder to make sure that the design is compatible with the construction techniques available so that the project can be performed in accordance with the schedule and project specifications. The cost of this wall was on the order of twice the cost for a wall constructed with good access. The wall design and construction methods described in this paper can most certainly be utilized on similar projects throughout the country.

Acknowledgments

The author wishes to acknowledge Caltrans, Ken Sweet and other field employees of Schnabel Foundation Company, Maggiora and Ghilotti, Robert Hill and Robert Hill and Associates, and Sacramento Valley Crane for their contribution towards making this a successful and special project.

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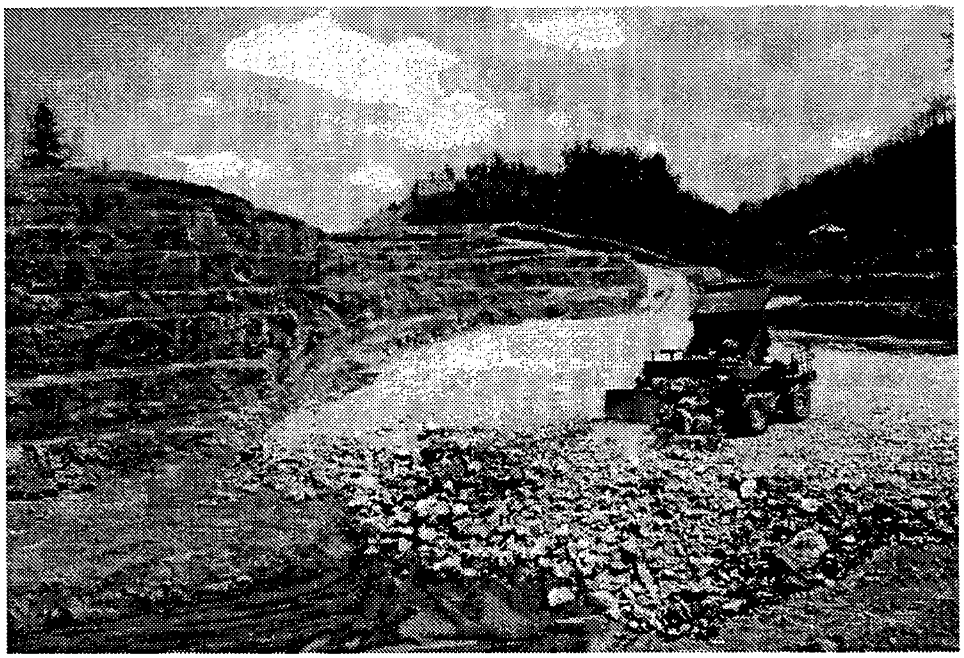
THE USE OF UNDERBENCHING IN EMBANKMENT CONSTRUCTION THROUGH MOUNTAINOUS TERRAIN - I-26 UNICOI COUNTY, TENNESSEE



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THE USE OF UNDERBENCHING IN EMBANKMENT CONSTRUCTION THROUGH MOUNTAINOUS TERRAIN - I-26 UNICOI COUNTY, TENNESSEE

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 underbenching procedure involved the removal of unstable materials (colluvium, soft soils) and the excavation of benches into firm weathered rock and in-place rock. Graded solid rock was then placed as a rock pad over the benches previously cut into the surface. Once the rock pads were completed, then embankment construction proceeded to grade.

A total of 41 embankments required underbenching which resulted in the removal of 2,413,496.6 cubic meters (3,159,027 yds.^{3}) of unstable material. Rock pad construction required the use of over 1,646,387.7 cubic meters (2,154,957.7 yds.^{3}) of processed rock.

Total costs for the 24 kilometer (15 mile) project were just over 170 million dollars with approximately 10 million dollars being used for the underbenching concept.

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 (Figures 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 (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.

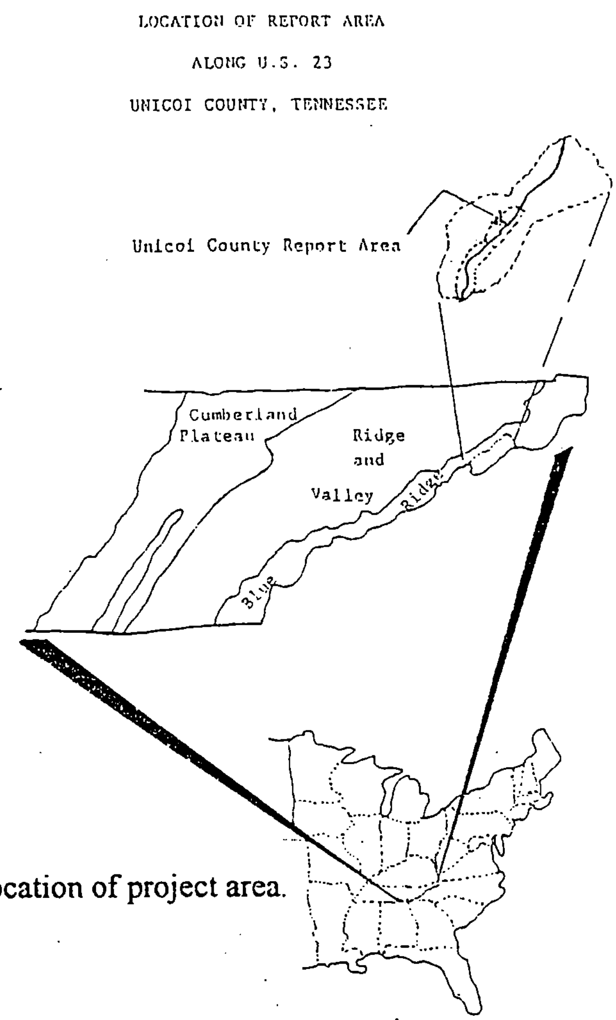

FIGURE 1. General location of project area.

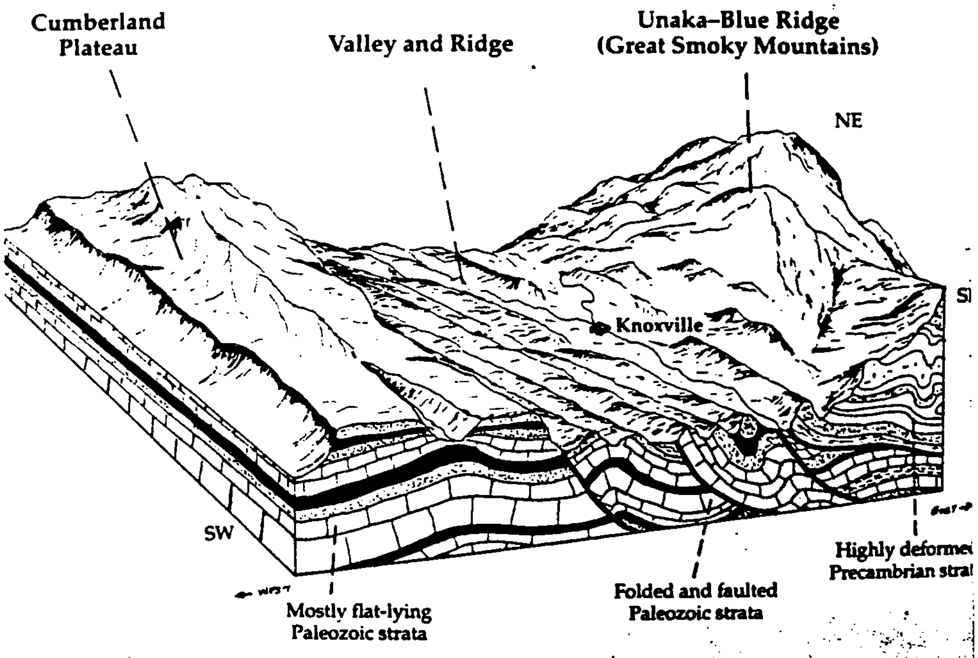

FIGURE 2. Schematic drawing of East Tennessee showing geologic structure and topography.

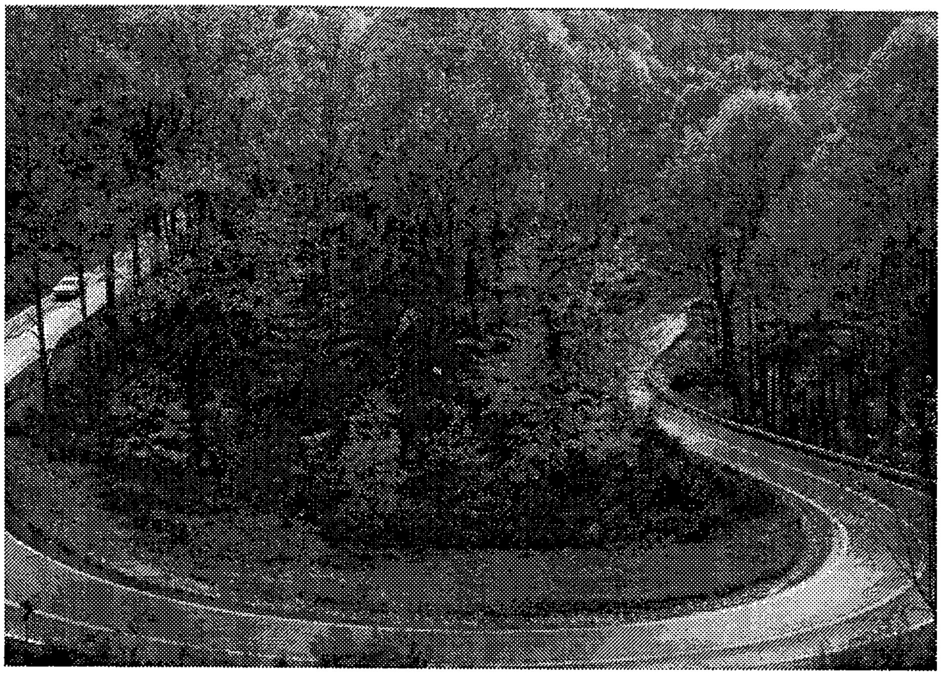

Figure 3: Old U.S.23 highway near Sams Gap (Tennessee/North Carolina state line).

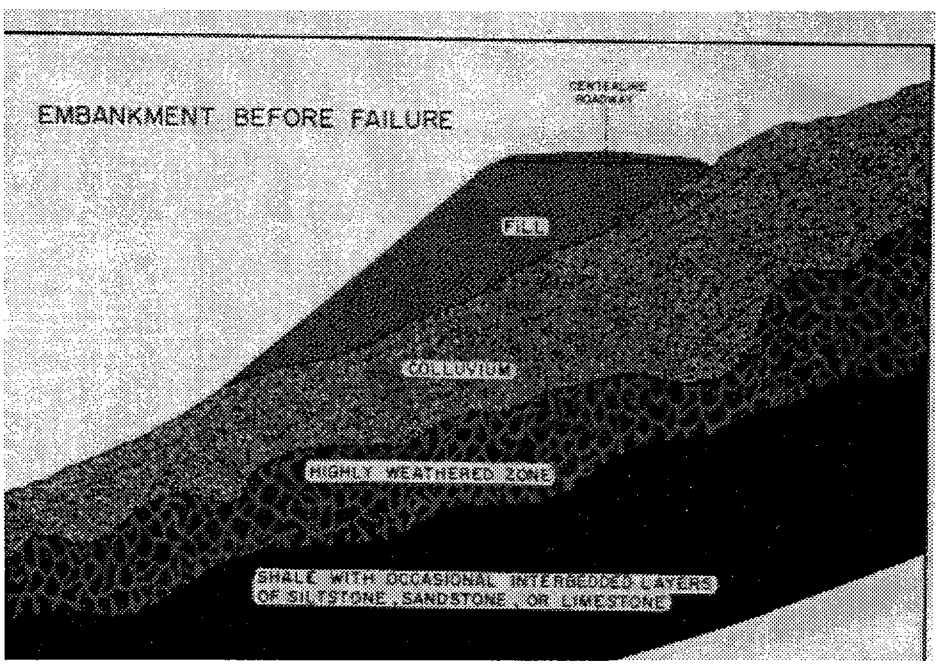

Figure 4: Schematic drawing of highway embankment constructed on unstable material (colluvium, highly weathered material).

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, implacing numerous discontinuities. Weathering and erosion of the exposed rock has produced a mantle of saprolite and colluvial material.

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 cm (2 INCHES) IN

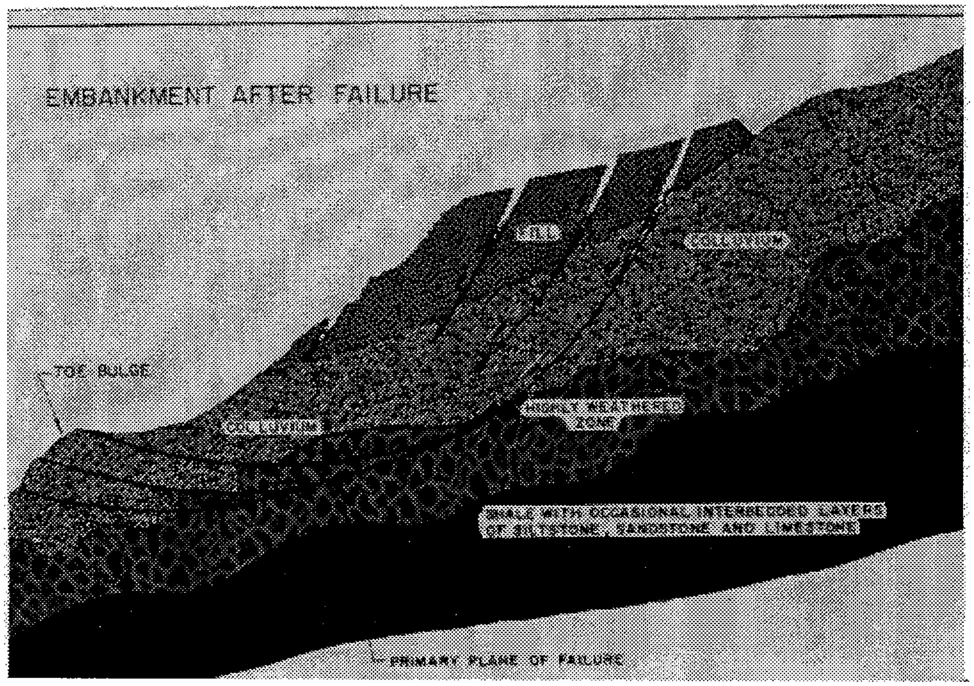

Figure 5: Schematic drawing showing failure of embankment due to unstable foundation material.

Figure 6: Actual roadway embankment failure due to unstable foundation material (S.R. 61, Union Company)

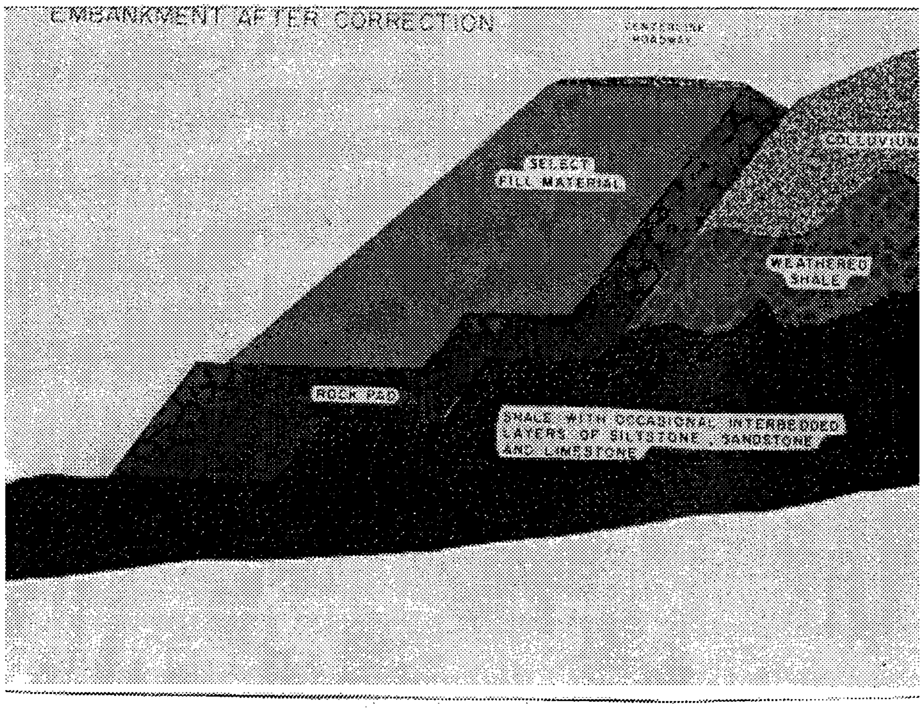

Figure 7: Schematic drawing illustrating underbenching concept with rock pad and engineered roadway embankment.

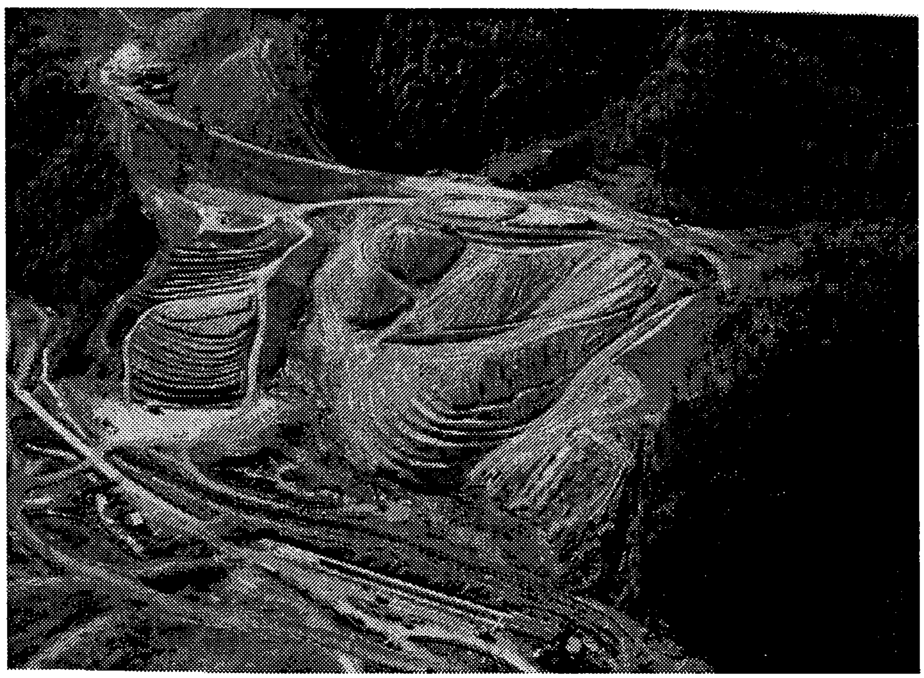

Figure 8: Large embankment area along future I-26 construction showing underbenching of natural slopes before placement of fill material.

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 (Figure 8). Vertical slopes and benches are excavated in this fashion until the unstable material is removed.

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 9). The embankment construction proceeded in conjunction with the rock pad construction where feasible, and followed

Figure 9: Processed rock from the project is placed over all underbench areas to provide for drainage and stability of the embankment.

Figure 10: Large slide-hill fill area where underbenching for embankment construction is underway.

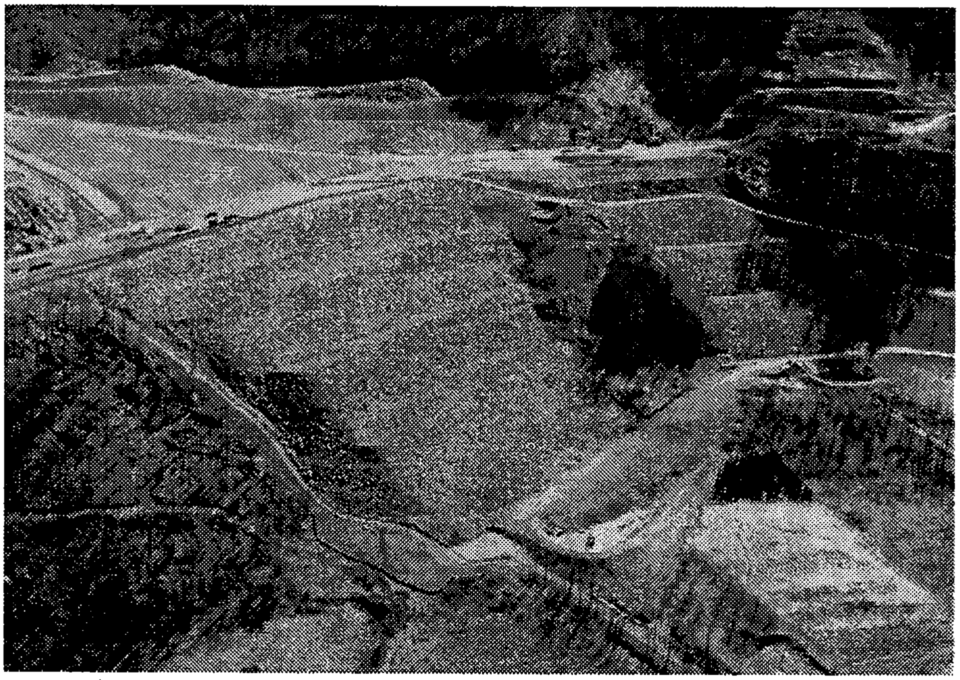

Figure11: Large embankment that contains extensive underbenching and rock pad on future I-26, Unicoi County, Tennessee.

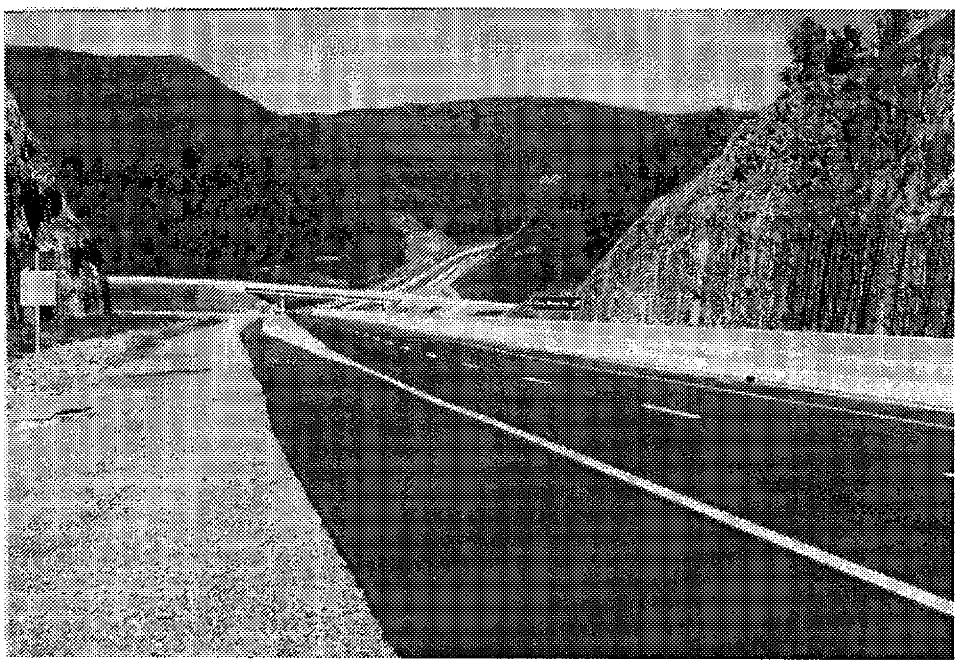

Figure 12: Completed roadway for future I-26 near Flag Pond, Tennessee.

standard embankment construction specifications (Figure 10). 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).

SUMMARY

The underbenching concept was employed by the Tennessee D.O.T. to provide long term stability for large highway embankments in the steep mountainous terrain of Unicoi County, Tennessee. The new location for U.S. 23 (Future I-26) required the construction of numerous large side-hill embankments in order to traverse the mountainous landscape.

Underbenching was provided for 41 embankments which required the removal of 2,413,496.6 cubic meters (3,159,027 yds.^{3}) of unstable material. Rock pad construction required the use of over 1,646,387.7 cubic meters (2,154,957.7 yds.^{3}) of processed rock.

Total costs for the 24 kilometer (15 mile) project were just over 170 million dollars with approximately 10 million dollars being used for the underbenching concept (Figure 12).

REFERENCES

Tennessee Department Of Transportation, 1995. Standard Specifications for Road And Bridge Construction. Manual effective March 1, 1995, p.82-83.

ROCK DURABILITY OF ARGILLACEOUS CARBONATE ROCKS IN CUT SLOPES FOR INDIANA HIGHWAYS

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Abstract

In Indiana, highway rock cuts are typically constructed through horizontally bedded, sedimentary rocks of marine origin. The Mitchell Plain Physiographic Division of south central Indiana extends from a few miles south of Indianapolis to the state boundary on the Ohio River. Carbonate rocks of Mississippian age comprise the rocks in this prominent physiographic unit.

Slope deterioration and subsequent rockfalls are caused by sloughing and weathering of the less durable units. Also, pore pressure build-up and toppling of blocks cause slopes to recede. A recent study by West (1994) demonstrates that resistance against toppling is reduced by nearly 50% when pore pressures are added.

Two rock cut sites were evaluated along Indiana State Road 37, south of Bloomington. The dual-lane road was constructed 23 years ago and the more argillaceous carbonate units have experienced accelerated deterioration. Both a field investigation and a laboratory testing program (slake durability) were performed. Erosion rates were compared to petrographic details and evaluated in regard to the current literature. Recommendations are provided for estimating in advance the erodibility of rock cuts in a carbonate sequence.

Introduction

Relocation of Indiana State Road 37 (S.R. 37) south of Bloomington, Indiana was accomplished in the 1970s. The former two-lane road from Bloomington to Mitchell, a distance of about 30 miles was eventually replaced by the new dual-lane highway. Within this overall relocation project, two portions were selected for analysis in this study. The first and major part, approximately 2.5 miles in length, was completed in 1973, and extends northward from the south side of Clear Creek into the adjacent limestone hills. The second portion is a cut section, up to 15 feet high, extending for a distance of about 600 ft. It is located approximately 6 miles north of the Clear Creek site, both on the west side of SR 37.

The highway relocation area lies within the Mitchell Plain Physiographic Section of Indiana, a landform subdivision in the unglaciated portion of the state. The Mitchell Plain has moderate to low relief developed on Middle Mississippian, essentially horizontal, limestones; they commonly display numerous sinkholes and solution channels. Bloomington lies about 50 miles south of Indianapolis.

Both of the selected sites occur in southern Monroe County, with the Clear Creek site located about 10 miles south of Bloomington and 2 miles north of the boundary with Lawrence County. The second site, located 0.2 miles north of Victor Pike (an east-west road) is situated only a few miles south of Bloomington. Since 1973 these road cuts have undergone significant erosion, yielding rock overhangs and rock falls. It is likely that pore pressure build-up behind large blocks added to forces that produced falls and topples (West, 1994).

The road relocation design provided for two dual lane pavements, 24 feet wide consisting of 7 inches of continuously reinforced concrete over a 6 inch crushed stone base course. The shoulders were 10 feet wide on the outside and 4 feet wide on the inside of the paved sections. The median was 60 feet wide. The shoulders consisted of 3 inches of bituminous pavement over 4 inches of compacted aggregate base.

Detailed information is available on the Clear Creek site (ATEC, 1969). Based on construction documents, it extends from Station 769+50 on the south to Station 900+00 on the north. Included within this 2.5 mile project was a rock cut up to 95 feet high in the limestone bedrock.

The Clear Creek site is located just north of the Harrodsburg exit, near the extensive, earth-filled dam which impounds the Monroe Reservoir. This feature is shown in Figure 1 as well as dual-lane highway 37. Note that old S.R. 37 lies about 1/3 of a mile to the west of the new dual highway.

Borings were obtained for the Clear Creek site in 1969, prior to highway design (ATEC, 1969). Six borings into the limestone bedrock were drilled for this road cut. These Nx borings (2-1/8 inch diameter) ranged from 30 to 105 feet in depth. Copies of two of the six original boring logs are provided in Appendix A of this paper (Borings 44 and 46). Note that core recovery data are included in the logs but no RQD values are provided. Although commonly measured today, in 1969, RQD values were not routinely determined for highway construction projects. Such data if available would likely have proved useful in this study.

