56th HIGHWAY GEOLOGY SYMPOSIUM

WILMINGTON, NORTH CAROLINA, MAY 4-6, 2005

PROCEEDINGS

HOSTED BY
THE
NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION

56th HIGHWAY GEOLOGY SYMPOSIUM

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HIGHWAY GEOLOGY SYMPOSIUM
56th ANNUAL

Wilmington, North Carolina
May 4th – 6th, 2005

The North Carolina Department of Transportation Geotechnical Engineering Unit welcomes you to the 56th Annual Highway Geology Symposium.

The Host Committee has put together what we hope is an interesting, educational and enjoyable Symposium. Authors will be presenting some very interesting topics such as geophysical methods, laboratory studies, design considerations and case studies of geo-engineering projects. The field trip will take us to an aggregate quarry and construction projects around the Wilmington area.

We hope that you have time to explore our beautiful state, to visit our beaches, historic Wilmington, and to enjoy the local seafood and Carolina barbecue. So again, welcome, enjoy the Symposium, and we hope your experience on the Carolina coast is an enjoyable one.

The 56th Annual Highway Geology Symposium Host Committee.

Tommy Douglas
Russell Glass
David Hering
Bill Moore
Don Moore
John Pilipchuk
Brad Worley
Cheryl Youngblood

HIGHWAY GEOLOGY SYMPOSIUM

HISTORY, ORGANIZATION AND FUNCTION

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

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

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

Since the initial meeting, 52 consecutive annual meetings have been held in 32 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, West Virginia, Ohio, Maryland, North Carolina, Pennsylvania, Georgia, Florida and Tennessee serving as host state.

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

List of Highway Geology Symposium Meetings

No.	Year	HGS Location	No.	Year	HGS Location
1st	1950	Richmond, VA	2nd	1951	Richmond, VA
3rd	1952	Lexington, VA	4th	1953	Charleston, W VA
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX
15th	1964	Rolla, MO	16th	1965	Lexington, KY
17th	1966	Ames, IA	18th	1967	Lafayette, IN
19th	1968	Morgantown, WV	20th	1969	Urbana, IL
21st	1970	Lawrence, KS	22nd	1971	Norman, OK
23rd	1972	Old Point Comfort, VA	24th	1973	Sheridan, WY

25th	**1974**	**Raleigh, NC**	**26th**	**1975**	**Coeur d’Alene, ID**
27th	**1976**	**Orlando, FL**	**28th**	**1977**	**Rapid City, SD**
29th	**1978**	**Annapolis, MD**	**30th**	**1979**	**Portland, OR**
31st	**1980**	**Austin, TX**	**32nd**	**1981**	**Gatlinburg, TN**
33rd	**1982**	**Vail, CO**	**34th**	**1983**	**Stone Mountain, GA**
35th	**1984**	**San Jose, CA**	**36th**	**1985**	**Clarksville, IN**
37th	**1986**	**Helena, MT**	**38th**	**1987**	**Pittsburgh, PA**
39th	**1988**	**Park City, UT**	**40th**	**1989**	**Birmingham, AL**
41st	**1990**	**Albuquerque, NM**	**42nd**	**1991**	**Albany, NY**
43rd	**1992**	**Fayetteville, AR**	**44th**	**1993**	**Tampa, FL**
45th	**1994**	**Portland, OR**	**46th**	**1995**	**Charleston, WV**
47th	**1996**	**Cody, WY**	**48th**	**1997**	**Knoxville, TN**
49th	**1998**	**Prescott, AZ**	**50th**	**1999**	**Roanoke, VA**
51st	**2000**	**Seattle, WA**	**52nd**	**2001**	**Cumberland, MD**
53rd	**2002**	**San Luis Obispo, CA**	**54th**	**2003**	**Burlington, VT**
55th	**2004**	**Kansas City, MO**	**56th**	**2005**	**Wilmington, NC**

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee composed of approximately 20-25 engineering geologist 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 relatively relaxed overall functioning of the organization is what attracts many of the participants.

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

The symposia are generally for two and one-half days, with a day-and-a-half for technical papers and a full day field trip. The Symposium usually begins on Wednesday morning. The field trip is usually Thursday, followed by the annual banquet that evening. The final technical session generally ends by noon on Friday. In recent years this schedule has been modified to better accommodate climate conditions and tourism benefits.