Table 1 is a summary of depth information for the six borings. Boring 44 extends to the lowest elevation, about 556 ft. In keeping with highway procedures, borings in cut areas are drilled about 7 feet below the planned grade line. Note that 5.4 feet of the Borden Group (siltstone) is shown in the log for Boring 44 (Appendix A). The contact elevation is 561.7 feet. This is the same rock unit that forms the foundation and underlies the reservoir for Monroe Lake; refer to Figure 1; normal pool elevation for the reservoir is indicated as 538 ft.

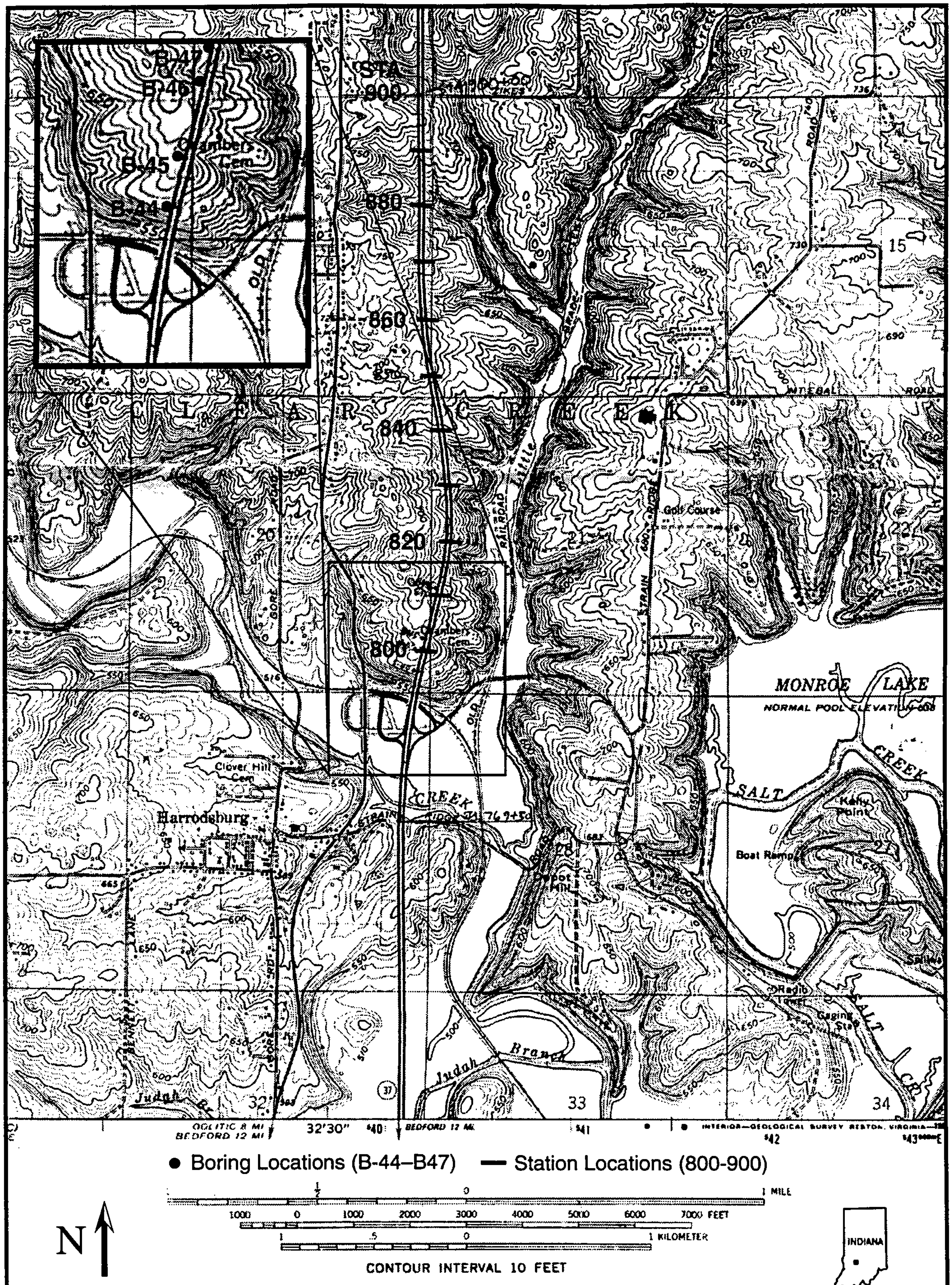

Figure 1 Locations Map, State Road 37, Clear Creek, Indiana

In 1975 a stratigraphic description of the 2000 ft. long road cut was published by the Indiana Geological Survey (Nicoll and Rexroad, 1975). A summary version of this description is provided in Table 2 for the 128 foot section. Note that 2.8 feet of the Borden Group are shown at the base of the section. Using the elevation of the upper contact for the Borden Group of 561.7 ft. as established above, this yields an upper elevation for the section of 687.5 ft. This is consistent with the 690± ft. surface elevations for most of the borings.

Table 1. Bedrock Borings in Cut Area, SR 37 near Harrodsburg.

Boring No.	Station Number	Offset (ft)	Surface Elevation (ft)	Boring Depth (ft)	Elevation Bottom of Boring (ft)	Approx. Grade Line Elevation (ft)
44	769+25	85L	644.3	88	556.3	563
45	802+00	85L	688	105	583.0	590
46	811+00	85L	690.5	44.7	645.8	653
46A	810+72	81.5L	690	72	618.0	625
46B	810+96	119L	693.7	45	648.7	656
47	816+00	85L	663.5	30	633.5	640

* L indicates Left, or in this situation, to the west.

Rock cuts were made on a 1/6 to 1 slope, horizontal to vertical, with pre-shearing holes drilled at a 3.5 to 5 ft. spacing. The cut consisted generally of limestone with occasional shale and clay seams. Sinkholes were noted at several locations in the project area (see the map of the project, Figure 1).

Good rock recovery was obtained for all the bedrock borings except for Boring 46, drilled 85 ft. left of Station 811+50. In this boring, a cavity was noted between 15.7 and 29.0 ft. deep, and another between 29.7 and 36.0 ft. deep. Boring 46A drilled 81.5 ft. left of Station 810+72, indicated solid limestone for the full depth of the boring, 72 ft. Boring 46B, drilled 119 ft. left of Station 810+96 also indicated solid limestone for the full depth of 45 ft. of drilling. These two additional borings suggest that the cavities found in Boring 46 extend only a relatively short distance and likely can be accommodated during construction. Photographs of the Clear Creek site are shown in Figure 2.

The second site (Victor Pike) lies higher in the geologic section than does the Clear Creek site. This section, about 600 feet long, is located on the west side of SR37. Maximum rock height in the cut is about 15 ft. The site contains some massive beds at the top of the cut, which fail along joint planes after undercutting has occurred. Photographs of the Victor Pike site are shown in Figure 3.

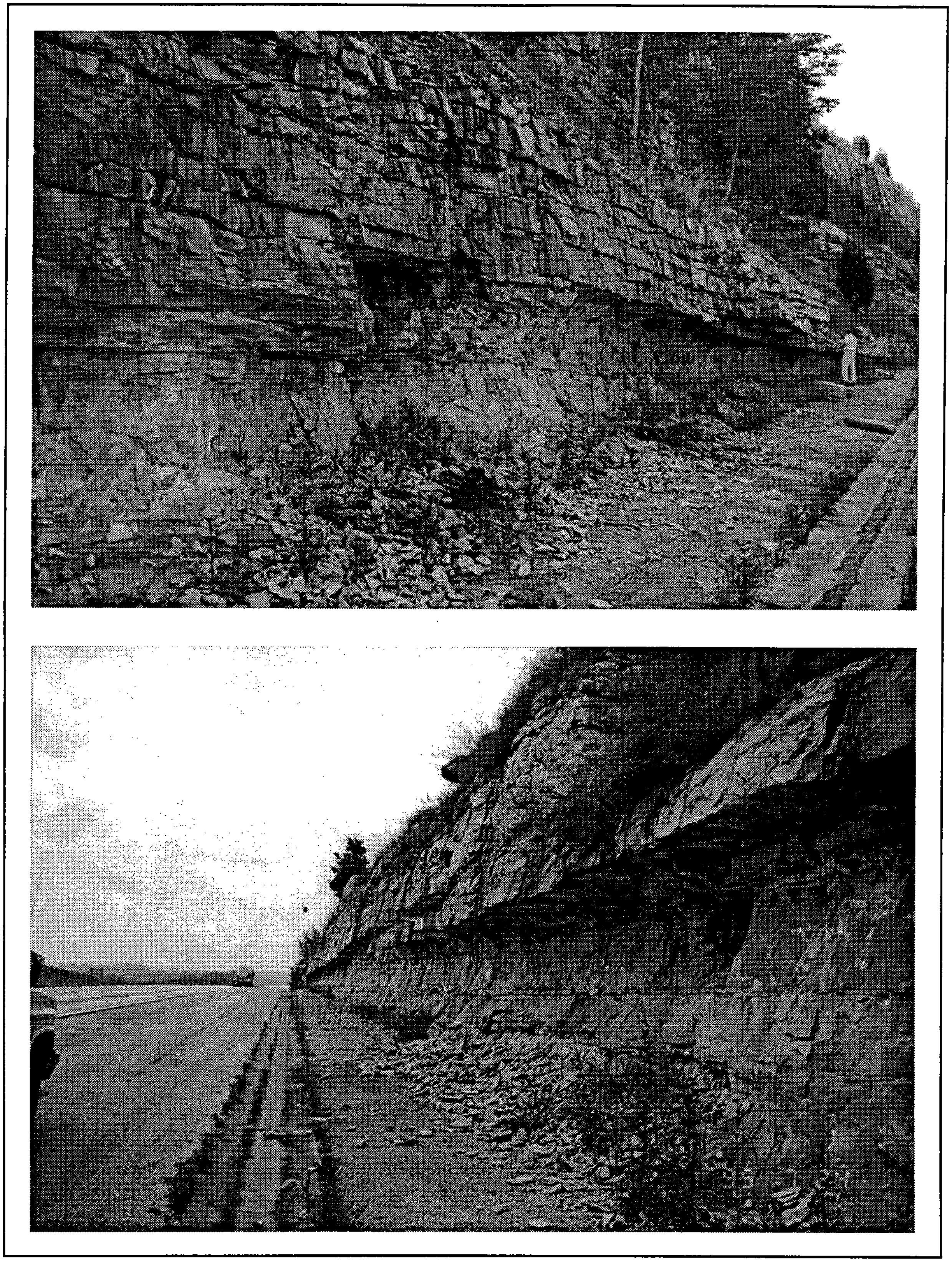

Figure 2. Photos, Clear Creek Site

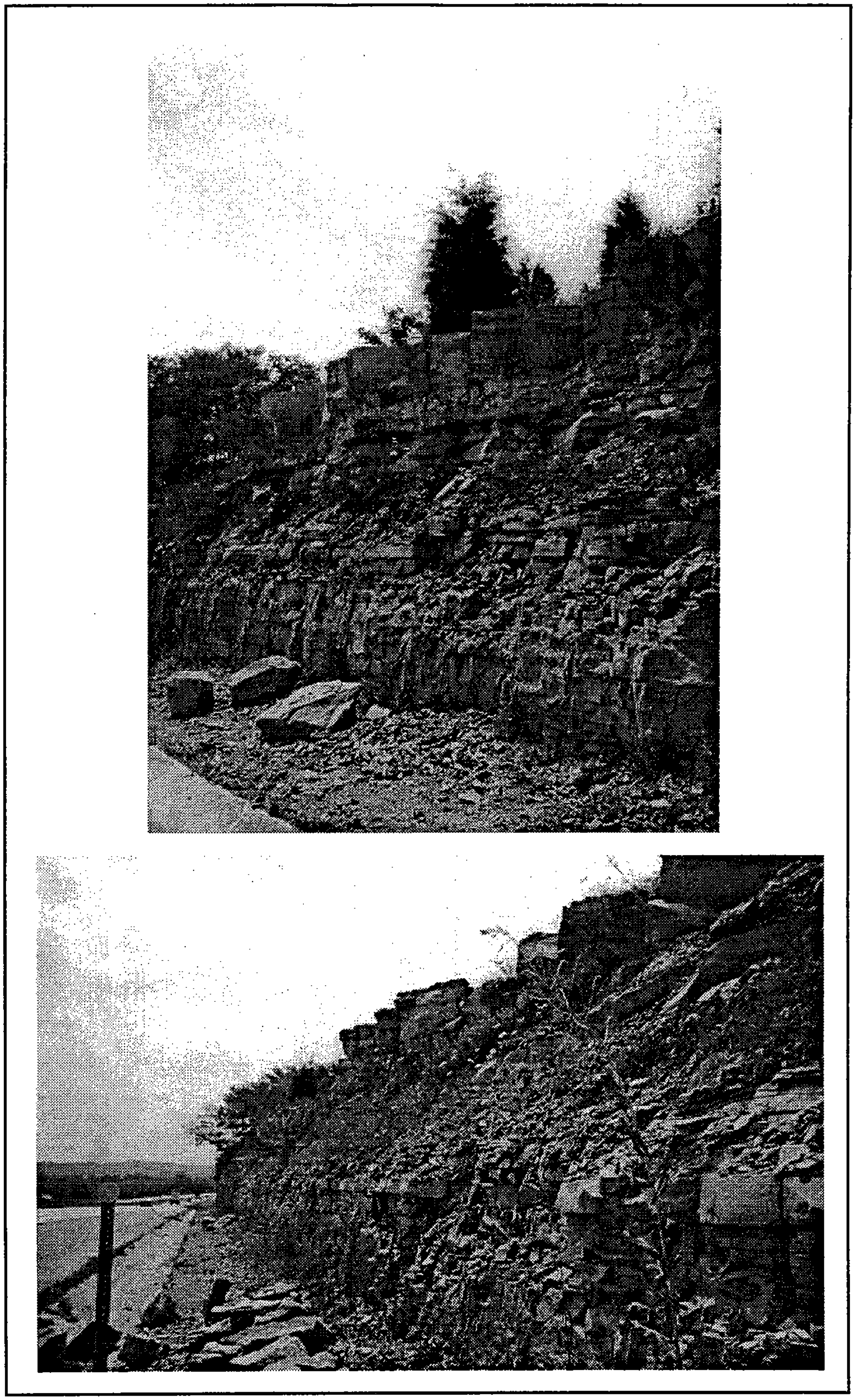

Figure 3. Photos, Victor Pike Site

Table 2. Measured Section

Road cut on Indiana Highway 37, North Bluff of Clear Creek, about one mile north of Harrodsburg, SW1/4 Sec 20, T7N, R1W, Monroe County, IN, Clear Creek 7-1/2 minute Topographic Quadrangle Map, 128 ft. measured. (Adapted from Nicoll and Rexroad, 1975.)

33'	Salem Limestone		1. Limestone, gray weathers to tan, mostly biocalcarenite. 33 ft.
		33'	
67.9'	Harrodsburg Limestone		2. Limestone w/irregular shale partings and a siltstone lens, 56.5 ft. 3. Limestone and lenticular siltstone, argillaceous, geodes present, 11.4 ft.
		100.9'	
24.9'	Ramp Creek Formation		4. Limestone and lenticular siltstone, 2.9 ft. 5. Interbedded limestone and dolosiltite, weathers to rusty brown, 22.0 ft.
		125.8'	
2.8'	Borden Group		6. Siltstone, dark bluish gray, argillaceous
		128.6'	
	Base of Cut		

Analysis of Slope Deterioration

In the summer, 1995 the authors examined the condition of the highway rock cuts along the dual lane State Road 37 south of Bloomington. A total of five road cuts were considered initially, but it was subsequently decided to concentrate the study on two areas showing the greatest effects of deterioration. As indicated above, the first is located immediately north of Clear Creek and the town of Harrodsburg. In this section the highway grade is fairly steep to the south toward Clear Creek (about 3%) extending for a distance of about 2000 ft. The highway continues downward through the geologic section exposing a vertical distance of 128 feet. The maximum rock cut for the 2000 ft. section is approximately 95 ft., near Stat 802. The lowest rock unit encountered, the Borden siltstone occurs at the south end of the cut about Station 769. The stratigraphic column for the cut is provided in Table 2.

Ledge samples were taken at the two sites. Samples were obtained from the highway pavement level and up to a height of about 10 ft. Individual rock units were sampled along the length of the two cuts. The extent of overhang of the beds was measured at the sample locations.

Figure 4 indicates the extent of the overhanging beds for the Clear Creek site. It consists of a cross section from south to north along the west side of the cut. The maximum amount of overhang observed is 38 inches. 1973 through 1995 yields a span of 23 years, thus indicating a rate of 1.65 in/yr or 4.2 cm/yr.

Figure 5 depicts the Victor Pike site, showing again the extent of overhanging beds and the sample locations. This cut slope is considerably smaller than the Clear Creek site, both in height and in overall length. The maximum amount of overhang shown is 18 inches. Applying the time span of 23 years of exposure, this yields 0.78 in/yr or 2.0 cm/yr. Seven ledge samples were obtained from Victor Pike site and five from the Clear Creek site. A megascopic petrographic description for each sample is supplied in Table 3. Petrographic examination of aggregates or of rocks used in construction is an important part in the study of rock quality purposes for engineering (West, 1995). Many of the ledge samples from rock units that showed a considerable amount of erosion, leading to overhanging rock units contained appreciable amounts of clay. The adverse affects of argillaceous constituents in carbonate aggregate was noted some years ago by West and Shakoor (1984).

Laboratory Testing

Two tests were used to evaluate sample durability, the slake test and the slake durability test. The slake test, originally developed to provide an indication of rock behavior during alternating wetting and drying, does, to some extent, simulate the effects of rock weathering. The

Table 3. Megascopic, Petrographic Description of Ledge Samples from SR37 Rock Cuts.

S1-1A Gray, fine grained, argillaceous dolomite w/o visible fossils.

S1-1B Gray, fine grained argillaceous limestone w/o visible fossils

S1-1C Gray, fine grained, argillaceous dolomite w/o visible fossils.

S1-1D Light brown, fine grained, thinly bedded, argillaceous limestone w/o visible fossils.

S1-1E Light brown, fine grained, thinly bedded, argillaceous dolomite w/o visible fossils.

S-1-2A Gray, fine grained, slightly argillaceous limestone w/o visible fossils.

S-1-2B Gray, fine grained, slightly argillaceous limestone w/o visible fossils.

S-2-1A Light gray, fine grained, slightly argillaceous limestone w/o visible fossils.

S2-2A Light gray, crystalline, medium grained limestone w/some fossil fragments.

S2-3 Light gray, fine grained, argillaceous dolomite w/o visible fossils.

S2-4 Gray, fine grained, thinly bedded, argillaceous limestone w/o visible fossils.

S2-5 Gray, coarse grained, crystalline to clastic, limestone w/some fossil fragments.

S1 samples from Victor Pike site and S2 samples from Clear Creek site.

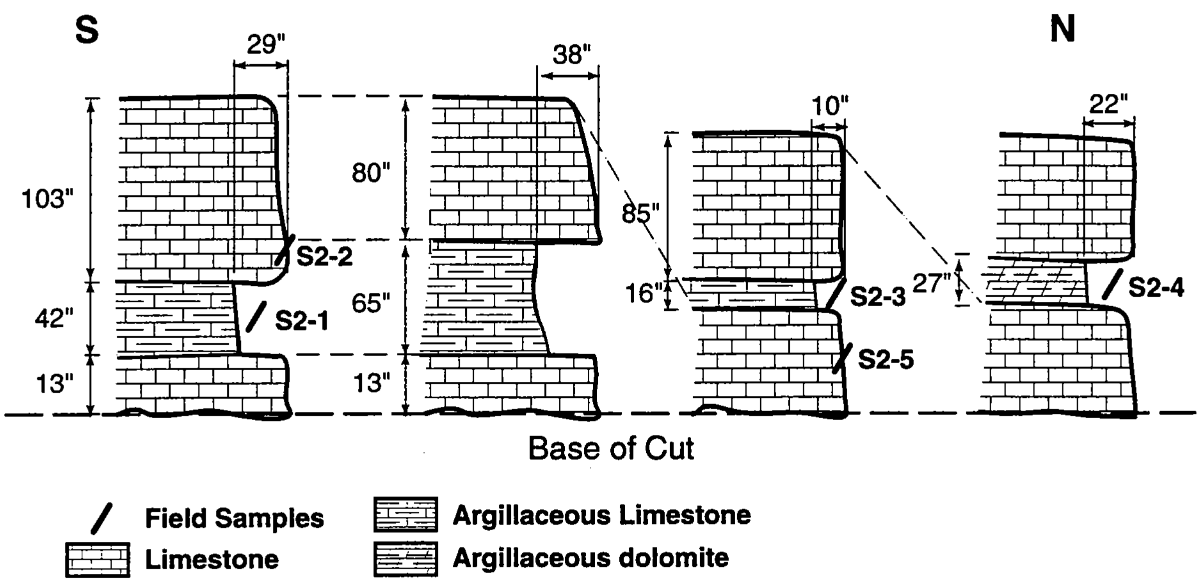

Figure 4. Cross Section of Clear Creek Site Showing Overhanging Beds.

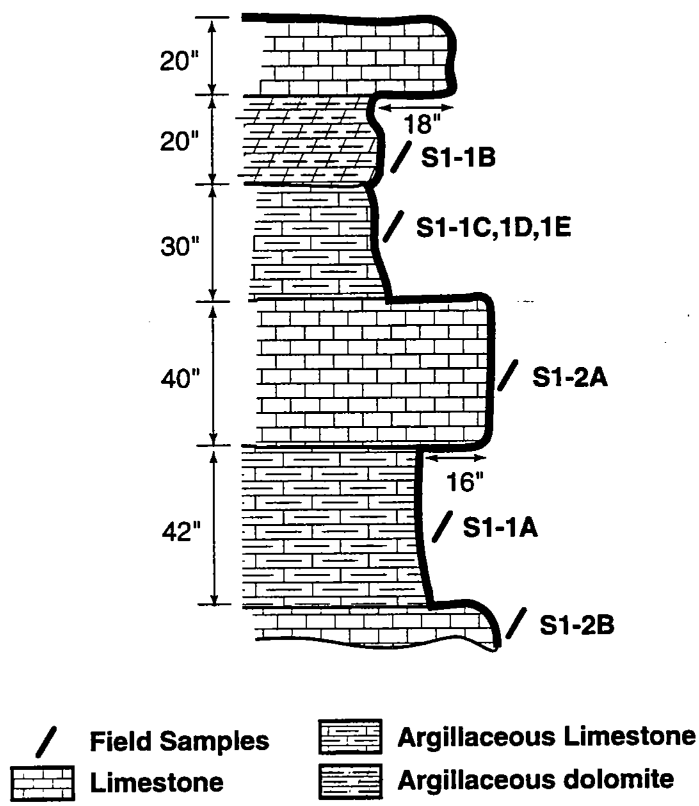

Figure 5. Cross Section of Victor Pike Site, Showing Overhanging Beds.

test procedure and its application have been discussed by several authors, including Wood and Deo (1975), Chapman et al. (1976), Withiam and Andrews (1982), and Hopkins and Deen (1984). The procedure is briefly described below.

1. Select a broken piece of sample weighing between 50 and 60 g.
2. Dry sample to a constant weight at 105°C and record dry weight. (Note: sample drying is an important step; the following test procedures must not be performed on a field-moist sample.)
3. Cool sample for 30 min at room temperature.
4. Place the sample in a jar and cover with distilled water. Check the condition of the specimen after 10 min, then at 1, 2, 4, 8, and 16 hr. duration.
5. Remove the specimen from the water and check for any change in pH of the water.
6. Dry to a constant weight and record weight of portion retained on 2-mm (No. 10) sieve. (Note: recording weight of intermediate cycles is desirable so that results may be compared with those of the slake durability test.)
7. Photograph specimen if significant degradation has occurred or at the end of the last test cycle.
8. Repeat procedure four additional times, or until total degradation, yielding five cycles.
9. Calculate the slake index for the sample.

$$S_I = \frac{\text{(original weight - final weight)}}{\text{original weight}} \times 100$$

This simple test usually will identify poorly performing shales in a matter of hours. If the specimens are quite resistant, however, this time consuming test will allow only a qualitative estimate of durability.

The slake durability test (ASTM D-4644-87) was developed by Franklin (1981). It is considerably more severe than the slake test. This was the second test performed on the rock samples. Laboratory testing using this device is shown in Figure 6.

For the slake durability test, a wire-mesh drum made with 2-mm (No.10) mesh, is rotated while partially submerged in a trough of water. The axis of the 140-mm-diameter drum is 20 mm above the water surface. The test is performed in the following way:

1. Select 10 pieces of sample (40 to 60 g each) with a total weight of approximately 500 g.
2. Identify and photograph the group adjacent to a millimeter scale.
3. Place rock fragments in the drum. Weigh drum and samples together. Place drum in an oven and dry the sample to a constant weight at 110°C.
4. Compute natural moisture content; then mount drum in trough.
5. Rotate the drum at 20 revolutions per minute for 10 min.
6. Remove the drum from the water, rinse, dry in oven, and weigh drum and sample remaining.
7. Repeat the process to produce two cycles.
8. Calculate the durability index as follows:

Figure 6. Slake Durability Testing. a) Water-filled reservoirs prior to testing;
b) Drums containing aggregate rotating in reservoirs during testing.

$$I_D = \frac{\text{weight of sample remaining inside drum}}{\text{original weight of sample}} \times 100$$

Test Results

The test results for the slake test are shown in Table 4. S1 samples are from the Victor Pike Site and S2 samples are from the Clear Creek Site. Note that the results are mixed, not all argillaceous rocks have significant weight changes but most do, and not all limestones are unaffected.

Table 4. Results of Slake Test, S1 samples from Victor Pike Site, S2 samples from Clear Creek Site.

Sample	Lithology	Before Slake Test (g)	After Slake Test (g)	Change of Weight (g)	%
S1-1A	Argillaceous dolomite	53.3	53	0.3	0.563
S1-1B	Argillaceous limestone	57.4	57.3	0.1	0.174
S1-1C	Argillaceous dolomite	57.3	56.8	0.5	0.873
S1-1D	Argillaceous limestone	70.1	68.1	2	2.853
S1-1E	Argillaceous dolomite	69.5	67.2	2.3	3.309
S1-2A	Limestone	61.6	61.6	0	0
S1-2B	Limestone	49.5	49.5	0	0
S2-1	Limestone	61.6	61.5	0.1	0.162
S2-2	Limestone	68.3	68.2	0.1	0.146
S2-3	Argillaceous dolomite	61.7	61.7	0	0
S2-4	Argillaceous limestone	62.3	62.3	0	0
S2-5	Limestone	69.4	69.4	0	0

Results of the slake durability test are shown in Table 5.

Table 5. Results of Two Cycle Slake Durability Test.

Sample	Lithology	Before Slake Test (g)	After Slake Test (g)	Change of Weight (g)	Change of Weight (%)	Slake Durability Index (%)
S1-1A	Argillaceous dolomite	477.8	472.3	5.5	1.2	98.8
S1-1B	Argillaceous limestone	475.8	472.6	3.2	0.7	99.3
S1-1C	Argillaceous dolomite	456.8	449.6	7.2	1.6	98.4
S1-1D	Argillaceous limestone	315.7	296.7	19	6.0	94.0
S1-1E	Argillaceous dolomite	326	314.6	11.4	3.5	96.5
S1-2A	Limestone	457.5	453.8	3.7	0.8	99.2
S1-2B	Limestone	507.4	503.9	3.5	0.7	99.3
S2-1	Limestone	466.4	460.5	5.9	1.3	98.7
S2-2	Limestone	441.9	438.5	3.4	0.8	99.2
S2-3	Argillaceous dolomite	449.9	441.8	8.1	1.8	98.2
S2-4	Argillaceous limestone	490.6	469.4	21.2	4.3	95.7
S2-5	Limestone	447.2	442.9	4.3	1.0	99.0

Evaluation of the test results show that the argillaceous carbonates have lower slake durability index values than do the limestone samples containing less clay. S1-1D for the Victor Pike site and S2-4 for the Clear Creek site show the poorest durability; both are argillaceous carbonates. Table 6 below is a comparison of amounts of erosion at the rock cut locations to the slake durability index for that unit.

Table 6. Comparison of Erosion Depth Versus Slake Durability Index.

Sample	Erosion Depth (inches)	Erosion in/yr	Slake Durability Index
S1-1A	16	0.70	98.8
S1-1B	18	0.78	99.3
S1-1C	18+	0.78+	98.4
S1-1D	18+	0.78+	94.0
S1-1E	18+	0.78+	96.5
S2-1	29,38?	1.26,1.65?	98.7
S2-3	10	0.43	98.2
S2-4	27	1.17	95.7

Shakoor has related Slake Durability Index to the rate of undercutting for mudrocks (Shakoor, 1995; Dick and Shakoor, 1995). The rock cuts in the current study did not contain mudrocks, but rather they included argillaceous carbonates that are considerably more durable than mudrocks. According to Shakoor (1995), rocks with a durability index ranging from 30 and 100 tend to follow the straight line $y = 2.10-0.0119x$ where y is the maximum rate of undercutting in in/yr and x is the durability index (I_D).

Figure 7 is a plot for the enlarged section of Shakoor's diagram for I_D ranging from 90 to 100. The 12 samples with estimated values for rate of undercutting versus I_D from Table 5 are shown as squares on the straight line. For the eight samples with a measured depth of erosion (Table 6), the actual values have been added as circles. These plot both above and below the line.

The results indicate that there is a fair amount of scatter around the straight line within the range from 0.9 to 1.0 in/yr of erosion. This is not unexpected as we are operating at the extreme upper end of Shakoor's data on mudrocks where information is limited.

It is of interest to observe that an erosion rate of about 0.75 in/yr is a reasonable estimate for rocks with I_D values in the range of 96.5 to 98.5.

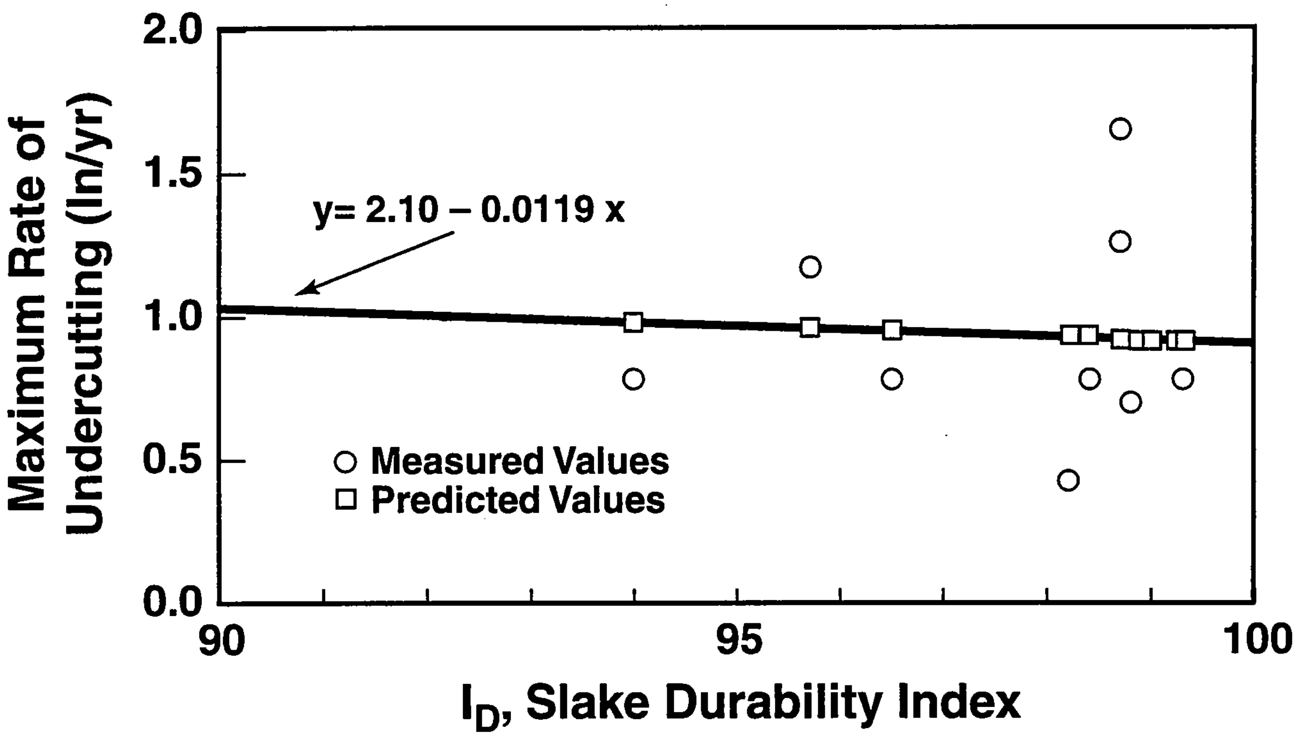

Figure 7. Maximum rate of undercutting, inches/yr., versus I_D, Slake Durability Index, for the Two Rock Cuts in Argillaceous Carbonates.

Conclusions

Rock cores were drilled for the Clear Creek study site providing boring logs for evaluation. Although recovery percentages were noted, the RQD was not determined. This is unfortunate as those data may have designated thinly bedded, less competent zones in the cores. Also, the log descriptions were not very detailed, so that changes in lithology and composition of the carbonate rocks were not described in any detail.

Measurements of the extent of weathering and formation of overhangs were accomplished at two sites. Slake durability values were also obtained on representative samples. These data along with measured rates of erosion made it possible to compare rock types with durability based on both rates of retreat and I_D values. The argillaceous carbonate rocks were found to be the most susceptible to erosion.

Two modes of slope retreat occur. The weaker, argillaceous-rich layers weather back producing overhangs. In the second mode, the thicker more competent rocks are undermined. Joints and pore pressure combine to isolate and then move blocks which tumble down the slope to the road level. These larger rocks are the ones more likely to cause vehicular accidents if they should reach the driving surface of the road.

It is proposed, based on the findings of this study that rock cores for cut slopes be logged in detail, providing information on the various types of carbonate and clastic rocks. Also, selected portions of the core should be used to perform slake durability tests. This will make it possible to predict at least to a limited degree the rate of erosion of the weaker layers. Knowledge of the stratigraphy can be used to estimate the likelihood of large, competent blocks of rock, falling and toppling from the slope.

Special slope designs can be used to reduce the effects of these slope processes. Shakoor (1995) suggests that benches be placed at the top of weaker beds, allowing the competent rock to set back to form the back wall. Considerable undercutting is required before the weaker rock works its way back to the cliff of stronger rock.

Other techniques are available to mitigate the deterioration of cut slopes in argillaceous-rich carbonate rocks (TRB, Special Publication 247, Landslides: Investigation and Mitigation, 1996; particularly Chapter 21 by Walkinshaw and Santi). Also the geotechnical manual by the Kentucky Department of Highways (1993) provides valuable information.

Acknowledgments

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Appendix A

Boring Logs for Project No. F-92(12)PE, F-92(18)

by ATEC & Associates, Indianapolis, 1969

108A

AMERICAN TESTING AND ENGINEERING CORPORATION

RECORD OF SOIL EXPLORATION

ATEC

With: Fink, Roberts & Petrie, Inc. — Boring #: 44 (Page1)

Project Name: F-92(12) P.E. SR 37 — Job #: E-67229

Location: 796+25 85'LT Monroe County, Indiana

SAMPLER

Datum: USC & GS	Hammer Wt. ___ Lbs.	Hole Diameter: 10"	Foreman: O. Cornelius
Ref. Elev.: 644.3 Ft.	Hammer Drop ___ In.	Rock Core Dia.: NXM	Inspector: R. Brinkman
Date Started: 5-14-69	Pipe Size ___ In.	Boring Method: HSA & RC & 2'AH	Date Completed: 5-15-69

SOIL DESCRIPTION Color, Moisture, Density, Plasticity, Size, Proportions	STRA. DEPTH	DEPTH SCALE	COND.	Blows/6"	NO.	TYPE	REC.	BORING & SAMPLING NOTES
SURFACE	0.0				1	2'AH	%	
Brown wet soft SILTY CLAY LOAM with trace of Organic Matter A-6 #10	2.0				2	2'AH		
Brown moist medium stiff SILTY CLAY with trace of Organic Matter A-7-6 #11		5			3	2'AH		
Red moist stiff CLAY A-7-6 #15	6.0		1		1	RC	100	
White coarse grain LIMESTONE with horizontal cracks		10	1		2	RC	100	
		15	1		3	RC	100	*Dry prior to Coring water introduced into hole for Coring
	25.0	20						
White medium to fine LIMESTONE with horizontal cracks and Clay Seams		25			4	RC	96	
		30						

SAMPLE CONDITIONS
D — DISINTEGRATED
I — INTACT
U — UNDISTURBED
L — LOST

SAMPLER TYPE
DS — DRIVEN SPLIT SPOON
PT — PRESSED SHELBY TUBE
CA — CONTINUOUS FLIGHT AUGER
RC — ROCK CORE

GROUND WATER DEPTH
AT COMPLETION 6.0* FT.
AFTER ___ HRS. ___ FT.
AFTER 24 HRS. 18.2* FT.

BORING METHOD
HSA — Hollow Stem Augers
CFA — Continuous Flight Augers
DC — Driving Casing
MD — Mud Drilling

STANDARD PENETRATION TEST—DRIVING 2" OD SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

AT-103A

AMERICAN TESTING AND ENGINEERING CORPORATION

RECORD OF SOIL EXPLORATION

ATEC

Contracted With: Fink, Roberts & Petrie, Inc. Boring #: 44 (Page 2)
Project Name: F-92(12) P.E. SR 37 Job #: E-67229
Location: 796+25 85'LT Monroe County, Indiana

SAMPLER

Datum: USC & GS | Hammer Wt. ____ Lbs. | Hole Diameter: 10" | Foreman: O. Cornelius
Surf. Elev.: 644.3 Ft. | Hammer Drop ____ In. | Rock Core Dia.: NXM | Inspector: R. Brinkman
Date Started: 5-14-69 | Pipe Size ____ In. | Boring Method: HSA & RC & 2'AH | Date Completed: 5-15-69

SOIL DESCRIPTION (Color, Moisture, Density, Plasticity, Size, Proportions)	STRA. DEPTH	DEPTH SCALE	SAMPLE: COND	Blows/6"	NO.	TYPE	REC. %	BORING & SAMPLING NOTES
White medium to fine LIMESTONE with horizontal cracks and clay seams	32.2		I		4	RC	96	
White medium to fine LIMESTONE with horizontal cracks		35	I		5	RC	97	
		40	I		6	RC	100	
		45						
		50	I		7	RC	100	*Dry prior to Coring. Water introduced into hole for Coring.
		55						
		60						
	62.0							

SAMPLE CONDITIONS
D — DISINTEGRATED
I — INTACT
U — UNDISTURBED
L — LOST

SAMPLER TYPE
DS — DRIVEN SPLIT SPOON
PT — PRESSED SHELBY TUBE
CA — CONTINUOUS FLIGHT AUGER
RC — ROCK CORE

GROUND WATER DEPTH
AT COMPLETION 6.0* FT.
AFTER ____ HRS. ____ FT.
AFTER 24 HRS. 18.2* FT.

BORING METHOD
HSA — Hollow Stem Augers
CFA — Continuous Flight Augers
DC — Driving Casing
MD — Mud Drilling

STANDARD PENETRATION TEST—DRIVING 2" OD SAMPLER 1' WITH 140# HAMMER FALLING 30": COUNT MADE AT 6" INTERVALS

AT-103A

AMERICAN TESTING AND ENGINEERING CORPORATION

RECORD OF SOIL EXPLORATION

ATEC

Connected With Fink, Roberts & Petrie, Inc.

Project Name F-92 (12) P.E. SR 37 — Boring # 44 (page 3)

Location 796+25 85'LT Monroe County, Indiana — Job # E-67229

SAMPLER

Datum USC & GS — Hammer Wt. ____ Lbs. — Hole Diameter 10" — Foreman O. Cornelius

Surf. Elev. 644.3 Ft. — Hammer Drop ____ In. — Rock Core Dia. NXM — Inspector R. Brinkman

Date Started 5-14-69 — Pipe Size ____ In. — Boring Method HSA & RC & 2'AH — Date Completed 5-15-69

SOIL DESCRIPTION Color, Moisture, Density, Plasticity, Size, Proportions	STRA. DEPTH	DEPTH SCALE	SAMPLE COND.	Blows/6"	NO.	TYPE	REC. %	BORING & SAMPLING NOTES
		62.5						
Light gray fine LIMESTONE with horizontal cracks		65	1		8	RC	100	
		70						
		75	1		9	RC	100	
	82.6	80						
Dark gray moderately hard SILTSTONE	88.0	85	1		10	RC	94	
Bottom of test boring 88.0'								*Dry prior to Coring. Water introduced into hole for Coring.

SAMPLE CONDITIONS
- DISINTEGRATED
- INTACT
- UNDISTURBED
- LOST

SAMPLER TYPE
- DS — DRIVEN SPLIT SPOON
- PT — PRESSED SHELBY TUBE
- CA — CONTINUOUS FLIGHT AUGER
- RC — ROCK CORE

GROUND WATER DEPTH
- AT COMPLETION 6.0* FT.
- AFTER ____ HRS. ____ FT.
- AFTER 24 HRS. 18.2* FT.

BORING METHOD
- HSA—Hollow Stem Augers
- CFA—Continuous Flight Augers
- DC —Driving Casing
- MD —Mud Drilling

STANDARD PENETRATION TEST—DRIVING 2" OD SAMPLER 1' WITH 140# HAMMER FALLING 30": COUNT MADE AT 6" INTERVALS

103A

AMERICAN TESTING AND ENGINEERING CORPORATION

RECORD OF SOIL EXPLORATION

ATEC

Contracted With: Fink, Roberts, & Petrie, Inc. — Boring # 46 (Page 1)

Project Name: F-92 (12) P.E. SR 37 — Job # E-67229

Location: 811+00 85'LT Monroe County, Indiana

SAMPLER

Datum: USC & GS	Hammer Wt. ____ Lbs.	Hole Diameter: 10"	Foreman: O. Cornelius
Surf. Elev.: 690.5 Ft.	Hammer Drop ____ In.	Rock Core Dia.: NXM	Inspector: R. Brinkman
Date Started: 5-16-69	Pipe Size ____ In.	Boring Method: HSA & RC & 2'AH	Date Completed: 5-19-69

SOIL DESCRIPTION (Color, Moisture, Density, Plasticity, Size, Proportions)	STRA. DEPTH	DEPTH SCALE	COND.	Blows/6"	NO.	TYPE	REC.	BORING & SAMPLING NOTES
SURFACE	0.0				1	2'AH	%	
Brown moist medium stiff SILTY CLAY LOAM with trace of Organic Matter A-6 #10	2.0				2	2'AH		
Brown moist medium stiff SILTY CLAY A-7-6 #11	5.0	5			3	2'AH		
Red moist stiff CLAY A-7-6 #15	6.2							
White medium grained LIMESTONE	9.8		I		1	RC	93	
White medium grained LIMESTONE with red Clay Seams	15.5	10	I		2	RC	50	
			I		3	RC	60	
		15	I		4	RC	50	
White LIMESTONE	15.7				5	RC	6	
Cavity with some Clay	29.0	20			6	RC	0	*Dry prior to Coring. Water introduced into hole for Coring. Filled at completion
		25			7	RC	18	
White Limestone	29.7	30						
Cavity with some Clay								

SAMPLE CONDITIONS	SAMPLER TYPE	GROUND WATER DEPTH	BORING METHOD
D — DISINTEGRATED	DS — DRIVEN SPLIT SPOON	AT COMPLETION 31.0 * FT.	HSA — Hollow Stem Augers
I — INTACT	PT — PRESSED SHELBY TUBE	AFTER ____ HRS. ____ FT.	CFA — Continuous Flight Augers
U — UNDISTURBED	CA — CONTINUOUS FLIGHT AUGER	AFTER 24 HRS. ____ FT.	DC — Driving Casing
L — LOST	RC — ROCK CORE		MD — Mud Drilling

STANDARD PENETRATION TEST—DRIVING 2" OD SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

AT-103A

AMERICAN TESTING AND ENGINEERING CORPORATION

RECORD OF SOIL EXPLORATION

ATEC

Contracted With: Fink, Roberts & Petrie, Inc. — Boring #46 (Page 2)

Project Name: F-92 (12) P.E. SR 37 — Job #: E-67229

Location: 811+00 85'LT Monroe County, Indiana

SAMPLER

Datum: USC & GS	Hammer Wt. ___ Lbs.	Hole Diameter: 10"	Foreman: O. Cornelius
Surf. Elev. 690.5 Ft.	Hammer Drop ___ In.	Rock Core Dia. NXM	Inspector: R. Brinkman
Date Started: 5-16-69	Pipe Size ___ In.	Boring Method: HSA & RC & 2'AH	Date Completed: 5-19-69

	SOIL DESCRIPTION Color, Moisture, Density, Plasticity, Size, Proportions	STRA. DEPTH	DEPTH SCALE	SAMPLE COND.	SAMPLE Blows/6"	SAMPLE NO.	SAMPLE TYPE	SAMPLE REC. %	BORING & SAMPLING NOTES
	Cavity with some Clay	36.0	35			8	RC	0	
	Light gray medium LIMESTONE with vertical and horizontal cracks		40	1		9	RC	66	
		44.7	45	1		10	RC	95	
	Bottom of test boring 44.7'								*Dry prior to Coring. Water introduced into hole for Coring. Filled at completion

SAMPLE CONDITIONS
- D — DISINTEGRATED
- I — INTACT
- U — UNDISTURBED
- L — LOST

SAMPLER TYPE
- DS — DRIVEN SPLIT SPOON
- PT — PRESSED SHELBY TUBE
- CA — CONTINUOUS FLIGHT AUGER
- RC — ROCK CORE

GROUND WATER DEPTH
- AT COMPLETION 31.0 * FT.
- AFTER ___ HRS. ___ FT.
- AFTER 24 HRS. ___ FT.

BORING METHOD
- HSA — Hollow Stem Augers
- CFA — Continuous Flight Augers
- DC — Driving Casing
- MD — Mud Drilling

STANDARD PENETRATION TEST—DRIVING 2" OD SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

FLOWABLE FILL USING WASTE PRODUCTS

C. W. "Bill" Lovell
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ABSTRACT

Flowable fill is an excellent replacement for compacted soil for locations in confined spaces, like around pipes and conduits and in corners. The flowable fill parameters of greatest importance are: (1) flow or spread so that gravity flow can cause the material to penetrate into the smaller spaces; and (2) an early hardening, sufficient for walking upon and covering up in a very short time. If the application is such that re-excavation is likely, the material should have a low final or ultimate strength. Such flowable fills are referred to as Controlled Low Strength Materials (CLSMs), and are the most used varieties.

The constituents of flowable fill are normally cement, water, fly ash, and sand. The fly ash may be Class F, or low-market value Class C, and the sand could be a coal combustion bottom ash or a waste foundry sand. Thus the flowable fill may be comprised almost totally of non marketable materials.

The paper describes how flowable fill mixes using waste products are designed, and how flow, early hardening, and low ultimate strength may be measured and predicted.

INTRODUCTION

Flowable fill is a grout-like material comprised of cement, water, fly ash, and sand. On occasion, other materials may be used as additives. The principal characteristics required are: flow by gravity into small and confined spaces, and an early hardening sufficient for walkability and being covered. In addition, for applications such as backfill for utility cuts across roads and streets, where reexcavation is a distinct possibility, a low ultimate strength is desired. Where the strength is held below about 150 psi, the material is a Controlled Low Strength Material (CLSM). Such flowable fills can be readily excavated by hand or with a backhoe.

While flowable fill has been used for about two decades, its applications seem to be increasing in the past few years. The recent past has also seen increased emphasis on the identification of useful applications of industrial byproducts which have little marketing potential. Several of the latter can be used in flowable fill, viz., spent foundry sand, coal combustion bottom ash, Class F fly ash and marginal quality Class C fly ash. In fact, except for a small quantity of cement, the flowable fill may be comprised totally of non-marketable solids.

The Indiana DOT has a program of identification and use of industrial byproducts, such as foundry sand and coal combustion ashes, in highway construction. As a part of this program both Class F fly ashes and spent foundry sands from greensand ferrous castings have been extensively studied in the laboratories of Purdue University. These studies are reported in total to date by (Javed & Lovell, 1994; Bhat & Lovell, 1996; Bastian & Alleman, 1996). This paper summarizes some of the principal findings of the above studies, as they apply to the use of spent foundry sands from greensand castings and Class F fly ash in flowable fill.

PARAMETERS OF MIXES

The three parameters of greatest interest are: (1) flow; (2) hardening rate; and (3) limiting ultimate strength.

Although flowable fill generally contains cement, it is erroneous to call it "concrete". It is more proper to consider it as a kind of "stabilized soil". The consistency of concrete is determined by a slump test, where wetter mixes slump more. The counterpart with flowable fill is the flow test where spread is measured. The wetter mixes demonstrate greater spread. See Figure 1. While the curing of concrete produces progressiving setting of the mix, it is more proper to characterize flowable fill as hardening. The critical degree of hardening is when the fill can be walked upon. This corresponds to an ability to produce a penetration resistance of about 65 psi. At this time, the bleeding water has escaped the mix. Obviously, the bleeding process is facilitated by good drainage conditions adjacent to the trench, and thus the time to achieve walkability is site dependent.

The final major parameter is a limiting (usually ≤ 150 psi) ultimate strength, to facilitate excavation by backhoe or by hand. Since strength values for concrete are measured in the thousands of psi, the contrast with flowable fill is again quite obvious. The stress-strain-strength characteristics of flowable fill are indeed quite similar to the compacted soil backfill which it typically replaces.

Flow Characteristics

Flowability of fresh flowable fill mixture is measured by the modified flow test (ACI Committee 229, 1994) using a 7.6 cm x 15.2 cm (3 in. x 6 in.) open ended cylinder as shown in Figure 1.

The principal factors influencing flow are the weights of solids (cement, ash, sand) and the weight of water. When are set forth in ratios for adequate spread (9 in.), experimental curves of the sort idealized in Figure 2 are produced. All proportions of a given set of cement, ash and sand materials require various amounts of water to produce the 9 in. spread. A certain amount of water, and fly ash, is required to produce flow (Point A), and as fly ash is added, less water is required (path B). At Point C a minimum amount of water is required to produce the flow. Adding fly ash increases the water demand (path D), and when fly ash has totally replaced sand, the water requirement is at a maximum.

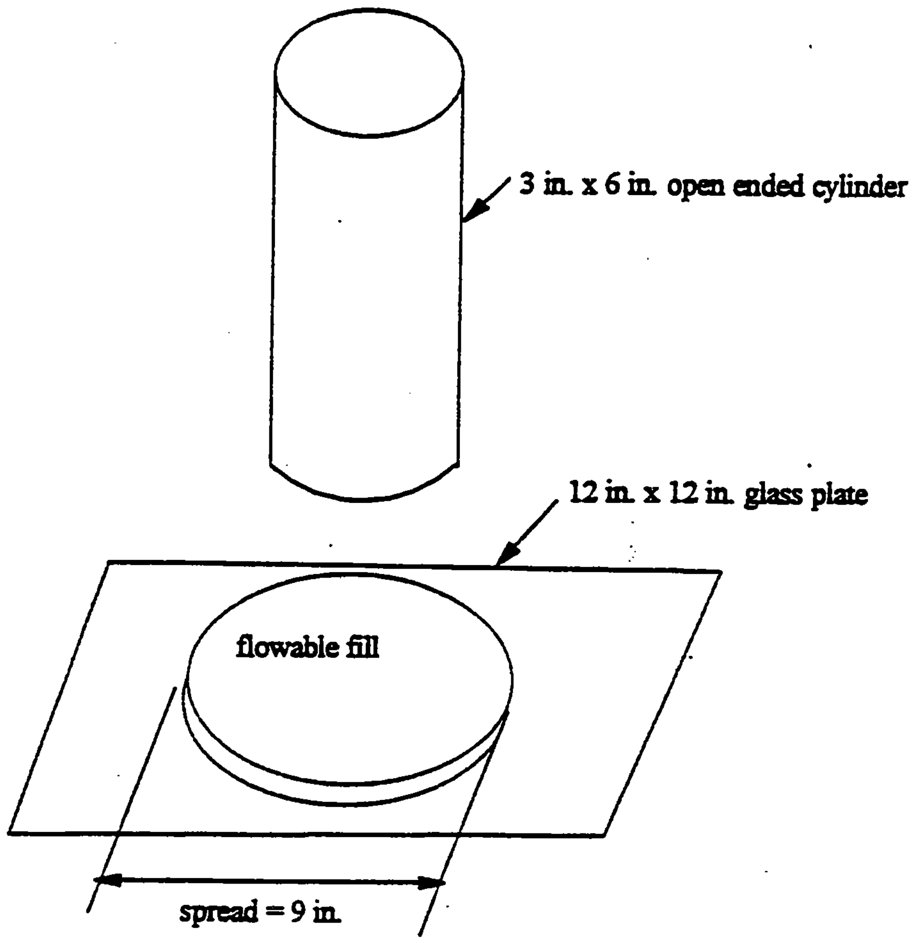

Figure 1 Modified Flow Test

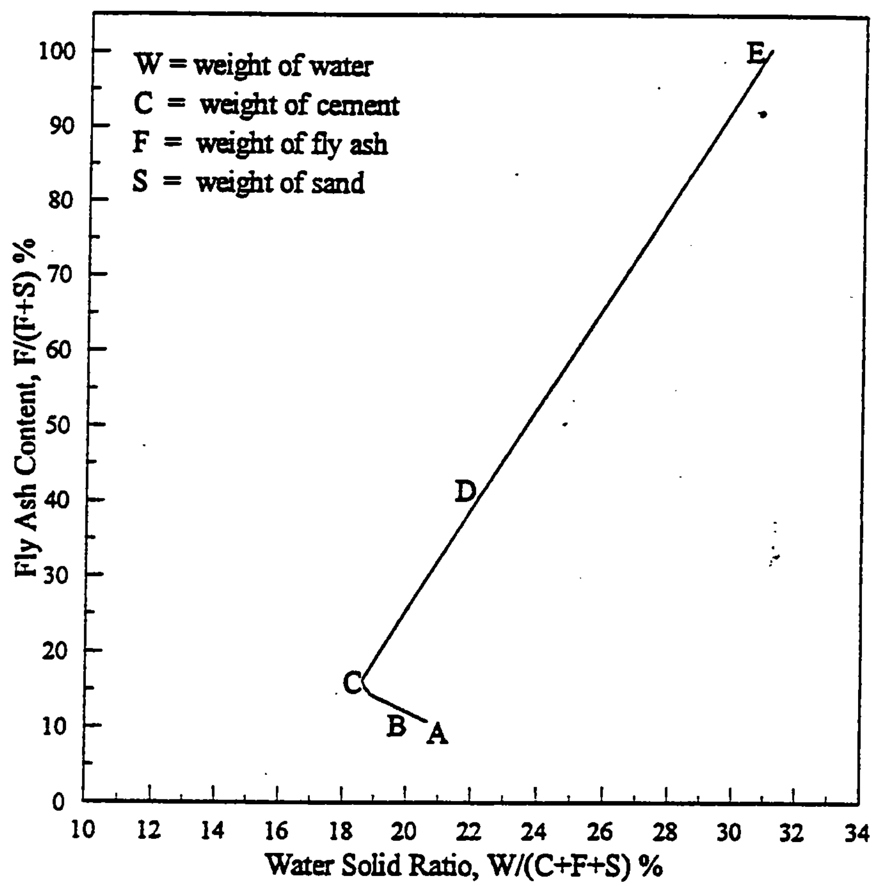

Figure 2 An Idealized Flow Curve

Thus, contrary to popular belief, fly ash serves as a "lubricant" only in small quantities. At high fly ash contents, viscous forces dominate the flow, and the fly ash particles tend to flocculate, thus losing the advantage due to the spherical fly ash particle shape. The point of minimum water requirement would seem to constitute a good target for mix proportions. It should produce the highest unit weight (least porous) mix, as well as minimizing bleeding and cement content.

Hardening Behavior

For the applications being emphasized in this paper, it is highly desired to minimize the time from placement to sufficient hardness for covering. The hardening rate will vary not only with the mix design, but also with the ability of bleeding water to drain into adjacent soil. The latter point was well demonstrated in the laboratory by monitoring hardening rate with and without geotextile drains. Well drained site conditions will produce significantly faster hardening rates.

Significant hardening to allow walking upon with only small imprints is termed a "walkable" condition. This condition correlates with penetration resistance as measured in the ASTM C403 test. The walkability value corresponds to a penetration value of 448 kPa (65 psi). Soil pocket penetrometer values were correlated, as shown in Figure 3. A penetration value of 65 psi corresponds to an unconfined compressive strength value of 10.53 psi and an ultimate bearing capacity (q_f) of 3.71 q_U = 39.1 psi.

Strength and Mix Design

This value may be considered to be comprised of two components: cementation and frictional. The factor which most influences strength is the water/cement ratio (W/C). This is well illustrated in Figure 4.

The limiting strength is usually taken at 150 psi (1035 kPa), and the 28 day strength should be somewhat less than this to accommodate additional strength gain. Curves of the type shown in Figure 2 (identifying the point of minimum water requirement) and Figure 4 (showing the water/cement ratios needed to appropriately limit 28-day strength) are the basis for a rational mix design procedure. This technique is described in detail in Bhat and Lovell (1996). A typical calculation using spent foundry sand and Class F fly ash would produce the following proportions in kg/m^3

Cement	55	:	3.1
Water	470	:	26.8
Fly Ash	257	:	14.6
Sand	974	:	55.5
Total	1756	:	100%

This mix would be tested experimentally for flow, hardening rate and 28 day strength to validate the predictions.

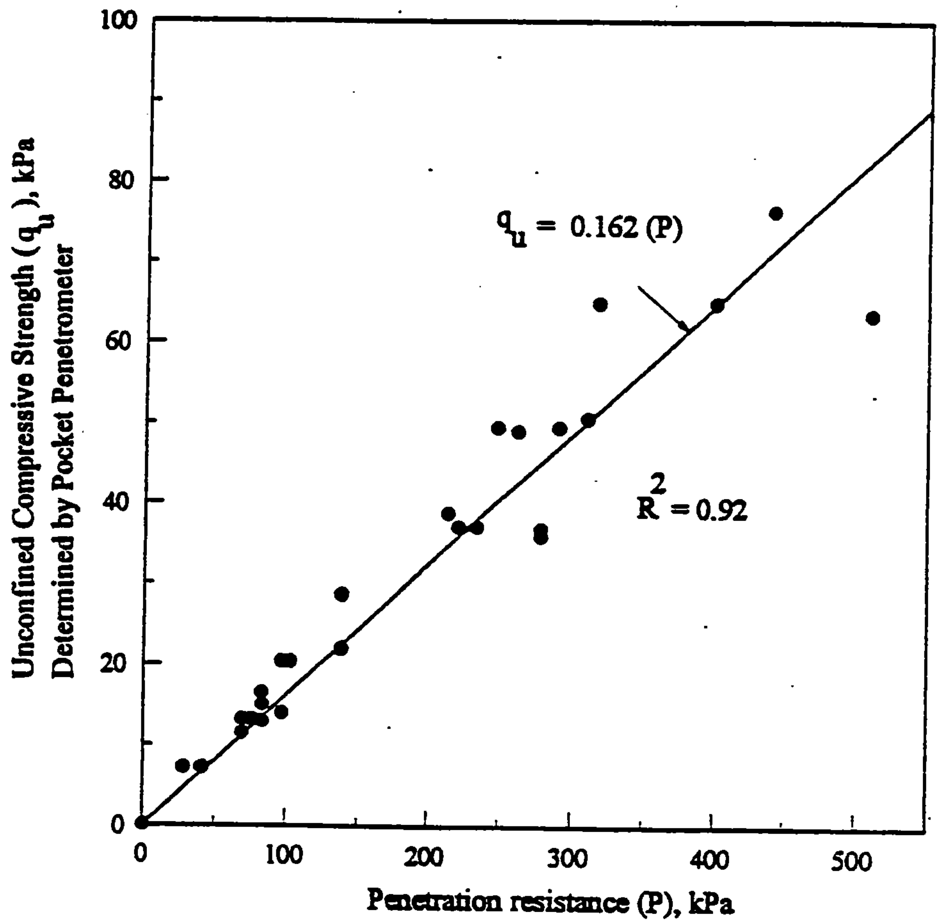

Figure 3 Penetration Resistance Vs Estimated Unconfined Compressive Strength

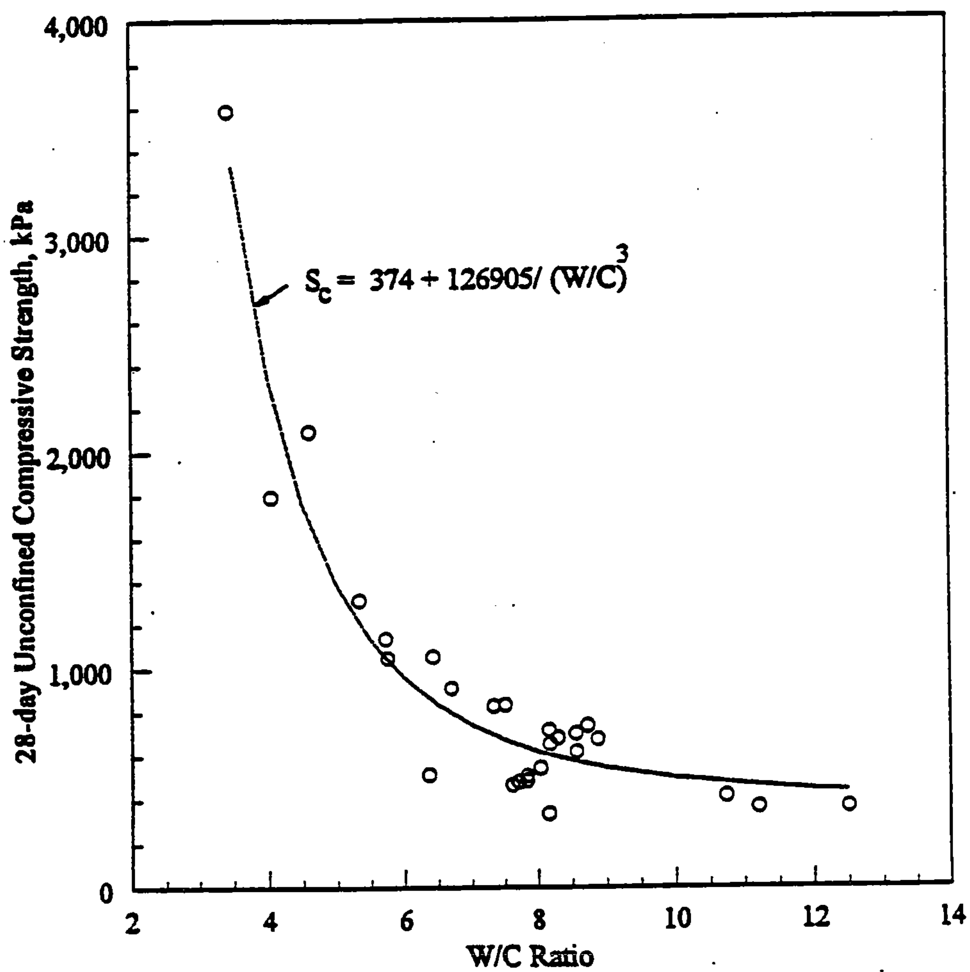

Figure 4 W/C Ratio Vs Unconfined Compressive Strength

Accelerated Strength Determinations

When dealing with new materials, trials are necessary, and a source of considerable delay is the 28-day strength testing. Accordingly, there is merit in developing a prediction from accelerated testing. ASTM Standard C684 would be used for concrete. However, these techniques cannot be directly applied to flowable fill, since these materials contain very little cement. Through experimentation, a technique was developed which involved 3 days of curing at 35C, followed by 24 hours of curing at room temperature which produced 70-75% of the 28 day strength. The correlation is demonstrated in Figure 5. See Bhat and Lovell (1996) for further details. More study is needed to produce a highly reliable prediction.

WASTE MATERIALS

Referring back to the previous mix, some 70% of it was comprised of non-marketable (waste) material, viz., spent foundry sand and Class F fly ash. Thus, flowable fill is an important alternative to disposal of these materials. It is also believed that coal combustion bottom ash is usable in these mixes, as well as co-disposed mixtures of bottom and fly ash...and perhaps others.

Environmental evaluation of these materials is an essential requirement prior to use. The detection of amounts of specific materials is undertaken with the Toxicity Characteristic Leaching Procedure (TCLP). A non-material-specific biological assessment (Microtox™ bioassay) has been used to provide an additional "layer" of evidence. See Bastian and Alleman, 1996). Spent greensands from ferrous castings are found to nearly always pass the above tests.

SUMMARY AND CONCLUSIONS

Flowable fill is an appropriate substitute for compacted soil in confined spaces such as that around pipes and conduits in utility cuts. In addition to flowability and an early hardening, the Controlled Low Strength Material (CLSM) variety can be excavated by hand.

Each of the above feature parameters may be measured by simple tests, and the cement, water, ash and sand components can be proportioned to achieve a suitable mix. This paper suggests a rational method for mix design.

While waste materials (ash and sand) must be environmentally suitable, both TCLP and bioassay tests indicate that most spent greensands from ferrous castings are likely to be satisfactory. These materials also have suitable physical/mechanical properties. Diversion of these sands from disposal to useful applications is much desired.

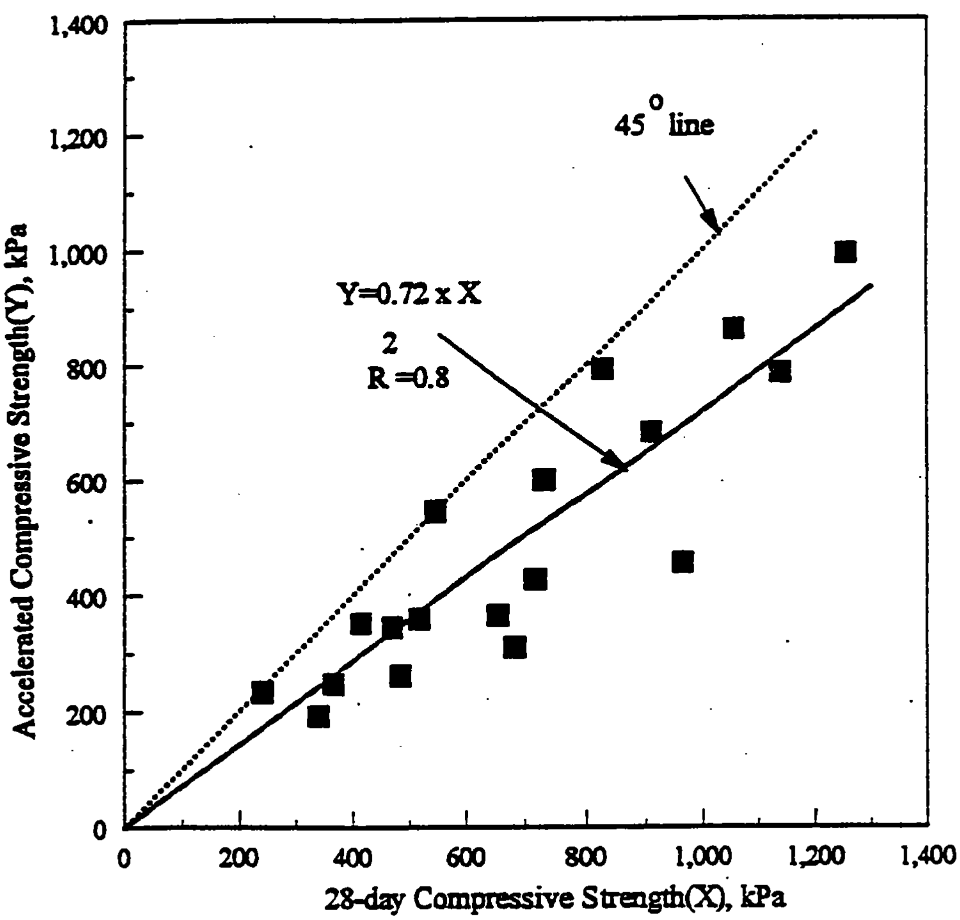

Figure 5 Accelerated Strength Test (Age of Samples: 3 days, Temperature: 35°C, Duration of Curing: 24 hours)

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SUBSIDENCE DEFINITION AND EFFECTS ON SURFACE CONSTRUCTION

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Abstract

Subsidence is the lowering of the ground surface by either naturally occurring Geological Processes or by man induced factors. The various subsidence types, Trough and Chimney - Plug manifestations are defined and evaluated to provide an acceptable risk approach for construction. In addition, case studies are presented to define the extent and magnitude of surface subsidence. The Angle of Draw, Angle of Break, and Subsidence Profiles define potential problem areas. The **Time Effect** is described and **"Rule - of - Thumb"** guides are provided for various underground openings and rock types upon the surface elevation.

Introduction:

The construction of surface facilities over areas which have subsided, are in the process of subsiding or in the future may subside, has been based on the Engineers experience of similar situations. An elucidation of the physical principles can give the designer and constructor better insight into the strategies of resolving the situation and mitigating long term effect.

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The lowering of the ground surface, subsidence, is a wide spread situation caused by many processes both natural and man made. The tectonic processes are major players on (and in) the earth. We only need to see the down thrown side of a fault to realize the magnitude of these settings. Constructing long term "stable" facilities is difficult as evidenced by the nuclear power plant site processes and by the selection process for radioactive waste repositories. Few people realized the site selection processes would stretch on for (many) decades, caused in the main part by tectonic factors and the NIMBY (not in my backyard) syndrome.

In General, most people think that lowering the earth's surface is caused by man induced openings beneath the earth's surface. The reasons for this are based from mining located near urban areas, where surface areas for various cultural activities are at a premium and people want to get many facilities close together.

The most wide spread lowering of the ground surface, after tectonism, is caused by the withdrawal of fluids. Water and petroleum products are the best known fluids but the dissolving of salt minerals (and carbonate rocks) cause significant surface lowering.

In the following sections, a discussion of the various physical causes are given and some possible remediation activities are presented.

Historical Development:

Extractive processes which remove rock or fluids from the ground cause subsidence or lowering of the surface. In general, any extraction will cause a lowering of the surface in an amount equal to the removed materials given an adequate time. The engineering - geological processes causing lowering and equilibrium will not stop until a "lowest" energy state is present(i.e. gravity forces have become minimized!). The popular concept that it is possible to extract rock (and fluids) and see no surface effect is not true.

An early statement of this position called Fayol's Law (Henri Fayol

(1841-1925)) states that if an opening is small enough or deep enough no effect is "observed" at the surface. Fayol, A French Mining Engineer, working in coal mines made an astute observation based on the surveying equipment and techniques of his time and the duration of the observations.

The earliest mining extracted minerals from the surface. As surface supplies were used up, the users followed the minerals underground, creating underground mining (and the resultant surface lowering). Technological advances, in the last hundred years, have allowed a variety of mining extractive methods to develop which cause different amounts of subsidence. A brief description of methods illustrate the reasons for different conditions.

The geometry of the ore (and volumes extracted) determines the type of mining or extraction method. The amount removed is from near zero in some type of vein extraction (as in narrow, planar gold/silver veins) or in limited pumping of fluids. While near complete extraction occurs in some long wall type mining methods or in solution mining (dissolving of salt) or large volume fluid extraction processes.

Because mining occurred in populated areas, predictive efforts to determine the amount of surface lowering are mostly concerned with prevention of damage during and after ground surface lowering occurs. A better approach is to analyze the problem and alleviate the underlying conditions. Probably the most publicized underground situations are the following: the different underground activities in populated areas, such as coal mining; and the formation of sinkholes as illustrations of karst features endemic in limestone areas. The former class of events illustrate the man induced effects on the surface, and the latter in many cases naturally occurring effects.

Interrelationship of factors:

Geologic factors such as changes in rock and soil structures, composition of rocks and soils, and fluid removal result in subsidence. In any specific locality, prediction of the amount of lowering of the surface (and strain history) is reasonable from historical studies as Waltham (1989) states:

> "The worldwide scale of subsidence damage through groundwater withdrawal reached a peak between 1950 and 1970, at a time of unprecedented urban growth and industrialization. An extensive literature on engineering aspects has since accumulated, including three UNESCO - Association of Hydrological Sciences as Publications nos. 88-89, 121 and 151. It is significant that the first conference (at Tokyo in 1969) was dominated by cased histories of the effects of water-table declines, while the third (at Venice in 1984) was more concerned with groundwater modelling and subsidence monitoring - many of its papers turned to other styles of subsidence. **Within that period, the mechanism of subsidence due to fluid withdrawal were largely understood, and the review effort edited by Poland (1984) is therefore a valuable and complete statement of the subject."**
>
> (Bold emphasis by Authors)

For sedimentary rocks, the actual sedimentation process and geologic framework radically effects the subsidence potential. Of particular note are the Cretaceous Age Black Shales (Pierre Shale and equivalent rocks in the Western U.S.) and the many Tertiary age basin deposits incorporating volcanic materials. The volcanic eruptive deposits often contain a high percentage of (montmorillonite) clays. The response to dewatering and subsidence easily correlate to the clay type. A three-fold increase as seen from the little compressible kaolinite through the illites and to the highly compressible montmorillonites. The montmorillonites are the common decomposition products of volcanic activity. A combination of the effects is illustrated in Table 1 (after Waltham, 1989). In examining Table I, the following features can be observed: As the montomorillonite content increases, the subsidence increases; as the layer thickness increases, so does the total subsidence; and the specific subsidence (a dimentionless parameter normalized to compare equal thicknesses) better illustrates the effect of the clay type.

The fluid withdrawal-subsidence models are body loaded, the loading is caused by gravity action on the rocks. An additional load, the roadway weight, weight of vehicles, traffic vibrations or other external effects will cause additional subsidence, until the resistive forces balance or exceed the applied load.

Table 1 Comparison of geological, hydrological and subsidence parameters at six sites, with data taken from various published sources referred to in the text. Subsidence at Santa Clara is quoted as the predicted ultimate value which has not been realized; all other subsidence figures are those recorded, which are not ultimate.

Site	Clay thickness m	Head decline m	Sub. m	Specific sub. m/m	Comp. $x10^{-4}$m/m/m	Mont. Content %	Age
London	60	100	0.35	.0035	0.6	0	Eocene
Savannah	50	48	0.19	.004	0.8	60	Miocene
Venice	130	9	0.12	.013	1.0	10	Recent
Houston	150	90	2.3	.025	2.1	50	Recent
Santa Clara	145	49	5.3	.11	7.4	70	Recent
Mexico	50	55	9.0	.16	32.0	80	Recent

Subsidence - Definition, concepts and reasons:

A general definition of subsidence (AGI - Glossary) is

> "**subsidence** (a) A local mass movement that involves principally the gradual downward settling or sinking of the solid earths surface with little or no horizontal motion and that does not occur along a free surface (not a landslide or failure of a slope). The movement is not restricted in rate, magnitude, or area involved. Subsidence may be due to natural geologic process such as solution, erosion, oxidation, thawing, lateral flow, or compaction of subsurface materials, earthquakes, slope crustal warping, and vulcanism (withdrawal of fluid lava beneath a solid crust); or mans activity such as removal of subsurface solids, liquids, or gasses and wetting of some types of moisture-deficient loose or porous deposits. See also: cauldron subsidence. Syn: land subsidence; bottom subsidence. (b) parts, such as the formation of a rift valley or the lowering of a coast due to tectonic movements.---Syn: sinking."

This leaves a dilemma in the engineering approach to solving the effect or alleviating the original problem. A reasonable design methodology for Rock Engineering according to Bieniawski (1992) ". . is to provide a clear and concise narrative statement (of the problem) followed by deposition of performance objectives and the ensuing design issues." In subsidence the problems are known but corrective efforts are primarily used to patch up symptoms of the basic mechanisms. Thus major remedial efforts are used to "buy time" not in attacking or solving the major subsidence causes.

Subsidence is caused by the removal of rock (or fluids in rock pore space) being used as support material. In principle, the rock (or more correctly the rock mass) is a complex material consisting of a solid part (the Rock Substance, i.e. the solid blocks or in the case of soils the granular materials) and the fractures, joints, breaks, etc. that separate the blocks (the analogous situation is the pore space in soils). All range of behavioral response are caused by the interaction of Rock Substance vs Fractures, Breaks, Joints, etc.

If the rock is massive and behaves in a mostly elastic way, a situation as shown by Morrison (1970) in Figure 1 develops.

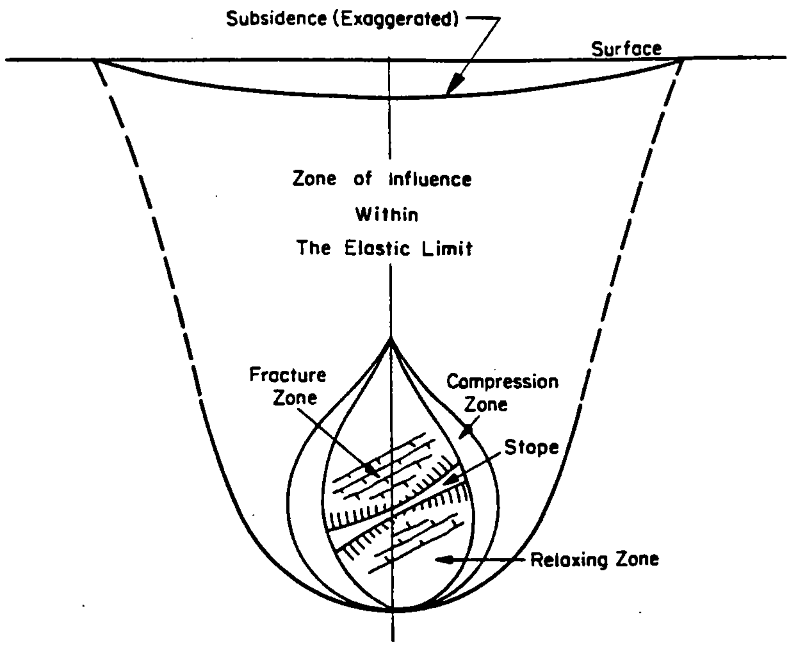

Figure 1. Elastic Subsidence Response (after Morrison)

In many cases this type of response, in the elastic range, is so small that little or no attention is needed. As a matter of fact, prediction by elastic modeling methods, such as the Finite Element Method gives good agreement with surface movements using laboratory measurements for the Elastic Modulus and Poisson's Ratio. In general, the predictive efforts from Finite Element Models using laboratory measurement for the Elastic Modulus and Poisson's Ratio are less than field measurements. Radical reductions of the rock substance Elastic Modulus are needed to match the field displacements (Studebaker, 1977). This is shown for back analysis of surface mine rock slopes by the comparison of elastic model study displacements to field displacements.

A greater extraction of rock (>15% to near 80%) causes different behavior on the surface usually showing a "piping" type break through to the surface, and a subsequent flattening of the sides of the crater until a stable shape is reached. Crane (1931) indicates this in Figure 2.

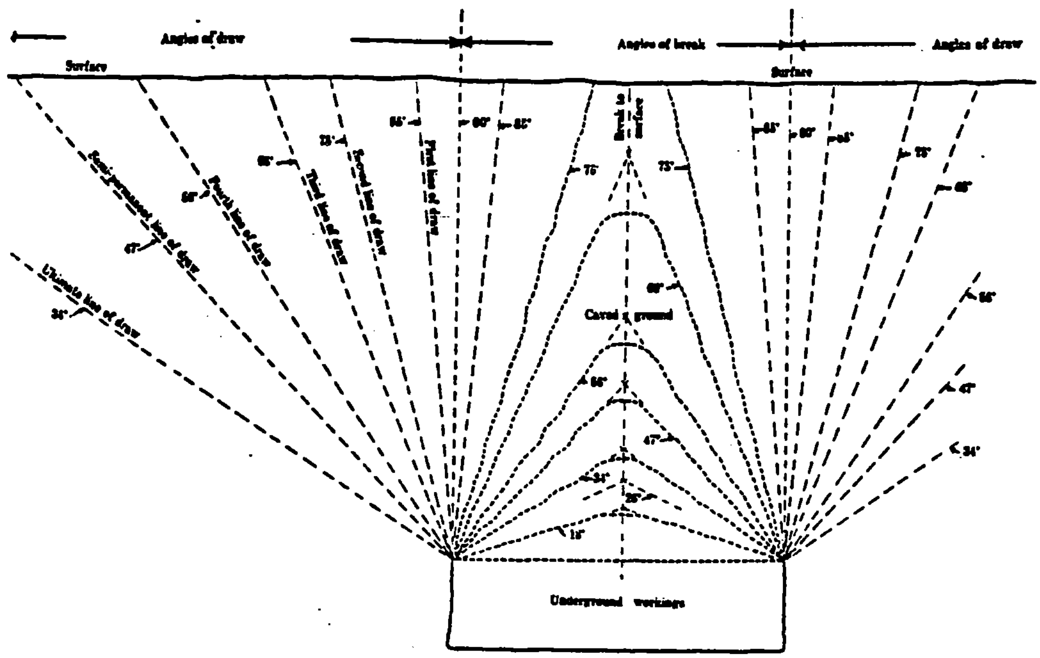

Figure 2. Progressive decrease in the angle of draw in underground workings. (modified from Crane)

Crane based this diagram on observation, and the concept is as valid in 1996 as when the observation and angles were measured. Note the piping forms by a spalling or raveling from the top of the opening (back or roof in mining terms) and continues upward caused by the tensile failure of the rock. Rock tensile strength are much weaker than compressive strengths! Once the breakthrough occurs (the pipe), rock will continue to spall and eventually go to about a 34° dip for this geologic setting. The actual final elastic response will extend beyond Crane's ultimate line of draw.

A third major case or end member of surface response is represented by extractions approaching 100% in a layer or zone. In mining a good example is a long wall coal mine panel. In general the surface response is depicted as a general lowering of the ground surface which is unaffected by the boundary or side effects. The National Coal Board (NCB) in England in their publication "Subsidence Engineers Handbook" (1975) discuss this situation in detail for the coal measure rock of the United Kingdom. Figure 3 indicates the general terminology for the NCB evaluations. It should be noted, the general composition of the overlying rock will change the details of the rock behavior but not the underlying principles (see Abel and Lee, 1980).

For near 100% extraction, three cases are present: pre-mining; post-mining; and the transition between. The pre and post-mining have almost no concern for construction and structures on the surface. The geologic nature of the "coal measure" rocks has a quick response to mining and once the mining is completed (or passes under) the area, the ground surface is quickly stabilized. The difficult surface facility responses are during the transition period. The surface goes through a period of compression and then a period of stretching (or tension) during the time when the surface is being lowered. The strains imposed are indicated in Figure 4 from the NCB. Considerable information is available to predict the effect on various types of surface building and facilities and it is possible, within reasonable limits to quantify the strain history end results.

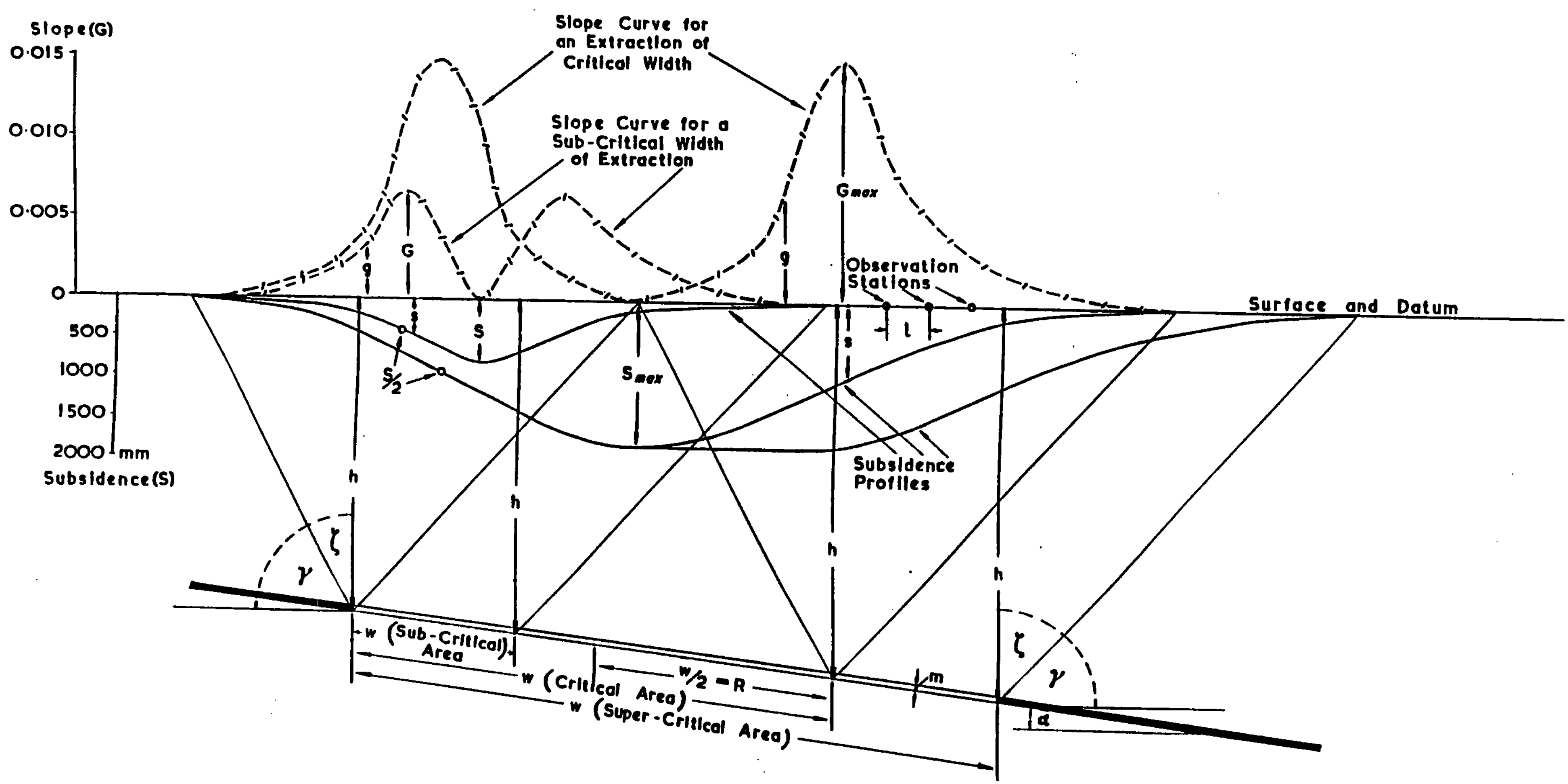

Figure 3. Typical section through workings illustrating standard symbols for subsidence and slope (after NCB).

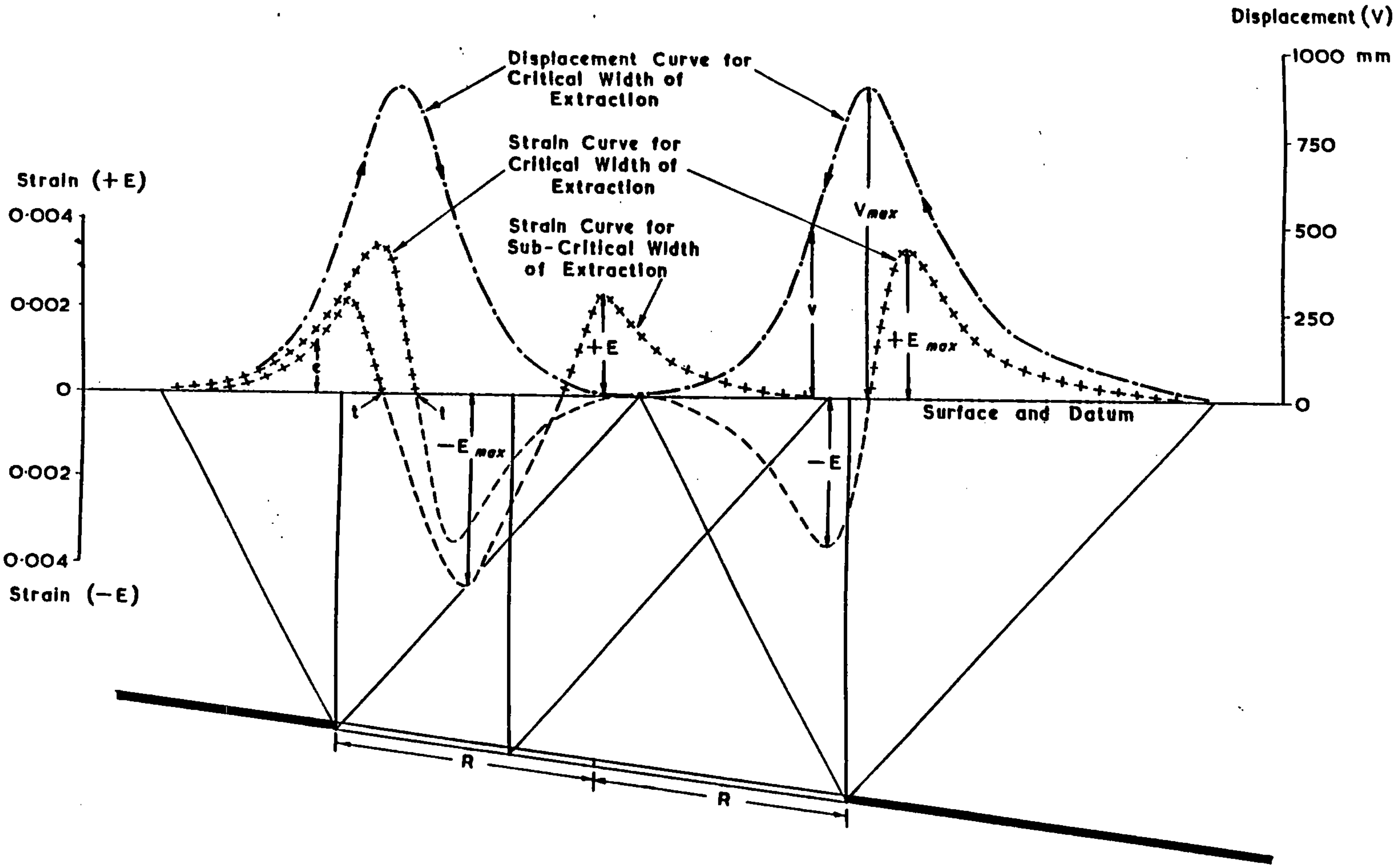

Figure 4. Typical section though workings illustrating standard symbols for strain and displacements (after NCB).

Collapsing Soils

A couple of examples along Interstate 90, in Montana, are useful to show the effect of collapsing soils (probably caused by the montmorilonite type clays). The examples are west of Big Timber between mile posts 392 - 394 (the DeHart exit) and the second location is between Whitehall and Cardwell, mile posts 250 to 256.

DeHart Exit

A series of interbedded sandstones, siltstones and shales lie under the I-90 alignment. The subsidence occurs as dips, which formed soon after the interstate completion in1971. The Yellowstone Valley widens above and below this area. The interbedded sequence of rocks are folded in the area of the dips. A syncline occurs in the thinner layered beds which appear to correlate with the road subsidence. Interestingly, the west bound lanes (which are lower) show greater subsidence development than the east bound lanes. The old highway (now a frontage road on the north) also have less subsidence development.(See Fig. 5A and B).

The dips appear to be caused by collapsing type soils but are minor in scope and have been corrected over the 25 year interval by maintenance techniques. The layers of sandstones appear massive east and west of the subsiding areas and the thinner bedded layers (and attendant folding) appear to control the subsidence. It is speculated the thinner layers contain more clay.

Whitehall to Cardwell

This section of I-90 was the subject of a discussion by Terry Yarger (MDOT) during the 1986 Highway Geology Symposium field trip. A dynamic compaction technique was used to densify the underling materials. The underlying Tertiary Basin Sediments are typical of the Western U.S. deposits, caused by wide spread vulcanism of that time. A combination of depositional conditions (loose packing of granular and silty materials caused by rapid deposition and a deltaic

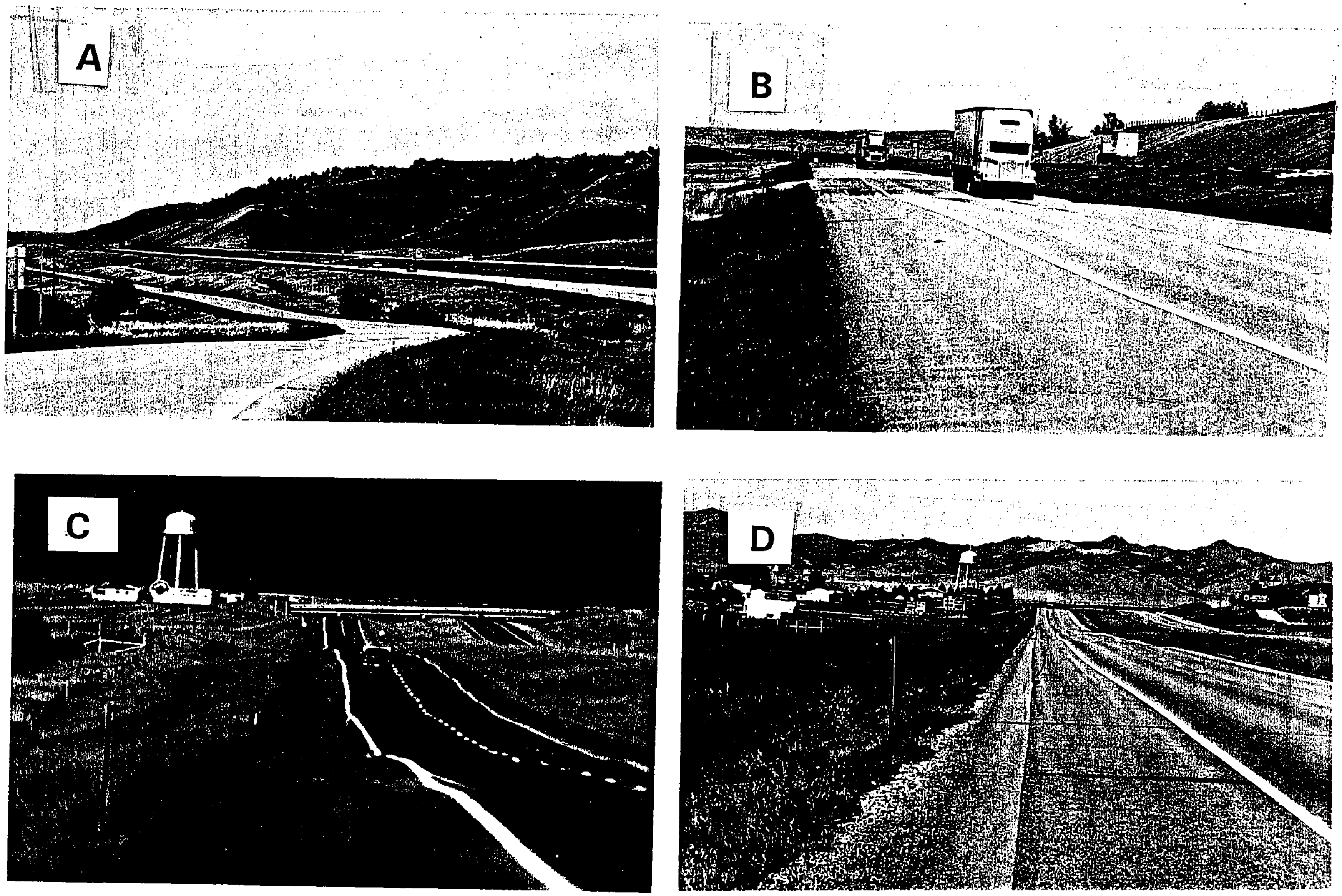

Figure 5. I-90 conditions MP 262-264. A. Overview DeHart Exit. B. Subsidence in west bound roadway. I-90 conditions Whitehall to to Cardwell. C. 1986. D. 1996

or basin fill environment) and expanding clays tend to give differential settlement.

In the 10 years since the field trip, this section of Interstate-90 has performed well. Some minor "waves" are now forming, but the overall ride is excellent. (Compare Fig. 5C and 5D).

Conclusions:

The removal of rock or fluids from beneath the earths surface will cause subsidence. A like situation occurs when additional load is applied to clay materials (montmorillonte) which cause a volume decrease and compaction. Total amount of subsidence often does not have major consequences. Surface facilities go through a complex compressive-tensile strain history in the transition from the original elevation down to the final elevation and are often subject to extensive damage.

The choice of a protective distance away from an underground (fluid or rock) removal is a function of the rock type and can be as great a distance as that represented by a 30° dip down from the surface to the opening. Time is the controlling factor, as the initial break through or pipe to the surface may be vertical or even overhanging, and the final results or effective areas are large.

In collapsing soils, the depth influenced by the loading will determine the total subsidence. These phenomena tend to grow with time and do not stabilize until the loading conditions imposed on the rocks are less than the long term residual strengths. The original rapid deposition and variable nature of the sedimentary deposits cause a wide variability in the subsidence giving rise to differential settlement.

A final concluding remark, the subsidence will approach the total volume of the extracted rock or fluids. Prudent design effort should recognize this fact and plan accordingly.

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Roadway Settlement Over Culverts: Causes and Cures

by:

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ABSTRACT: Settlement problems over culvert installations on Wyoming roadways are examined and potential corrective measures are discussed. Over 160 culvert sites in Wyoming were inspected as part of a field investigation. Of these sites, fifteen locations were drilled and sampled. Based on field investigations and information obtained from WYDOT personnel, three probable causes of settlement have been identified: (1) inadequate compaction, (2) shallow cover of fill above culverts, and (3) the use of plastic, compressible fill materials derived from bentonitic Cretaceous shales. Design/construction measures for minimizing roadway settlement over culverts are reviewed. The two primary alternatives are reinforced backfill and the use of controlled low-strength material (CLSM) for backfill.

Introduction and Background

Installation of culverts, water lines, stock passes, and other similar structures across roadways typically involves excavation of existing ground followed by placement of the structure, backfilling of the excavation with compacted soil, and construction of a pavement over the backfill. If vertical deformation occurs in the soil or rock beneath the structure, in the backfill material, or in the structure itself, it is likely to be manifested as vertical deformation, or settlement, at the surface of the roadway. The result of roadway settlement may be relatively minor and practically unnoticeable by motorists, or it may cause a significant "dip" in the roadway, posing a safety hazard and possibly vehicular damage. In extreme cases, settlement may result in cracking and premature deterioration of the pavement, requiring an overlay or pavement replacement. Figure 1 depicts the general nature of the problem. Delta (Δ) as seen in Figure 1 is the maximum deflection due to settlement.

Settlement of highway pavements overlying buried culverts has been identified as a problem by the Wyoming Department of Transportation. In some locations, settlement is severe enough to pose a hazard to driver safety and has created the potential for vehicular damage. Even moderate settlement decreases highway quality and contributes to pavement damage that requires costly maintenance and possibly early replacement of some pavements before the design life has been achieved.

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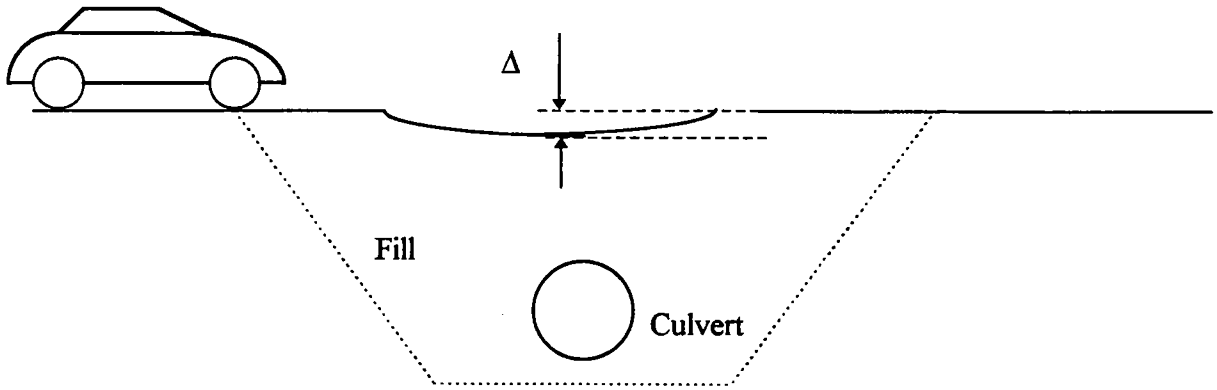

Figure 1. Roadway settlement over culvert.

This paper describes a study currently underway at the University of Wyoming aimed at developing design and construction procedures to eliminate or minimize damaging roadway settlement above culvert crossings. Preliminary investigations revealed the following WYDOT perceptions: (1) the number of sites in Wyoming at which excessive settlement has occurred is not known, however the general perception of maintenance personnel is that the problem is widespread and not limited to any particular region of the state; (2) the mechanism(s) by which excess settlements occur are not known; and (3) existing design and construction procedures are not resulting in satisfactory performance The research described in this paper is aimed at further defining the scope of the problem, identifying the controlling mechanisms, developing methods to prevent or minimize future settlement, and presenting the findings in the form of recommendations for design and construction of backfilled culverts and pipes for implementation by WYDOT personnel. The research objectives are to:

1. Determine if roadway settlement appears to be related to culvert or pipe characteristics, geological environment, construction practices, or other factors.

2. Establish the mechanism(s) of settlement, for example, compression settlement of fill around and above culverts and pipes under traffic loads, settlement of soil beneath culverts and pipes, deformation of the culverts and pipes, or some combination of causes.

3. Evaluate design and/or construction procedures to prevent or minimize settlement for new construction.

To achieve the stated objectives, research is being carried out in two phases. The first phase consists of field and laboratory investigations to establish the most likely cause(s) of settlement. The second phase consists of experimental and analytical evaluation of design and construction procedures to eliminate or minimize roadway settlement for future culvert installations. Each of these research activities and results obtained to date are described in the following sections.

Field Investigation of Roadway Settlement

Methods

The field investigation involved collecting and analyzing information on existing sites of culvert in Wyoming. The information was obtained from three sources: (1) WYDOT maintenance personnel, (2) WYDOT project records, and (3) site visits and inspection. Information on culvert sites at which excess settlement has occurred was obtained initially by conducting a telephone survey of district maintenance personnel. Based on these discussions, four areas of the state involving twelve road construction projects were selected for more detailed investigation. The four areas are: (1) WYO 487 between Medicine Bow and Casper, (2) WYO 70 east of Baggs, (3) WYO 113, Pine Haven Road, and (4) various locations in District 5 near Lander, Thermopolis, Meeteetse, Cody, and Worland. All available records for each project were reviewed, including information pertaining to the soils and any available records from construction, such as field inspection reports or field test results.

The project team visited each of the twelve selected sites to make detailed inspections. Items which were inspected and documented included culvert type, size, and location, evidence of ground movement, damage to culverts and pipes such as buckling and excessive deflections, field identification of subgrade and backfill soils, description of drainage facilities, possible sources of water to the subgrade and backfill soils, approximate magnitude of roadway settlement, and the extent of pavement damage, if any.

For each culvert location, a qualitative damage assessment was made. Damage is described by three categories: minor, moderate, or severe. Minor damage is characterized by zero to slight settlement (0-3 inches). Moderate damage consists of settlement in the range of 3-6 inches or that required patching. A designation of severe damage indicates settlement of over 6 inches, collapsing pipes, and/or heavy maintenance requirements since the completion of construction. This classification is somewhat arbitrary and is based on crude estimates of actual settlement, but is deemed adequate for this investigation

Following the initial site inspections, fifteen culvert crossings were chosen for subsurface drilling and testing. All of the sites chosen for drilling exhibited moderate or severe roadway damage. The WYDOT Geology Program provided a drilling rig and crew. The first and fourth authors were present to observe drilling operations, record blow counts, and log soil samples. Samples of subgrade and backfill soils were collected and transported back to the Geotechnical Engineering Laboratory at the University of Wyoming, where they are currently being tested. Testing includes compaction tests, identification of potentially detrimental compositional factors such as the presence of swelling clays or gypsum, and collapse-consolidometer tests. Table 1 gives a summary of the locations that were sampled along with additional information about each site.

Table 1. Summary of Drilling Locations

Location and Road	Milepost[1]	Culvert Size[2], in	Cover Height, ft	Cover/Diameter Ratio (C/D)	Depth of Boring, ft (m)
Med. Bow-Casper	47.97	3@36	5.3	1.8	24.0 (7.4)
Shirley Basin	50.24	60	4.6	0.9	38.9 (11.9)
WYO 487	50.47	60	9.3	1.9	29.4 (9.0)
	51.18	36	10.0	3.3	23.1 (7.1)
	51.54	36	3.4	1.1	24.1 (7.4)
Baggs-Encamp.	3.14	2@36	4.5	1.5	24.1 (7.4)
WYO 70	3.25	42	4.7	1.3	25.1 (7.7)
	3.65	2@36	6.0	2	19.6 (6.0)
	4.36	2@36	16.0	5.3	19.6 (6.0)
Thermop.-Meet.	3.4	36	15.0	5	24.1 (7.4)
Owl Creek	4.2	66	15.0	2.7	29.1 (8.9)
WYO 120	4.98	112 x 75	10.0	1.3	24.1 (7.4)
Thermop.-Meet. Grass Creek WYO 120	26.6	2@24	4.5	3.5	7.9 (2.4)
Worland-Thermop.	152.3	42	8.0	2.3	22.6 (6.9)
US 20	152.5	96	35.0	4.4	52.2 (15.9)

1. Culvert locations indicated by approximate milepost.
2. All culverts sampled were round CMP with the exception of the arch pipe with the given dimensions.

Results and Probable Causes of Roadway Settlement

Detailed results of the field investigation, including descriptions of each culvert site inspected and drilling logs for each of the fifteen sites chosen for further study, are presented in Turner et al. (1996). Only the significant findings are summarized herein.

Information collected by examining project records, speaking to maintenance personnel, and site inspections suggests several possible causes of roadway settlement and damage at culvert crossings. One obvious concern of all Wyoming DOT field personnel is the possibility of inadequate compaction of backfill around and above culverts. There is a perception that there are not enough inspectors on most road construction jobs to insure that the contractor conforms to compaction specifications. Several maintenance foremen mentioned that in some cases no inspector was present during the entire culvert backfilling operation. Additional inspectors are not likely to be assigned because of budget limitations and, therefore, this situation is not likely to improve.

A second factor which appears to be significant is the depth of cover between culvert and pavement. It was observed that a high percentage of sites with moderate to severe damage seemed to have relatively low cover depths. To evaluate this further, graphs were developed showing depth of cover (in meters) versus ratio of cover depth to culvert diameter (referred to herein as cover ratio). Data points were distinguished on the basis of minor, moderate, and severe damage, as defined previously in this report. These graphs are included herein as Figures 2-5. Figure 2 shows all of the culverts on the roadway section that was investigated on WYO 487 between Medicine Bow and Casper. For a cover height and cover ratio of 3.5 or less, 32% of the culverts show moderate or severe damage. In addition, all locations showing severe damage are within this range of cover height. For cover heights between 3.5 m and 5 m and cover ratios between 3.5 and 5, 33% of all culvert sites showed moderate damage, while above a cover depth of 5 m and cover ratio of 5, no sites showed anything greater than minor damage.

Figures 3 and 4 show similar graphs for WYO 70 between Baggs and Encampment (Fig. 3) and Pine Haven Road (Fig. 4). In each case, only minor or no damage was observed for cover heights over 3.5 to 5 m and for cover ratios over 3 to 6.5. Figure 5 shows a similar plot for the sites in District 5. The data collected in District 5 differs from the other three in that only locations reported as being problems were investigated, while other culvert sites on the same projects were not inspected. Therefore, it was not possible to determine the percentage of sites with moderate or severe damage for a given cover height or cover ratio. District 5 includes several sites at which severe and moderate damage were observed even though cover heights and cover ratios were relatively high.

Although there are some exceptions, these data strongly suggest that culvert sites having shallow cover, say 5 m or less, are more likely to undergo damaging roadway settlement. Only two sites exhibiting severe damage have cover heights of greater than 5 m, and it may be that factors other than cover height are significant at these sites.

A third factor which emerges from the investigation is the possible influence of site geology and its influence on the types of soils used as backfill. It was found that the majority of sites at which damaging settlement has occurred are located in areas underlain by Cretaceous sedimentary rocks known to contain bentonite, which is a strong indicator of swelling, highly compressible soils. Furthermore, soil samples obtained from drilling at these sites are for the most part fine-grained soils such as clay, sandy clay, silty clay, or clayey silt. Although laboratory tests on samples of these soils are not complete, testing to date indicates that many of the soils used as backfill are highly plastic and compressible fine-grained soils. Standard Penetration Test N-values indicate "medium" stiff consistency for these cohesive soils.

Considering the types of soil used as backfill, a possible and likely source of roadway settlement in backfilled areas is compression of the backfill soil. Wyoming Department of Transportation Specifications (1996) call for the following:

"a. ***Backfill Material.*** Backfill material placed under, adjacent to, and over pipe conduit, precast and cast-in-place structures, shall be a fine, compactible, excavated soil or granular fill material. Backfill material shall not contain frozen lumps, chunks of highly plastic clay, or stones which would damage the structure."

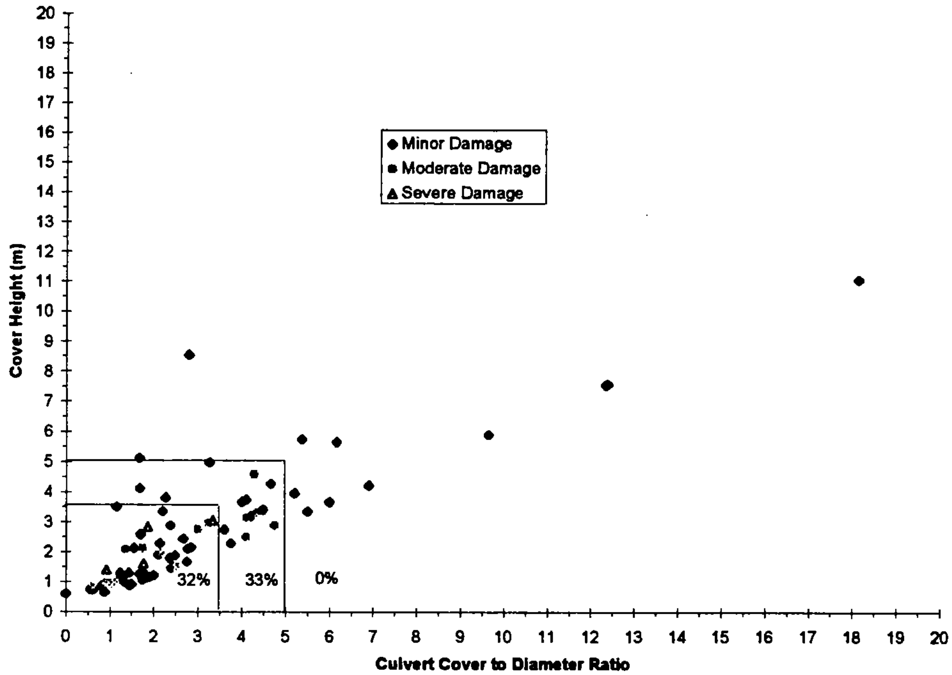

Figure 2. Cover height versus cover ratio, Shirley Basin

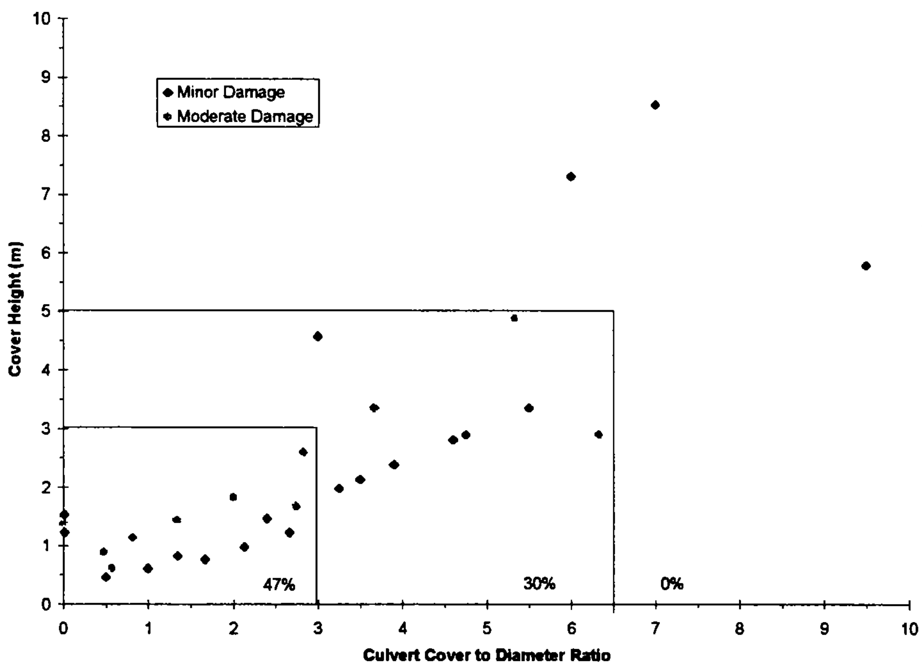

Figure 3. Cover height versus cover ratio, Baggs-Encampment

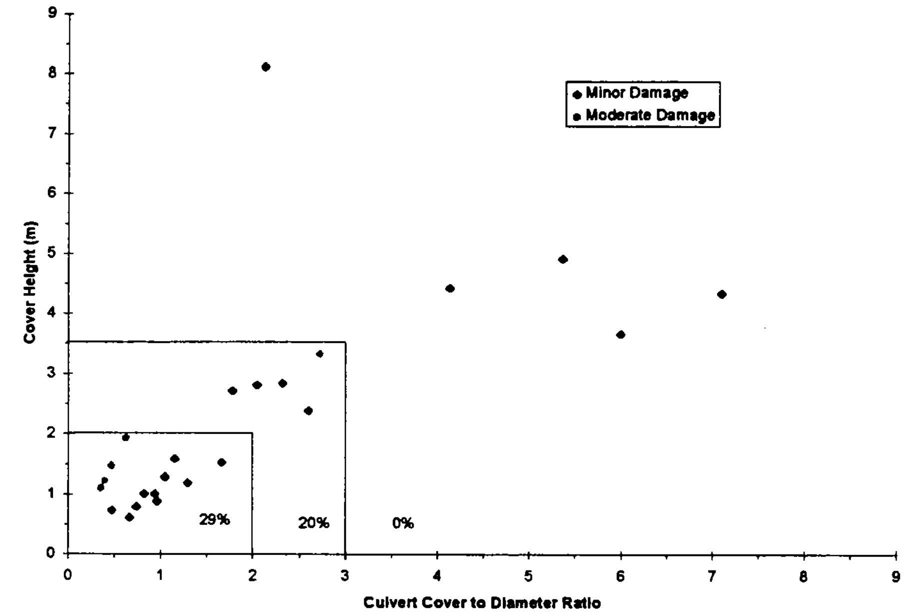

Figure 4. Cover height versus cover ratio, Pine Haven Road

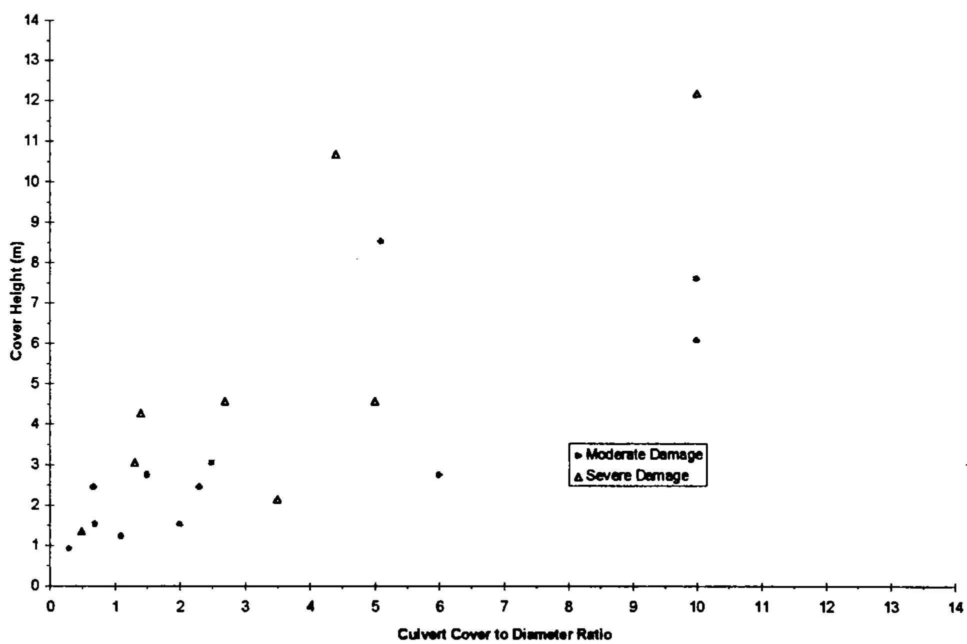

Figure 5. Cover height versus cover ratio, District 5 sites

Materials used as backfill and conforming to the above specification could show considerable variability in terms of geotechnical properties, in particular with regard to compressibility. For example, a "fine, compactible, excavated soil" could contain a significant percentage of fines (silt and clay size particles), and may exhibit a high degree of compressibility even when compacted to 95 percent of maximum dry density. Compressibility generally is greater for cohesive (fine-grained) soils than for cohesionless (coarse-grained) soils. Laboratory tests currently being conducted on backfill and subgrade soils from the drill sites include Atterburg limits, grain size distribution, consolidation, and triaxial compression tests.

In summary, there appear to be three factors having a high probability of resulting in damaging roadway settlement at culvert sites on Wyoming highways:

1) inadequate compaction, as a result of fewer inspection personnel
2) shallow cover of fill above culverts
3) the use of plastic, compressible subgrade soils and/or fill materials derived from bentonitic Cretaceous shales.

Methods to Control Roadway Settlement

Measures that can be taken to reduce or eliminate settlement of roadways above culverts and pipes can be placed into two categories: (1) improvements in compaction specifications and procedures and (2) other ground modification techniques in addition to compaction. Improvements in compaction specs and procedures involves more detailed screening of soils to evaluate their suitability as backfill, specifications tailored for specific soil types and compaction equipment, and improved quality assurance and control during field compaction.

Reinforced Backfill

Besides compaction, which would be classified as mechanical ground modification, other ground modification techniques may be required to minimize settlement. One technique that appears to have a high potential for addressing this problem is soil reinforcing. In recent years, great strides have been made in the development of low-cost, durable materials for soil reinforcing, and the applications have become widespread. The purpose of reinforcing a soil mass is to improve its strength and bearing capacity and reduce settlement and lateral deformation. This is accomplished by placing inclusions in the soil which are able to resist tensile stresses and which interact with soil through friction and/or adhesion. One type of reinforcing which may be applicable to backfilled culvert and pipe installations is geogrids. Geogrids are open-meshed polymeric sheets, relatively flexible and lightweight. They are easily handled in the field and connections of adjacent sheets can be made with interweaving cords. Geogrids are available in various grades of strength and stiffness and are fabricated by stretching of prepunched sheets of high-density polyethylene (HDPE) or polypropylene.

A recent study conducted by Bauer (1994) found that "geogrids used as overlays for flexible conduits at shallow depth were also found effective in reducing surface settlements and deformations of the model conduits." The study involved the test setup shown in Figure 6(a), in which the backfill above the buried conduit was reinforced with one or more layers of geogrid. Figure 6(b) shows load-settlement responses for cover soils reinforced with two layers of geogrid, compared to cover soil with no reinforcing. These results indicate the potential for geogrid reinforcing to control surface settlements above buried conduits.

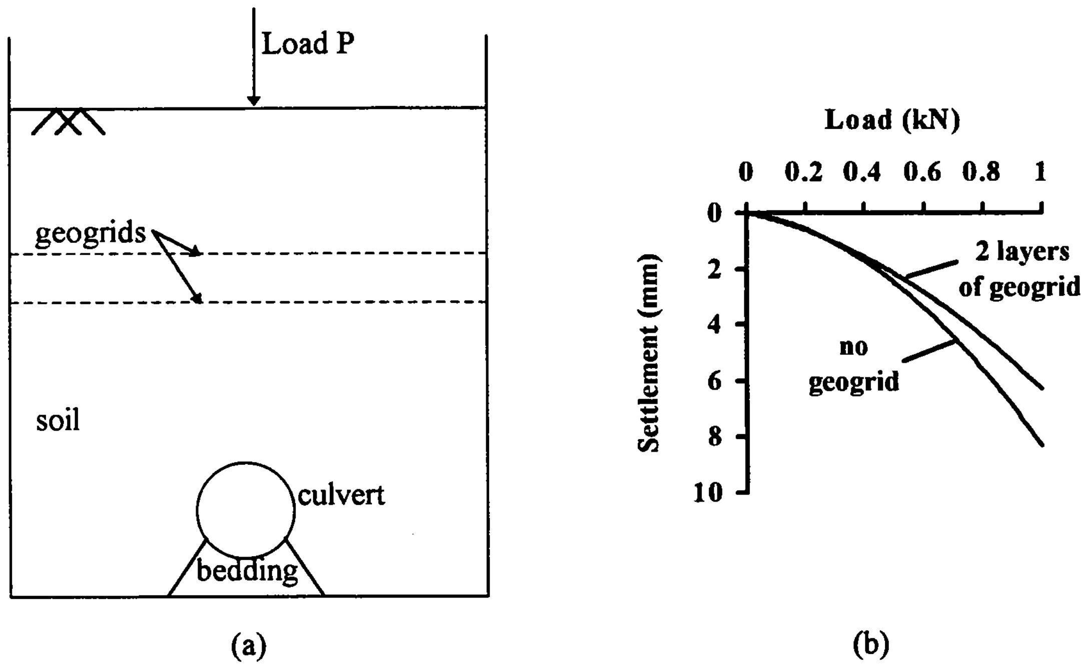

Figure 6. Test setup and load settlement responses.
(Modified from Bauer, 1994)

Controlled Low-Strength Material (CLSM)

As defined by ACI Committee 229 (1994), controlled low-strength material (CLSM) is a self-compacted, cementitious material used primarily as a backfill in lieu of compacted fill. Controlled low-strength materials are defined by "Cement and Concrete Terminology (ACI 116R)" as materials that result in a compressive strength of 1200 psi or less. Most current CLSM applications require unconfined compressive strengths of 300 psi or less. The lower strength requirement is necessary to allow for future excavation of CLSM.

Controlled low-strength material should not be considered as a type of low strength concrete, but rather a self-compacted backfill material that is used in place of compacted fill. Additionally, CLSM should not be confused with compacted soil-cement. Soil cement, as defined by ACI Committee 230 on Soil-Cement, requires compaction and curing. CLSM typically requires no compaction or curing to achieve the desired strength. Although CLSM generally costs more per cubic yard than most soil or granular backfill materials, its many advantages often result in lower in-place costs.

Table 2. Ranges for CLSM Mix Designs

Material	Quantities (lb/cubic yard of fill)
Cement	50 - 280
Water	375 - 750
Fly Ash	0 - 2,000
Fine Aggregate	2,100 - 3,160

Mix Design. A typical CLSM mix contains cement, water fly ash, and fine aggregate. Ready mix producers can combine these components in varying proportions to meet specific performance requirements and to take advantage of locally available materials. Ranges for quantities used for different mixes are given in Table 2. As cited by Smith (1991), various organizations have recommended mix proportions including: Ohio Ready Mixed Concrete Association, Michigan Ready Mixed Concrete Association, National Ready Mixed Concrete Association, Iowa Department of Transportation, and the South Carolina Department of Highways and Public Transportation.

The principal components of a CLSM mix are cement, water, fly ash, and fine aggregates. Alternatively, chemical admixtures and non-standard materials can also be added. Chemical admixtures such as water reducers, superplasticizers, and accelerators. Additionally, air-entraining admixtures can also be included. The air-entraining agents improve flowability, reduces the density of the mix, reduces effects of freeze/thaw, and enhances the economy of the mix. Overall, however, chemical admixtures may not be cost effective unless needed for a specific requirement.

Other non-standard materials may also be used in CLSM mixtures. These materials, if available, may be more economical depending on the project requirements. Examples of materials that may be used as aggregates include bottom ash produced in the coal combustion process, discarded foundry sand, and reclaimed crushed concrete (ACI 229, 1994). As mentioned previously, aggregates that may swell due to expansive reactions or other mechanisms should be avoided. Some non-standard materials that may not be suitable for CLSM mixtures include wood chips, wood ash, or other organic materials.

According to ACI Committee 229 (1994), there are fifteen advantages to using CLSM as backfill. These include: readily available using local materials, ease of deliverance, ease of placement, versatility, strength and durability, can be excavated, less inspection, faster return to traffic, no settlement, reduction in excavation cost, increase in worker safety, all-weather construction, reduction in equipment needs, eliminates need for storage, and the use of a by-product (ACI 229, 1994). Some of the key advantages for this project include: versatility, strength and durability, less inspection, control of settlement, and all-weather construction.

CLSM offers the potential for eliminating damaging settlement of the roadway. If a soil backfill is not compacted properly when placed, settlement may occur after being paved. Cracks and dips may form in the roadway due to this settlement. The CLSM, on the other hand, does not form voids during construction and reportedly does not settle or rut under loading.

Versatility is important in Wyoming, as not all sites have the same requirements. The CLSM mix can be adjusted to meet the requirements at each site as needed. For instance, strength can be varied by adjusting the amount of cement or fly ash used in the mix. Other properties of the mix can also be adjusted such as flowability and setting times. CLSM mixes often have load-carrying capacities greater than a compacted backfill of soil or granular fill. Additionally, CLSM is less permeable and more resistant to erosion. As stated previously, CLSM can reach compressive strengths of up to 1200 psi which would work well as a more permanent structural fill.

Less inspection is a very desirable component of CLSM. Some of the problems with road settlement pointed to the possible lack of enough inspectors. CLSM is self-compacting, thereby reducing extensive field tests that must be done on soil backfill. A final advantage that seems important within the application of this study is all-weather construction. With varying conditions in Wyoming, construction schedules are always interrupted causing delays and additional costs. With the possibility of rain or snow at almost anytime, trenches can fill up with water making pumping necessary. CLSM displaces standing water in trenches. In addition, materials used for CLSM can be heated as for ready-mixed concrete to be used in cold weather.

As backfill, CLSM can be readily placed into a trench, hole or other cavity. As noted previously, no compaction is needed. Therefore, the trench width or size of excavation may be reduced. Granular or site excavated backfill, even if compacted properly in the required layer thickness, may not achieve the uniformity of CLSM.

A case study from Peoria, Illinois (Smith, 1991) illustrates the use of CLSM to mitigate severe settlement problems of soil backfill in utility trenches. In 1988, CLSM was used as an alternate backfill material. The CLSM was placed in trenches up to 9 feet deep. Although fluid at the time of placement, the CLSM "hardened" to the extent that a person's weight could be supported within 2 to 3 hours. Shrinkage cracks were very minimal. Additional tests were conducted placing a pavement patch within 3 to 4 hours. A pavement patch was successfully placed over a sewer trench immediately after the trench was backfilled. As a result of the success of the CLSM, the city of Peoria has changed its backfilling procedure to require the use of CLSM on all street openings.

CLSM has not been used widely on WYDOT projects. However, on two recent projects, the contractor requested and was given permission to use a "flowable fill" consisting of cement, sand, and water at no additional cost (Flom, personal communication). Eleven culverts were backfilled with this mix on US 191 in 1995, with fill heights ranging from 0.6 m to 4.6 m. These sites will be monitored to assess the performance of CLSM.

Research to Evaluate Settlement Mitigation Methods

A second research phase is currently underway at the University of Wyoming. Laboratory load tests are being conducted on scale models of culverts embedded in compacted soil. The model culverts and backfill are constructed and instrumented in a 1.5 m x 2m x 1.5 m deep concrete chamber designed and fabricated for this purpose. Loads are applied by an MTS electro-hydraulic actuator which is computer controlled and capable of simulating repeated loads due to traffic.

Both unreinforced backfill and geogrid-reinforced backfills will be tested to evaluate potential improvements provided by reinforcement. Vertical deformations will be predicted using finite element analysis (FEM) of the soil-structure system. The influence of variables such as the number of geogrid layers, their depth and spacing, soil properties, and magnitude of load will be evaluated both experimentally and analytically in order to optimize geogrid placement for minimizing settlement. If it can be demonstrated that FEM analysis yields satisfactory agreement with experimental results, it will be used to design and predict the behavior of a full-scale test section of a geogrid-reinforced culvert installation. Similar model tests will be conducted on cement-stabilized backfill.

Summary and Conclusions

A field investigation of roadway settlement above culverts on Wyoming highways identified the following probable causes:

1. Inadequate compaction, due to difficulties in making proper backfill observations and testing

2. Low soil cover, defined as 5 m or less

3. Use of plastic, compressible soils derived from bentonitic shales as backfill

Sections of roadway which exhibited combinations of the above three factors, such as WYO 487, Medicine Bow to Casper, exhibited the most severe roadway damage at culvert sites.

Several methods for reducing or eliminating roadway settlement were reviewed, including geosynthetic reinforcement of backfill soils and the use of Controlled Low-Strength Material (CLSM), which is a flowable, cementitious mixture consisting of cement, water, fly ash, and fine aggregate. The use of reinforced backfill has yet to be proven as an effective and economical solution to the settlement problem, but limited laboratory research suggests that it warrants further study. Use of CLSM is a proven technology which is currently being used successfully by several state DOT's, including WYDOT on a limited basis, and it is highly recommended as a means to control roadway settlement above culverts. CLSM may offer other advantages, the most important of which are ease of construction and less need for rigorous inspection. Laboratory load tests of both reinforced backfill and CLSM backfill are underway at the University of Wyoming to evaluate the effectiveness of each of these methods for reducing settlement above culverts.

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Debris Flow Mitigation Using Flexible Barriers

by

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Abstract

During the summer of 1995 a 40,000 square acre wild fire occurred in San Luis Obispo County. Most of the fire spread through steep mountainous terrain where several state highways are located. In the aftermath, Caltrans Geologists reviewed the highway cut slopes in the burn areas for potential slope instabilities associated with burn areas. Several locations were identified as having a potential for rockfalls and debris flows if the following winter brought heavy rains. At one location a wire rope rock net was recommended and constructed by maintenance personnel due to rockfall activity. The 1995/1996 winter storms were severe, eroding many of the slopes in the burn area. At the location of the rock net a 60 cubic meter debris flow cascaded down the mountain into the barrier. All the material was stopped while in adjacent areas debris flows covered the roadway. This is the first time a flexible wire rope rock net was recorded to have stopped debris flows. Common wisdom up to this point was that the fine grained material would pass through the net grid.

Barriers of this type are widely used for rockfall mitigation and have been tested by the California Department of Transportation. As a result of this event additional studies on the mechanics and interaction between these barriers and debris flows have been performed by the California Department of Transportation, California Polytechnic State University, and the U.S. Geological Survey . In these studies four different flexible barrier designs, developed by Geobrugg for debris flows, were tested. Flows, ranging between 9 and 10 cubic meters, were initiated in the USGS debris flow flume.

The results of these studies and actual field performance support the use of these systems in debris flow mitigation.

Introduction

In July 1994 a forest fire burned through 40,000 acres of mountainous terrain surrounding the City of San Luis Obispo, California. While much of this area was in National Forest 10 to 20 percent affected rural communities, county roads and state highways. In the aftermath of this disaster county and state officials quickly set out to assay the damage and plan mitigation. All disciplines were represented from environmental to transportation. The principle concern for transportation personnel was the effect the denuded slopes could have on slope stability above and below Transportation corridors. The California Department of Transportation in part with the California Department of Forestry rehab team and county officials set about the task to review the state highways in the effected areas. With the potential for heavy winter rains much of the effort was focused on hill side erosion in the form of debris flows, mud flows, and rockfalls.

Investigation

Initially all pertinent Geologic data available was reviewed. Areas with a history of instability were identified as were areas underlain by known unstable geologic units. Proceeding with this data the target areas were reviewed by air and photographed. This aerial reconnaissance proved an invaluable tool for prioritizing target areas and developing a better understanding of the impact the fire had on slope stability. Following this review geologic personnel did a ground survey of each site. Each area was mapped, unstable sections were delineated, and hazard potential was assigned. Local highway maintenance personnel and residents were interviewed in order to develop a history of the instabilities. This was done in an attempt to determine what areas were directly de stabilized by the fire versus areas that were unstable prior to the fire and what effect the fire had on those areas.

The hardest hit section of roadway was along California State route 41. This corridor connects Highway 101, one of the States major north south corridors, with coastal communities along Highway 1, the Pacific Coast Highway. Within this section of roadway several locations were identified as having a extremely high potential for debris flows, mud flows, and rockfalls. This area has a history of such events and with the complete elimination of vegetation there was a heightened concern of what might occur during heavy winter rains. This concern was heightened by the doubling of rockfalls in this section during and immediately following the fire. Interestingly rockfall activity tapered off to normal within days after the fire. It was within this section where much of the initial mitigation was implemented.

Mitigation

Slope mitigation consisted of scaling, slope draping, and flexible rockfall barriers. More than 300 meters of slope was scaled, 500 square meters of slope was covered with drapery, and 80 meters of flexible rockfall barriers were installed.

Scaling is the removal of loose rock by hand or with the use of pry bars. This work was performed by maintenance crews trained in the use of ropes. Using state personnel proved timely and cost effective. In most instances only cut slopes in bedrock were scaled but in specific sections burned natural slopes, consisting of colluvium, were also scaled of loose rock and debris.

Draping the slope to control rocks as the rocks move downslope was installed in one section on natural slopes. This work was also performed by state personnel. The area covered consisted of colluvium. The slopes were approximately 15 meters high and 30 meters wide. Chain link fence fabric was used to cover the slope. The seams were connected with hog rings. Anchoring was achieved by hammering in T-stakes 1.5 meters into the colluvium. To date this installation is performing well with no evidence of failures.

A flexible rockfall barrier was installed where steep natural slopes occurred above the roadway. These slopes consist of colluvium with occasional bedrock outcrops and are over 500 meters long. The entire slope has been denuded exposing hundreds of loose rocks, small drainage's, and swales. Draping the slope would require covering the mountainside and scaling would have been endless and neither is effective against soil movement. A barrier at grade would provide the greatest protection.

Flexible Barrier

The barrier used in this installation is a "rockfall net". A "rockfall net" is a flexible barrier capable of catching and containing falling rocks **(Figure 1).**

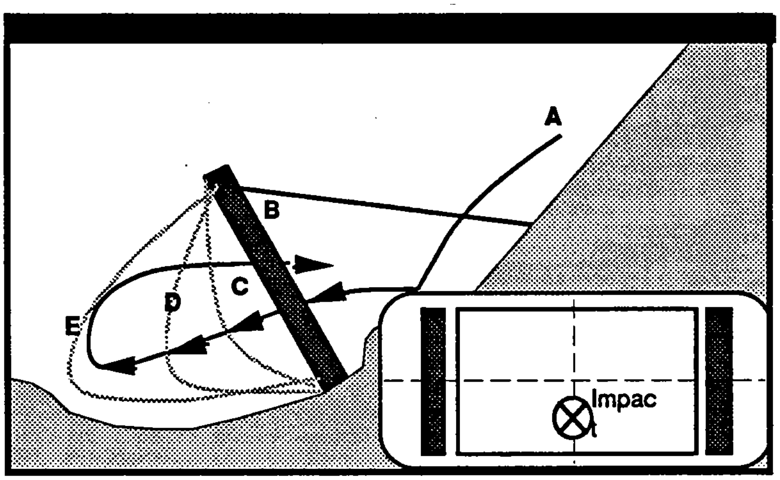

Figure 1: Fully Flexing Net

This capability is a result of the net's design as a flexible system rather than a standard, fixed-wire fence system. A properly designed system is flexible enough to absorb the anticipated energy with minimum damage to the system. Considerable flexibility is inherent in the net material and the support wire rope infrastructure. Additional flexibility is added by using energy dissipating "friction brakes". The friction brakes are attached to the wire rope support system and dissipate energy through friction as the wire ropes are pulled in tension. The system installed at this location consisted of rectangular panels of woven wire rope vertically supported by steel posts supporting a cable infrastructure and designed with frictional brake elements (**Figure 2**).

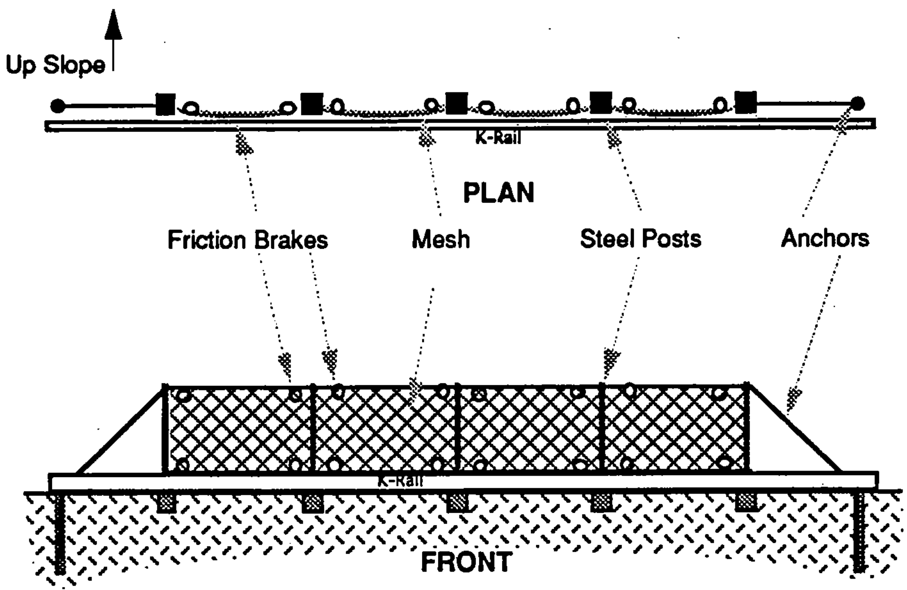

Figure 2: Typical Plan View and Front View of a Geobrugg Rockfall Net.

Each woven wire rope net panel was 5 meters wide and 3 meters high. The panels are woven into a 200 mm x 200 mm diagonal pattern. The nets are covered on the impact side with chain link mesh. Chain link mesh is used because of its flexibility. Each panel was connected to a 18 mm cable infrastructure with 9.5 mm wire rope. The nets and infrastructure are supported 200 mm diameter steel pipe which are set in 1.6 meter holes backfilled with pea gravel.

The net barrier was located 0.6 meters from the edge of traveled way thus requiring crash barrier for errant vehicles. A k-rail was placed in front of the barrier. The rock net barrier was designed such that the bottom of the net was at the same height as k-rail (**Figure 3**). In this way barrier height was increased, net flexibility was not compromised, and traffic safety was maintained.

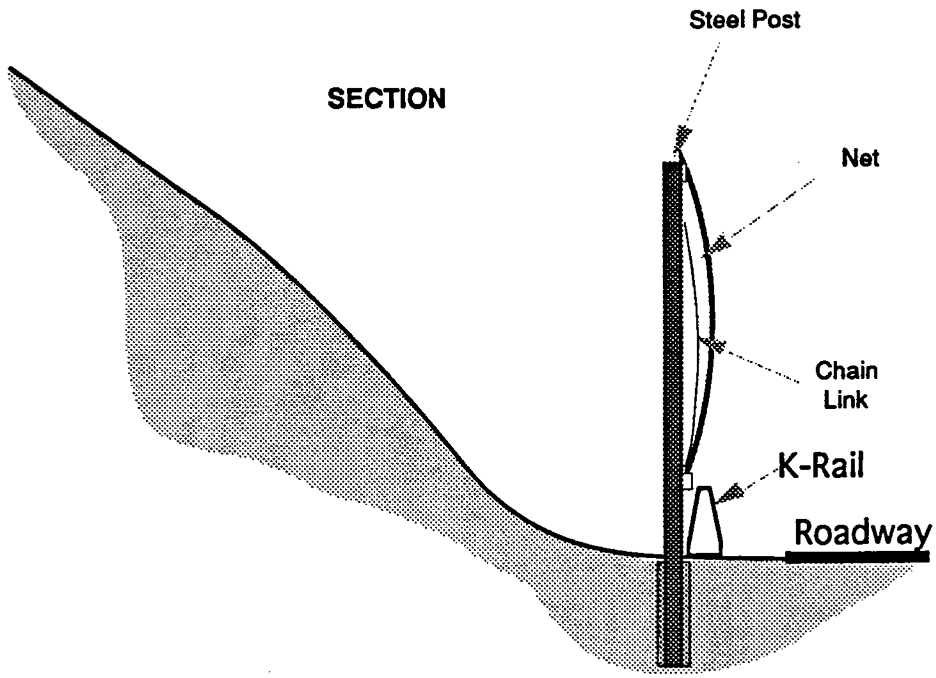

Figure 3: Typical Side View of a Brugg Rockfall Net

Performance

The 1994/1995 winter storms were severe, eroding many of the slopes in the burn area. At the location of the rock net a series of debris flows impacted the barrier. Each flow appeared to be associated with an individual rainfall period so that during a storm event several high intensity rainfall events would occur initiating debris flows. An estimated 60 cubic meters of debris flow material accumulated behind the barrier as the result of three to six individual flows. Estimated impact energies ranged between 80 and 130 kilo-joules of kinetic energy. Energy estimates were based on comparisons with known net performance from real test data and calculations based on flow mass and published velocity ranges of debris flows.

Less than 3 percent of the material passed through the barrier while in adjacent unprotected areas debris flows covered the raodway requiing closure. This was attributed to rapid draining of the mass and soil arching. Laboratory testing indicated the soil to be non-plastic, well graded, gravely sand (USCS classification SW). Although the percentage of fines lost during the flow is unknown, these test results are consistent with typical debris flow soil properties.

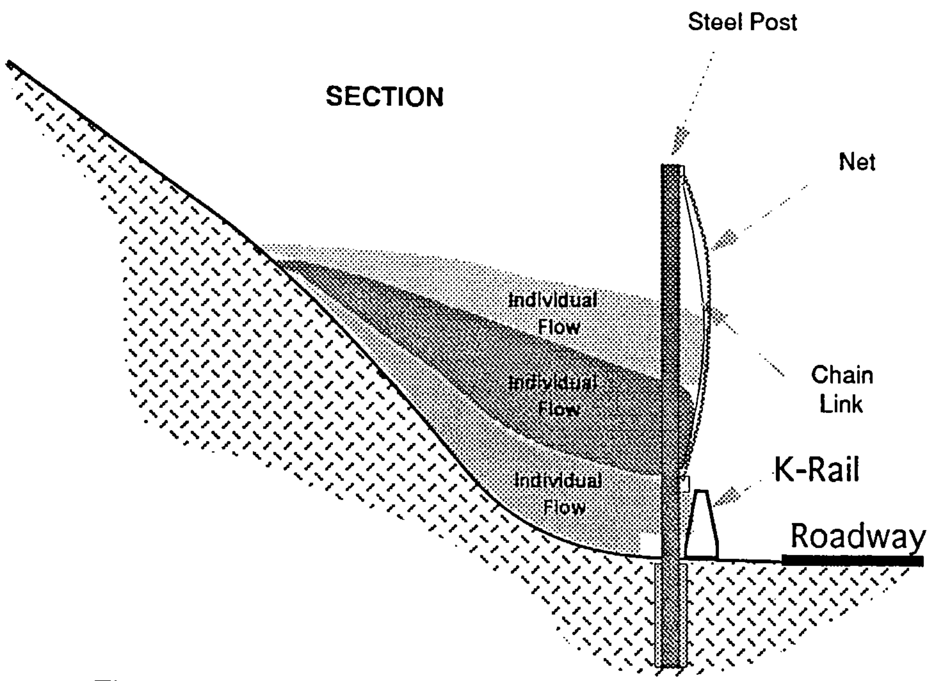

Figure 4: Debris Flow Sequence into Barrier

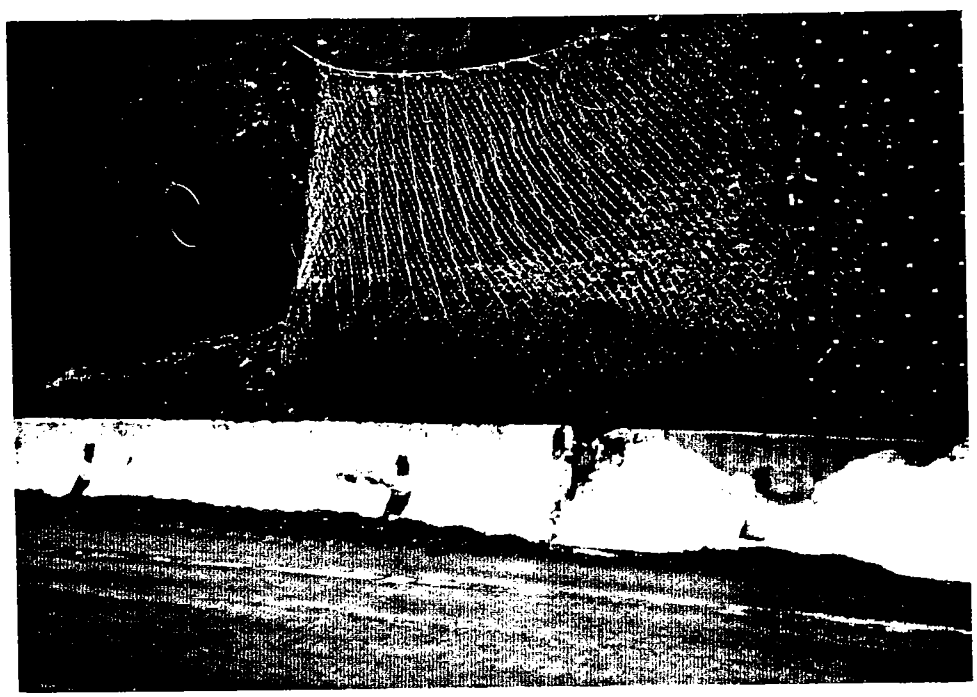

Photo 1: Debris Flow Stopped in Flexible Barrier on California State Rotue 41.

Maintenance operations further proved the designs usefullness in this application, In between storms maintenance personnel were able to clean the nets by removing the k-rail and cleaning our the debris form under the barrier. Once the k-rail was removed the mass slid our in front of the net for easy cleaning. Also while cleaning personnel were protected by the barrier.

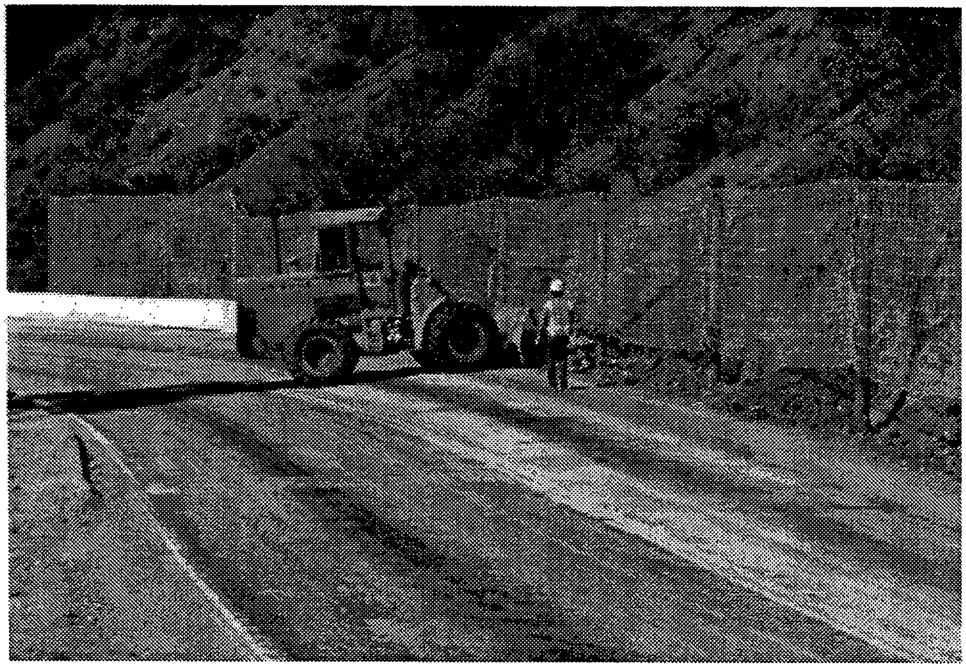

Photo 2: Maintenance

Testing

In June 1996 four different net designs were tested at the USGS debris flume research facility in H.J. Andrews Experimental Forest near Blue Lake, Oregon. This facility has a 90 meter long flume with a 30 degree slope angle. Flows as large as 10 cubic meters were created and released. The four designs, supplied by Geobrugg, consisted of various types of flexible barriers with varying mesh sizes. The purpose of this work was to determine energy and soil arching capabilities of the different designs.

The findings of this testing corroborated with actual field models. Debris flow mechanics are such that the material will arch and drain rapidly enough such that the flow acts as a solid mass upon impact and does not flow through the net mesh (**Photo 1**). It was observed that the initial flows have the greatest impact by subsequent flows spread laterally gradually filling the area being the barrier with little dynamic impact. Maintenance of these systems was minimal and cleaning operation were acceptable although 3 to 4 meters of space as required in front of the barrier for equipment.

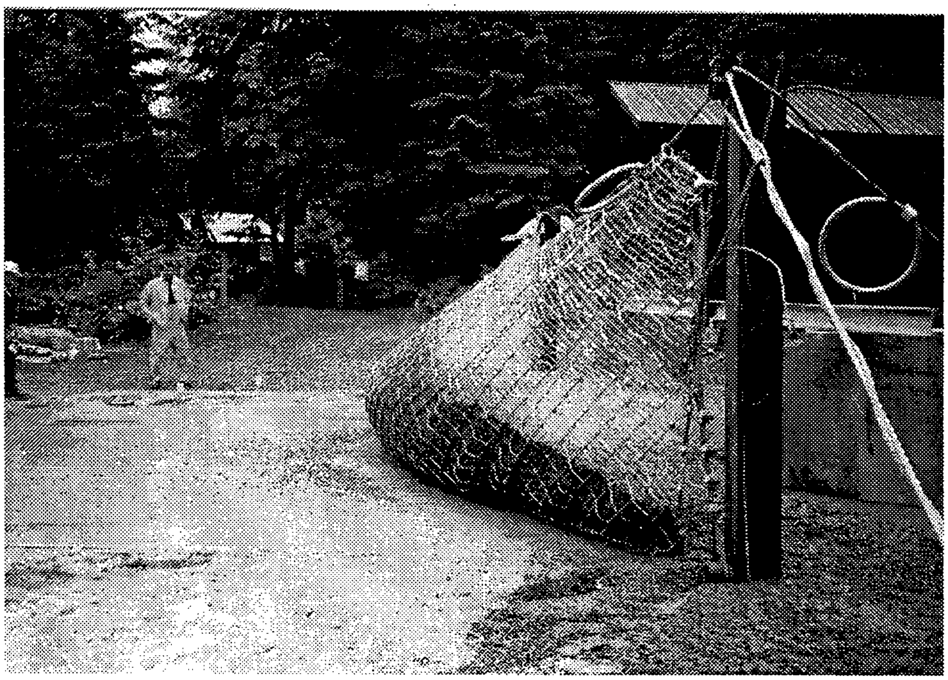

Photo 3: Debris Flow Flexible Barrier Test.

Debris flow dynamic loading was examined as Newtonian flow in terms of kinetic energy. Unlike rock fall mass is the largest variable with velocity relatively constant at these levels. The mass not only varies with the size of the flow but also with the variability of the standing wave which are characteristic of debris flows. Preliminary indications suggest energies, within the parameters of these tests, range between 100 and 300 kilo joules. But the loading on the system differs in the mass size and total area loaded.

The final report is currently being prepared by the California Polytechnic State University Team. The results of this work is forthcoming and will be available through the University.

Conclusions

Barriers of this type are widely used for rockfall mitigation and have been tested by the California Department of Transportation, the Colorado Department of Transportation, and others. The successful field performance of the barrier and subsequent barrier testing indicate that modified flexible rockfall barriers are capable of stopping debris flows as large as 10 cubic meters with acceptable levels of Maintenance. Larger flow mitigation may be possible with further studies and case histories.

Preliminary Investigations Using ALROC™ Potliner Sand in Asphalt Concrete Mixtures

by

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Preliminary Investigations Using ALROC™ Potliner Sand in Asphalt Concrete Mixtures

Kevin D. Hall[1]

ABSTRACT

ALROC™ is a sand-sized aggregate material that results from crushing spent "potliner" from aluminum manufacturing processes. The potliner sand is treated and stockpiled as a non-hazardous waste material. A preliminary investigation was conducted to determine whether ALROC could be used as a source aggregate for hot-mix asphalt concrete.

Specimens of hot mix asphalt concrete containing ALROC potliner sand as one of the sand-sized source aggregates were prepared for mix design. Mix designs were performed using the Marshall mix design method and Superpave™, the mix design system developed under the Strategic Highway Research Program (SHRP). Gyratory compaction curves for ALROC mixes, generated for Superpave, were compared to compaction curves representing a typical hot mix asphalt concrete. During Marshall mix design, volumetric properties, including voids in the total mix (VTM), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA), were measured for the ALROC mix specimens and compared to properties measured from a typical hot mix asphalt concrete.

Results from both Marshall and Superpave procedures show that ALROC can be used to produce hot mix asphalt concrete. However, the optimum asphalt content of mixes produced using ALROC generally was 2 to 2.5 percent higher than for typical mixes. The most probable explanation for the discrepancy is the relatively high absorption of ALROC compared to limestone and natural sand. It was noted that relatively high percentages of ALROC were used in mix design (ALROC comprised 14 to 20 percent of the aggregate blend). A smaller percentage of ALROC in the aggregate blend may reduce the additional demand for asphalt cement in a mix. The overall behavior of ALROC mixes was similar to that of typical mixes. Gyratory compaction curves for the two mix types had similar shapes, and the trends of volumetric properties versus asphalt content were similar for the two mix types.

A comprehensive investigation is needed to fully quantify the effect of ALROC on the properties of hot mix asphalt concrete. The study should include both Marshall and Superpave procedures, a number of source aggregates, and various percentages of ALROC in the aggregate blend. Based on the preliminary results reported here, measures for reducing the absorption capacity of ALROC could improve the usefulness of the material in hot-mix asphalt.

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Preliminary Investigations Using ALROC™ Potliner Sand in Asphalt Concrete Mixtures

Kevin D. Hall

Introduction

Hundreds of thousands of tons of asphalt concrete are produced in the United States each year for pavement construction. Each ton of this asphalt concrete requires sound, quality aggregates in order to perform as intended in the pavement structure. Some regions of the country have an abundance of quality aggregates for hot-mix asphalt production; other areas must import aggregates to satisfy construction demands. Methods of reducing the demand for virgin aggregates by asphalt concrete producers have been a "hot topic" in the industry for a number of years. One such method is the use of waste materials in hot-mix asphalt concrete, such as ground tire rubber, glass, roofing shingles, and "recycled" asphalt concrete. The use of waste products in asphalt concrete serves two purposes, namely the reduction in demand for virgin aggregates and the elimination of a potential source of waste in an already overburdened solid waste stream. However, the asphalt industry is understandably reluctant to introduce any material into asphalt concrete that will jeopardize the performance of the asphalt in paving applications.

This study examines the potential for using a waste product, ALROC™ (a sand-sized aggregate-like material resulting from the production of aluminum), in hot-mix asphalt concrete. The data presented here focuses on asphalt mix design, rather than mixture performance in a field application.

Background Information

Hot mix asphalt concrete (HMAC) is a combination of aggregates and asphalt cement, compacted to some specified density. The type, size distribution, and physical properties of the aggregate can play a significant role in HMAC durability and performance. Typical aggregates used for HMAC in Arkansas include crushed limestone, crushed gravel, natural gravel, limestone screenings (sand-sized particles resulting from the limestone crushing operation), and natural or river sands. Prior to being routinely used in asphalt concrete, an aggregate must meet certain physical requirements such as angularity, toughness, and soundness (resistance to weathering). Gradation specifications for HMAC are established by the pavement agency (e.g. the Arkansas Highway and Transportation Department) and are usually based on extensive experience and research with various gradations. Typically, aggregates from various sources are blended to meet a specified gradation; it is rare if not impossible to find a single aggregate source that naturally has a specified gradation.

Currently, most states (including Arkansas) use the Marshall method for designing asphalt concrete mixes. Once a suitable aggregate blend is developed, the Marshall mix design procedure is used to identify the *optimum asphalt content* for the mixture.

Specimens of hot mix asphalt concrete are prepared in the laboratory at various asphalt contents, using a Marshall hammer to compact the mix into a specified mold. Each compacted specimen is analyzed with respect to its *volumetric* properties (air voids [VTM], voids in the mineral aggregate [VMA], and voids filled with asphalt [VFA], its *Marshall stability* (a measure of strength), and its *flow* (a measure of resistance to deformation). Each of the properties VTM, VMA, VFA, stability, and flow, are plotted against asphalt content. The optimum asphalt content is initially estimated as the asphalt content that results in 4 percent air voids (VTM) in the mix. Each property is checked at that asphalt content against established design criteria ***(1)***.

Although the Marshall procedure has been used for nearly 50 years, premature pavement failures in the 1970's and 1980's highlighted the need to re-evaluate asphalt mix design. An extensive research program, SHRP (**S**trategic **H**ighway **R**esearch **P**rogram) was conducted from 1988-1993; one result from SHRP was Superpave™, a new asphalt mixture design and analysis procedure. Superpave was developed as a *performance-based* design procedure. In other words, the tests and procedures performed in the laboratory during mix design relate to the performance of the mix in the field. Parts of the Superpave system are currently being readied for implementation in many states ***(2)***.

Superpave contains three *levels* of mix design. A higher level of design incorporates more sophisticated mix performance testing and, consequently, provides a greater reliability that the mix will perform as designed. The basic design level, Level 1, uses the volumetric properties of the mix (4 percent air voids) to determine the optimum asphalt content, much like the Marshall procedure. However in Superpave, hot-mix specimens are prepared using a *gyratory* compaction machine as opposed to "impact" compaction employed by the Marshall hammer. Gyratory compaction produces mixes in the lab that more closely imitate field compaction that does impact compaction ***(3)***. Another advantage of the gyratory compactor is that mix compaction can be *monitored*. In other words, from the compaction curve generated by the gyratory compactor the designer can identify potential mix problems, such as a mix that compacts too easily (a possible tender mix) or a mix that will not obtain sufficient compaction. In addition to obtaining 4 percent air voids at a specified number of gyrations, limits are placed on how quickly a mix compacts and the ultimate density a mix can achieve ***(4)***.

Description of Investigation

__Materials__. ALROC™ is a treated, spent potliner material that results from the production of aluminum. Some physical properties of the material that are important to asphalt mixes, such as gradation (particle size distribution), specific gravity, absorption, and angularity are shown in Table 1. Of these properties, two causing some concern are the absorption and bulk specific gravity (G_{sb}). The samples showed relatively high absorption rates and low values of specific gravity.

Gradation		Specific Gravity / Absorption	
Sieve	*Percent Passing*		
3/4"	100.0	Bulk (G_{sb}):	1.908
3/8"	99.2	Apparent (G_{sa}):	2.649
#4	86.6		
#8	74.9	Absorption:	14.7 %
#10	71.5		
#16	62.8		
#30	47.0	**Fine Aggregate Angularity**	
#50	23.7		
#100	2.7	Uncompacted Voids = 46.5%	
#200	0.4		

Table 1. ALROC™ Material Properties

Aggregates used in the HMAC mixes in this study are typical limestone aggregates that are used extensively in hot mix asphalt in Arkansas. These aggregates are currently being used in research conducted at the University of Arkansas for the Arkansas Highway and Transportation Department (AHTD). The "unmodified" mix used in this study for comparative purposes was developed as part of the AHTD research effort. ALROC™-based mixes were developed by substituting ALROC™ for one of the source aggregates (a river sand) in the unmodified mix; the goal of the substitution was to create an ALROC™-modified mix having a gradation similar to that of the associated unmodified mix.

Figures 1, 2, and 3 show the gradation for both the ALROC™-modified and unmodified HMAC mixes. Note that three mix gradations are shown: "coarse", "medium" and "fine" (Figures 1, 2, and 3, respectively). All three gradations meet AHTD specifications for a "Type 2" surface mix; the gradation specification bands are shown with the mix gradation curves. Gradation curves representing ALROC™ mixes are very similar to those representing unmodified mixes. Comparisons of compaction properties for Superpave specimens were made for all three gradations. Marshall procedure HMAC specimens were fabricated using only the "medium" gradation shown in Figure 2.

Methodology. The purpose of this investigation was to provide data for making a preliminary estimation of the usefulness of ALROC™ as a fine aggregate for hot mix asphalt concrete. Selected properties of HMAC specimens prepared using ALROC™ were compared to properties of a "typical" HMAC mixture.

Specimens were prepared in accordance with both Marshall and Superpave mixture design methodologies. For the Marshall specimens, the asphalt content giving approximately 4 percent air voids (the typical "design" void content in the laboratory), and the volumetric properties Voids in the Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA) are compared for ALROC™ and unmodified HMAC mixes. For Superpave specimens, the laboratory compaction curves are compared for ALROC™ and unmodified specimens.

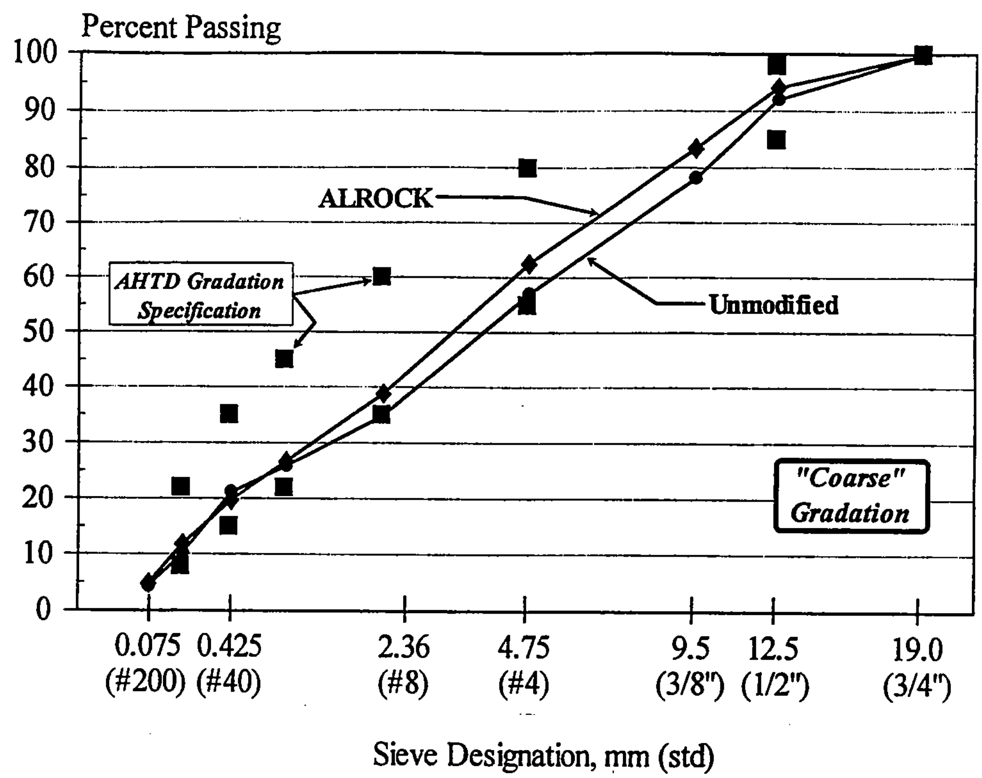

Figure 1. Coarse Gradations for ALROC™ and Unmodified Mixes

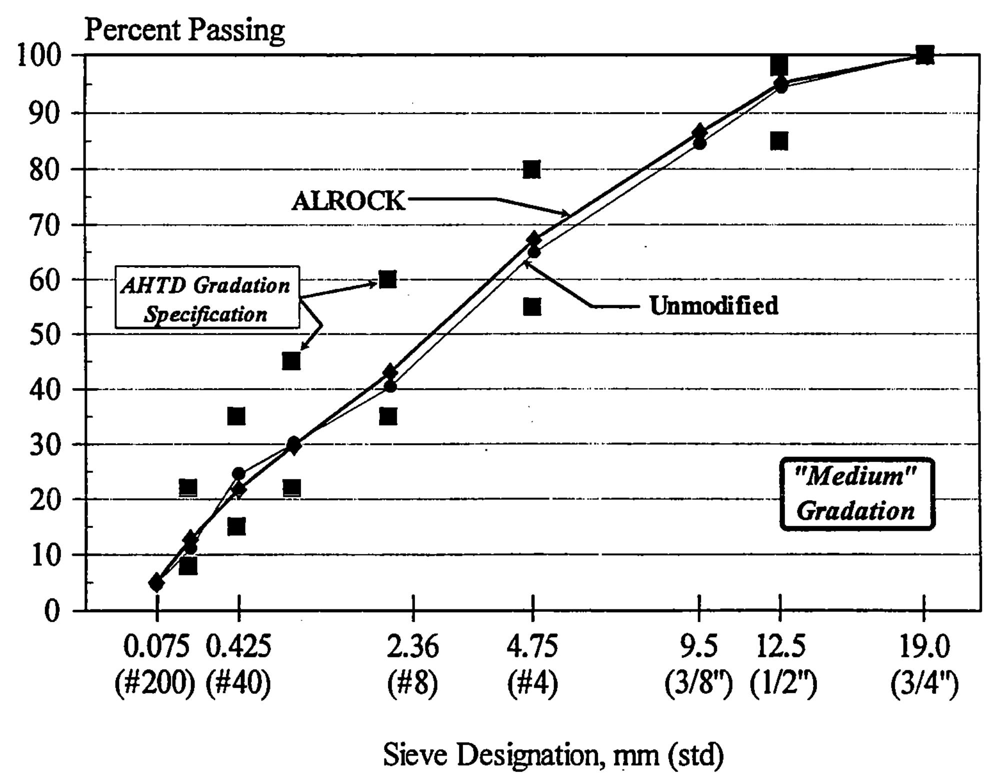

Figure 2. Medium Gradations for ALROC™ and Unmodified Mixes

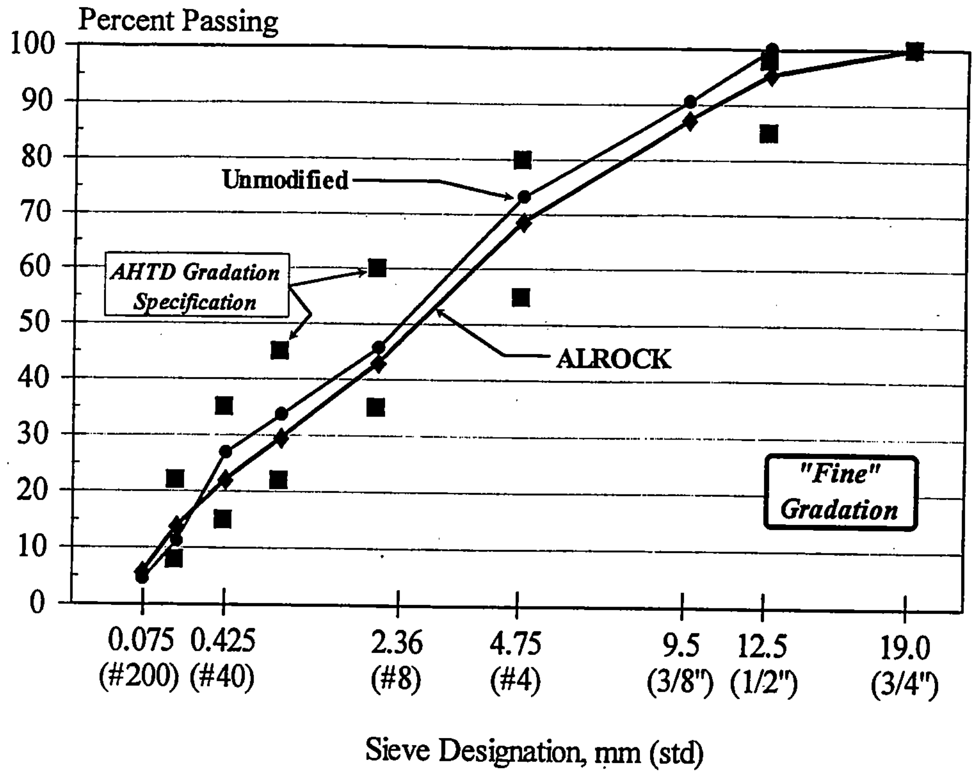

Figure 3. Fine Gradations for ALROC™ and Unmodified Mixes

Marshall procedure based HMAC specimens were prepared at four asphalt cement (AC) contents, using compaction criteria for a "heavy traffic" mix (75 blows / side). Superpave specimens were compacted using the Superpave gyratory compactor (SGC); specimens were subjected to a minimum of 172 gyrations.

Results and Discussion

<u>Superpave Specimens</u>. Figures 4, 5, and 6 are each a plot of the "compaction curve" generated by the SGC, showing percent of G_{mm} (theoretical maximum density) versus number of gyrations. The amount of air voids in the specimen can be estimated by subtracting percent G_{mm} from 100. Each figure contains four curves -- ALROC™ @ 4.0% AC content, ALROC™ @ 5.5% AC content, unmodified @ 4.0% AC content, and unmodified @ 5.5% AC content.

In Superpave mix design, three points on the compaction curve are critical ***(4)***. The first is the minimum number of gyrations, typically between 7 and 10, at which the mix should not be compacted above 89% of G_{mm} (less than 11% air voids). The second is the "design" number of gyrations, which ranges from 68 to 172, at which the mix should exhibit 4% air voids (96% of G_{mm}). The design number of gyrations is a function of the traffic and temperature expected at the paving site ***(2)***. The third critical point is the maximum number of gyrations, ranging from 104 to 288, at which the mix should not exhibit less than 2% air voids (not greater than 98% of G_{mm}). Placing a control on the minimum number of gyrations ensures the mix is not "too compactible" or tender.

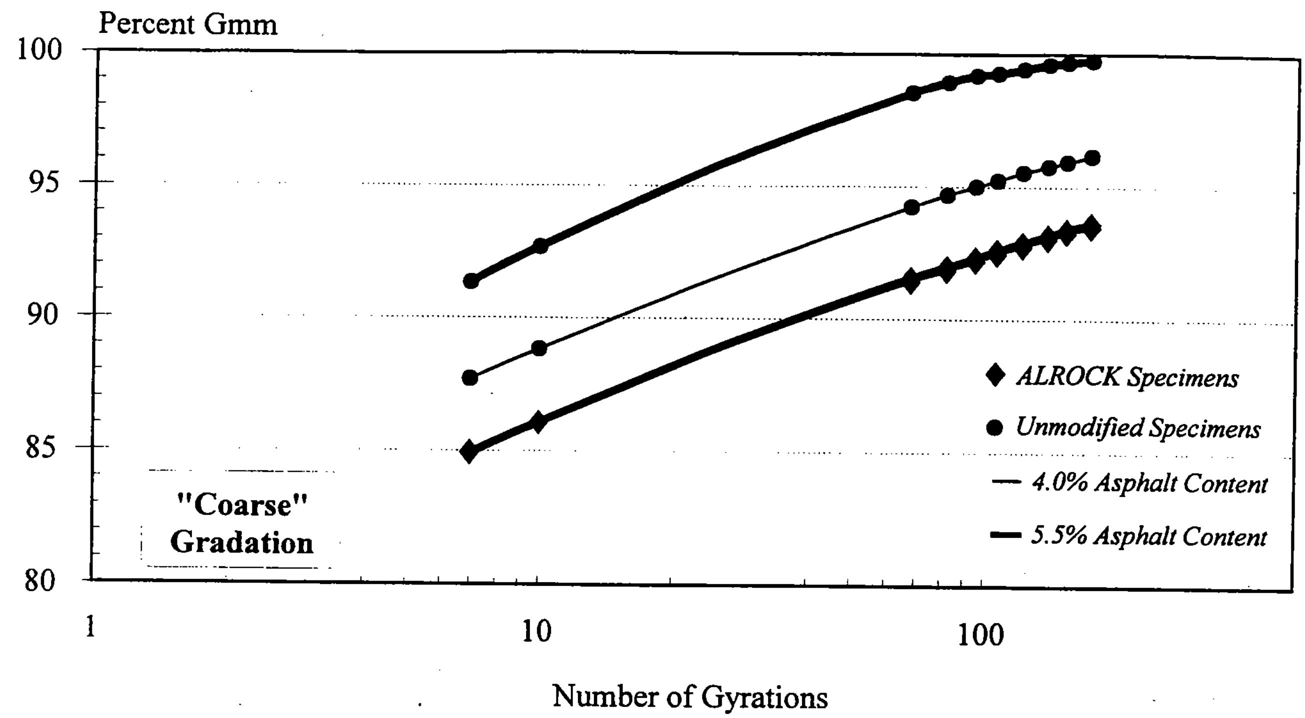

Figure 4. Gyratory Compaction Curves - Coarse Mix Gradation

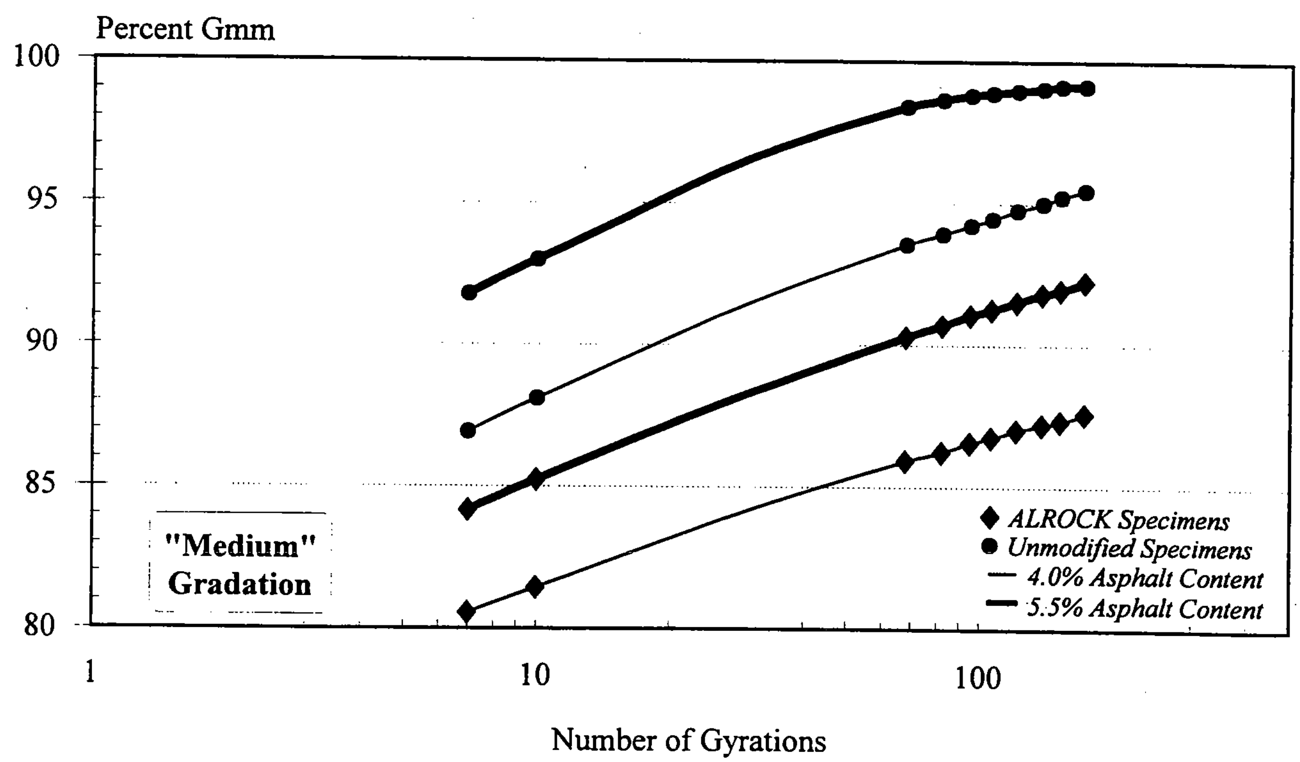

Figure 5. Gyratory Compaction Curves - Medium Mix Gradation

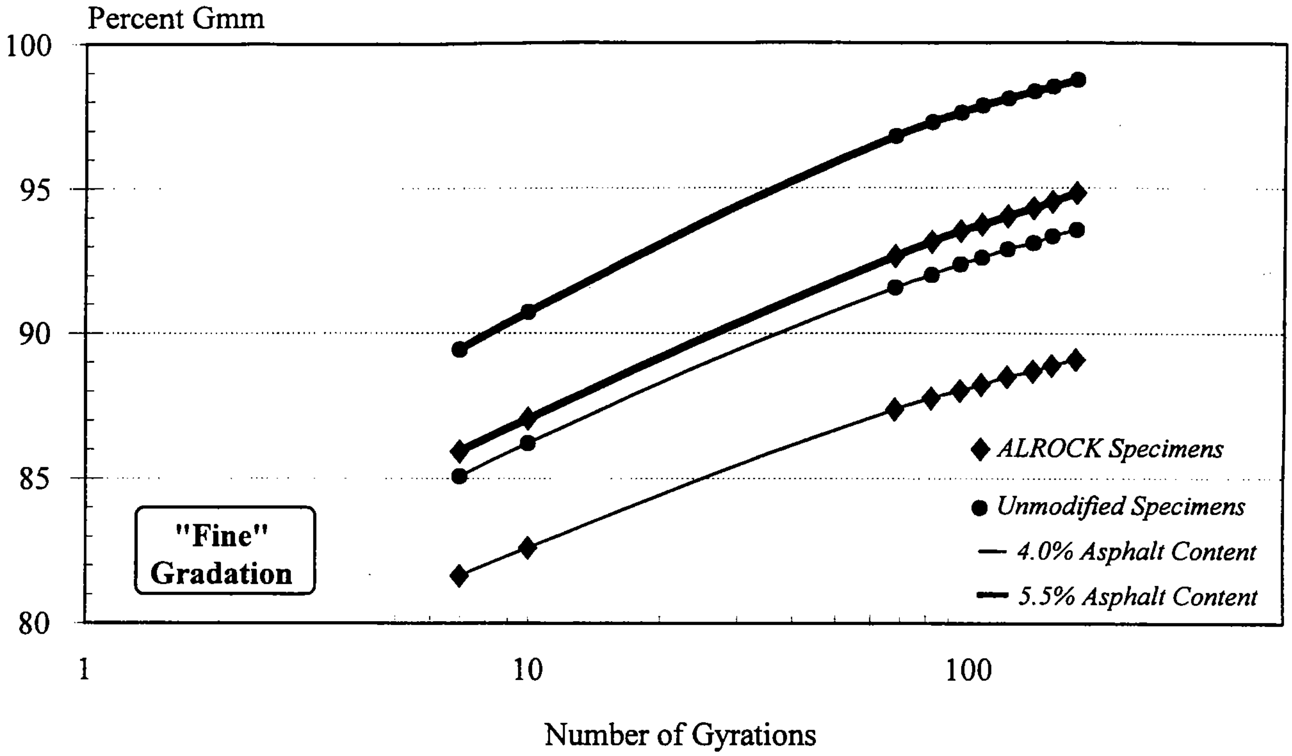

Figure 6. Gyratory Compaction Curves - Fine Mix Gradation

Placing a control on the maximum number of gyrations helps ensure the mix will not tend to rut after being in service. The design number of gyrations helps ensure adequate air voids for optimum mix durability and performance.

In general, ALROC™ mixes show similar compaction characteristics (the shape of compaction curves are similar) to unmodified mixes. However, for the same asphalt content, ALROC™ mixes do not compact to the same density levels as the unmodified mixes. This fact suggests the ALROC™ mixes require a higher asphalt content to obtain a specified density relative to unmodified mixes. In other words, a higher asphalt content will tend to push the compaction curve "up" (while maintaining the same basic shape) to a satisfactory density level. A purely qualitative assessment of the curves shown in Figures 4, 5, and 6 suggests the difference in asphalt content between ALROC™ and unmodified mixes required to achieve the same density is on the order of about 2.0 to 2.5 percent.

Other than the increase in asphalt content, ALROC™ mixes do not show adverse compaction characteristics. However, based solely on compaction curves it cannot be conclusively stated that ALROC™ may be used to produce Superpave asphalt mixes; additional testing is necessary. The required increase in asphalt content for ALROC™ mixes is most likely due to the relatively higher absorption of ALROC™. It is noted, however, that the ALROC™ modified mixes contain 20 to 25 percent ALROC™ in the aggregate blend. Lower percentages of ALROC™ in the aggregate blend may reduce the need for additional asphalt.

Marshall Specimens.

Figures 7, 8, and 9 contain information relative to Marshall mix design for ALROC™ and unmodified mixes. Figure 7 is a plot of air voids (VTM) versus asphalt content; based on the data shown, the ALROC™ mix would result in a higher optimum asphalt content (corresponding to 4 percent air voids) than the unmodified mix. The difference in optimum asphalt content between the two mixes is about 2.3 percent. This result is similar to the difference in asphalt content observed in the SHRP specimens. Again it is noted that ALROC™ comprised about 20 percent of the aggregate blend; lower percentages of ALROC™ may result in a smaller difference in optimum asphalt content.

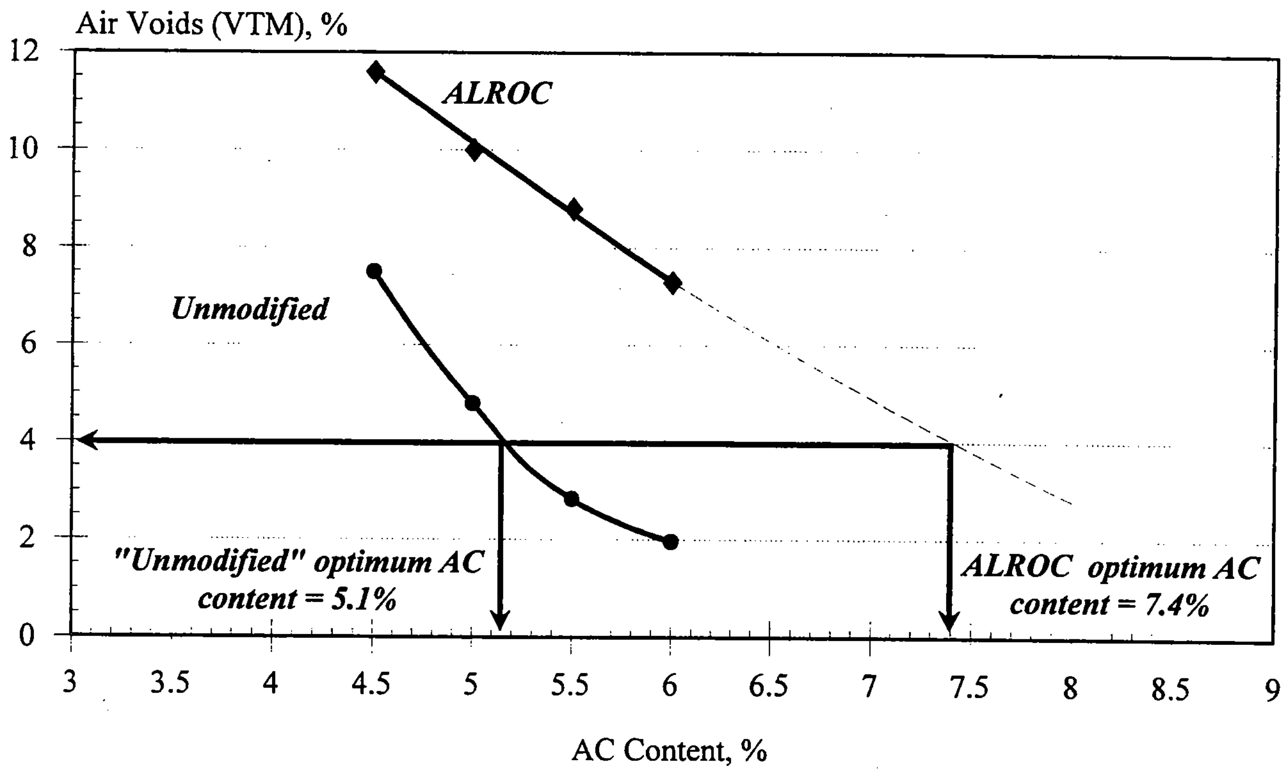

Figure 7. Percent Air Voids in Compacted Mixtures

Figure 8 is a plot of VMA versus asphalt content. It is shown that an ALROC™ mix can be produced that meets VMA specifications. In addition, the trend of VMA versus asphalt content for the ALROC™ mix is similar to that of the unmodified mix. Figure 9 shows VFA versus asphalt content. Again, the trend of the data is similar for both the ALROC™ and unmodified mixes. Based on the extrapolation shown, an ALROC™ mix may be made that meets current VFA specifications.

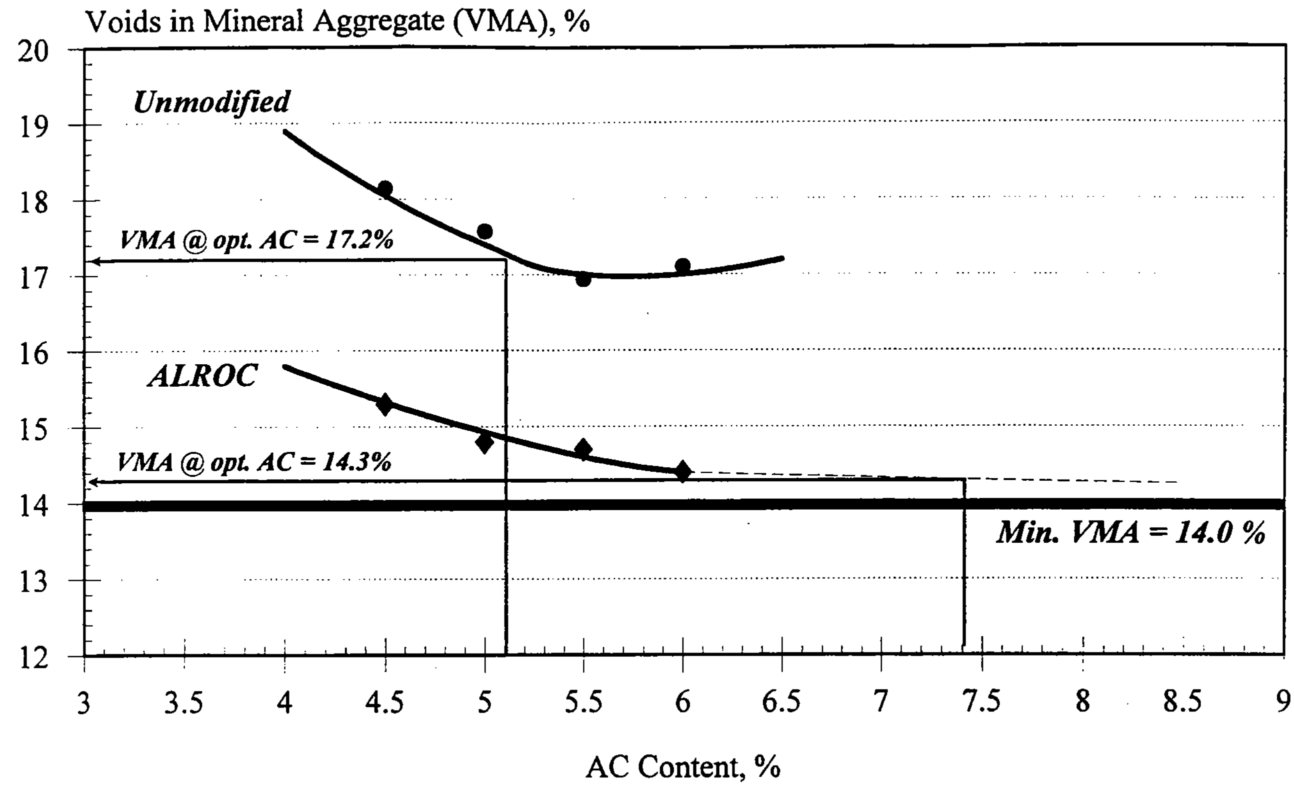

Figure 8. Percent Voids in Mineral Aggregate (VMA) in Compacted Mixtures

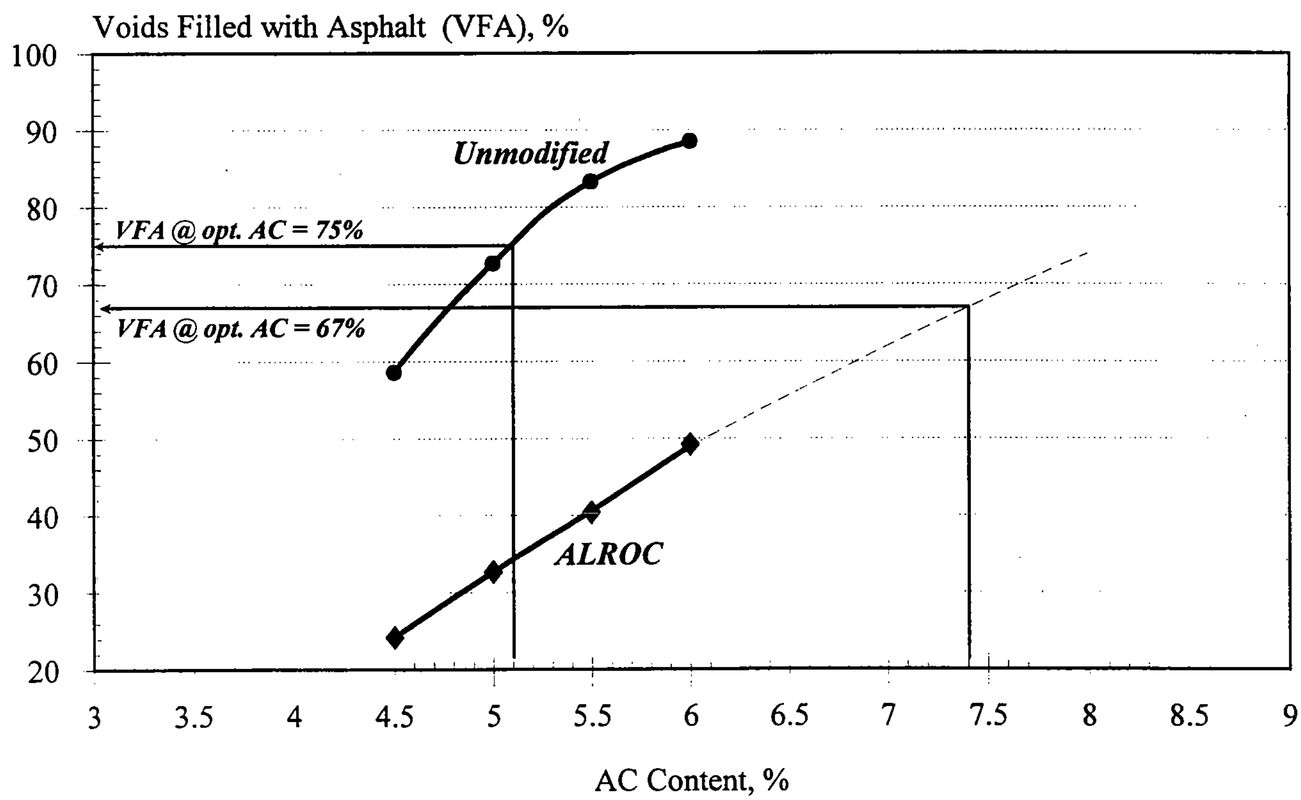

Figure 9. Percent Voids Filled with Asphalt (VFA) in Compacted Mixtures

Values of Marshall stability and flow for the ALROC™ mix specimens fabricated in the lab would be meaningless due to the relatively high air void level at low asphalt contents.

Conclusions

Based on the preliminary data shown here, it is reasonable to conclude that ALROC™ can be used to produce hot mix asphalt concrete. The overall behavior of the ALROC™ mixes was similar to that of typical unmodified mixes. Gyratory compaction curves for the two mix types had similar shapes, and trends of volumetric properties versus asphalt content were similar for the two mix types.

The optimum asphalt content of mixes produced using ALROC™ was generally 2 to 2.5 percent higher than for typical unmodified mixes. The most probable explanation for this discrepancy is the relatively high absorption of ALROC™ compared to limestone and natural sands. If ALROC™ is not (or cannot be) treated to reduce its absorption capacity, only small percentages may be used an asphalt concrete mixture to reduce the demand for asphalt cement.

As stated, the data shown here represent only a preliminary investigation into the use of ALROC™ in asphalt concrete mixtures. Only one percentage of ALROC™ was used in mix designs (ALROC™ comprised about 20 percent of the aggregate blend). A comprehensive investigation is needed to fully quantify the effect of ALROC™ on the properties of hot-mix asphalt concrete.

References

1. *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types*, Manual Series No. 2 (MS-2), 6th Ed., The Asphalt Institute, Lexington, Kentucky, 1995.

2. *The Superpave Mix Design Manual for New Construction and Overlays*, Report No. SHRP-A-407, Strategic Highway Research Program, National Research Council, Washington, D.C., 1994.

3. Sousa, J., et al, "Evaluation of Laboratory Procedures for Compacting Asphalt-Aggregate Mixtures", Report SHRP-A/UWP-91-523, National Research Council, Washington, D.C., 1991.

4. *Superpave Level 1 Mix Design*, Superpave Series No. 2 (SP-2), 1st Ed., The Asphalt Institute, Lexington, Kentucky, 1995.

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."

Thirteenth Annual Highway Geology Symposium - March 16, 1962 - Phoenix, Arizona.

Hall, Bruce M., "Status of Geological Registration."
Gallaher, B. J., "Desert Materials -- Types and Uses."
Eckel, E. B., "Use of Engineering Geology Maps."
Smith, P. C., "Purpose of the Highway Materials Inventory."
Sergent, B. D., "Soil Mechanics in the Southwest."
Mitchell, Stanley N., "Problems Facing the Engineer and the Geologist in a Highway Department Employing an Engineering Geologist."
Kersch, George A., "Regional - Aerial Geological Investigations in Highway Geology."

Fourteenth Annual Highway Geology Symposium - March 22, 1963 - A. & M. College of Texas - College Station, Texas, Texas Highway Department, and the Texas Bureau of Economic Geology.

Pelzner, Adrian, "Applications of Agricultural Soil Surveys to Highway Geology."
Laughter, C. N., "Correlation of Culvert Performance and Soil Conditions."
Henry, H. A., "Engineering Geology Operations in the Texas Highway Department."
Benson, Gordon R., "Geology -- A Vital Part of Subsurface Engineering in Illinois."
Welp, Theodore L., "Materials Geology, 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."

Fifteenth Annual Highway Geology Symposium - March 19, 1964 - Missouri Geological Survey and Water Resources - Rolla, Missouri.

Helmer, R. A., "The Duties and Training of a Geologist in the Oklahoma State Highway Department."
Grimes, Walter W., "Geology and Foundation Problems of Glacial Drift in Eastern South Dakota."
Migliaccio, Ralph R., "Engineering and Construction Problems in the Valdez District, Alaska."

Landrum, J. D., "A Foundation Investigation of Cherokee Cave Under Route I-55, City of St. Louis."
Lounsbury, W. R., and Schuster, R. L., "Petrology Applied to the Detection of Deleterious Materials in Aggregates."
West, T. R., and Aughenbaugh, N. B., "The Role of Aggregate Degradation in Highway Construction."
Heagler, John B., Jr., "Mineralogy and Soil Stabilization."

Sixteenth Annual Highway Geology Symposium - March 25-26, 1965 - University of Kentucky - Lexington, Kentucky.

Goodwin, W. A., "The Application of Geology in the Benefication of Aggregates."
Laughlin, G. R.; Scott, J. W.; and Havens, J. H., "Freeze-Thaw Characteristics of Aggregates."
Lounsbury, R. W., and West, T. R., "Petrography of Some Indiana Aggregates in Relation to Their Engineering Properties."
Sherwood, W. Cullen, Dr., "The Role of Aggregate Type in Pavement Slipperiness."
Smith, Preston C., "Landslide Research."
Struble, Richard A., "Shallow Subsurface Exploration Utilizing Airphoto Interpretation and Geophysical Techniques."
Dobrovolny, Ernest, "The Highway Research Board and its Committee on Engineering Geology."

Seventeenth Annual Highway Geology Symposium - April 21-23, 1966 - Iowa State University 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."

<u>Twenty-Third Annual Highway Geology Symposium</u> - 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 Turnpike in the Lower California Peninsula, Mexico."
Branthoover, Gerald L., "Geological Engineering Investigation of Talus Slopes in Lewiston Narrows, Pennsylvania."
Herbold, Keith, "Cut Slope Failure in Residual Soil."
Vogelsand, W. H., and Munyan, A. C., "Engineering Geology of the Semi-Indurated Strata of the Virginia Coastal Plain."
Thomas, Carl 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."

<u>Twenty-Fourth Annual Highway Geology Symposium</u> - 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."

Twenty-Fifth Annual Highway Geology Symposium - 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)

Twenty-Sixth Annual Highway Geology Symposium - 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."

<u>Thirty-Seventh Annual Highway Geology Symposium</u> 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., Wyllie, D., Dunsmore R., Smith R., Winger J., Paroni, A., Draeger, J., and Perfect, J., "Construction of a Tied Back Soldier Pile Rock Retaining Wall along I-90 in Northern Idaho, 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 Thomton, 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."

<u>Forty-Sixth Annual Highway Geology Symposium</u> - 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
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

PLEASE NOTE: The price listed does not include the charge for postage and handling. These charges will be added to the price listed. Copies of papers are $5.00 each plus postage and handling.

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