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours.

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

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

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

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

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

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

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

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

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

56th ANNUAL
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WILMINGTON, NORTH CAROLINA
MAY 4 – 6, 2005

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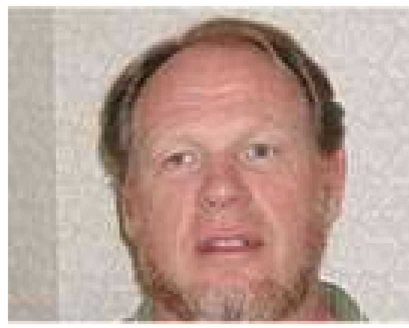

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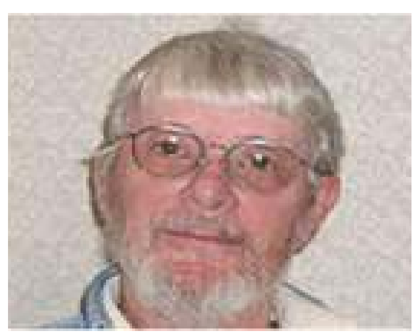

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Earl Wright
Ed J. Zeigler
Steve Sweeney

*****Deceased

HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

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

Hugh Chase*	**-**	**1970**
Tom Parrott*	**-**	**1970**
Paul Price*	**-**	**1970**
K.B. Woods*	**-**	**1971**
R.J. Edmonson*	**-**	**1972**
C.S. Mullin*	**-**	**1974**
A.C. Dodson*	**-**	**1975**
Burrell Whitlow*	**-**	**1978**
Bill Sherman	**-**	**1980**
Virgil Burgat*	**-**	**1981**
Henry Mathis	**-**	**1982**
David Royster*	**-**	**1982**
Terry West	**-**	**1983**
Dave Bingham	**-**	**1984**
Vernon Bump	**-**	**1986**
C.W. “Bill” Lovell	**-**	**1989**
Joseph A. Gutierrez	**-**	**1990**
Willard McCasland	**-**	**1990**
W.A. “Bill” Wisner	**-**	**1991**
David Mitchell	**-**	**1993**
Harry Moore	**-**	**1996**
Earl Wright	**-**	**1997**
Russell Glass	**-**	**1998**
Harry Ludowise	**-**	**2000**
Sam Thornton	**-**	**2000**
Bob Henthorne	**-**	**2004**

*Deceased

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Highway Geology Symposium

Future Symposia Schedule and Contact List

Year	State	Host Coordinator	Telephone Number	Email
2006	Colorado	Frank Harrison	(303) 980-0540	frank_harrison@golder.com
2007	Pennsylvania	Chris Ruppen	(724) 495-4079	cruppen@mbakercorp.com
2008	New Mexico	Erik Rorem	(505) 438-6161	erik.rorem@geobrugg.com
2009	New York	Mike Verling	(518) 471-4378	michael_vierling@thruway.state.ny.us

56th Annual Highway Geology Symposium
May 4 - 6, 2005
Hilton Riverside, Wilmington, North Carolina

GENERAL INFORMATION

On behalf of the North Carolina Department of Transportation, we welcome you to the 56th Annual Highway Geology Symposium. The symposium is scheduled for May 4th through the 6th at the Riverside Hilton Hotel in Wilmington, North Carolina.

The 56th Annual HGS, beginning on Wednesday, May 4, consists of a full day of technical presentations, a full day field trip, and concludes with a final half day of technical presentations on Friday, May 6.

TRB WORKSHOP:

The Symposium will be preceded on Tuesday, May 3rd by a 1/2 day Transportation Research Board (TRB) workshop. The workshop will consist of an afternoon technical session on “Aggregates for Highway Construction – Characterization and Performance”. This workshop is sponsored by committees AFP10 Engineering Geology, AFP20 Exploration and Classification of Earth Materials, and AFP70 Mineral Aggregates. The registration fee for the TRB workshop is $30.00 Please see registration form.

Guest Tour:

The day will begin with a trolley tour of the historic district of Wilmington. Then to Bellamy Mansion (www.bellamymansion.org), where guests will tour the circa 1859 mansion and gardens. Lunch will be served at Latimer House, home of the Lower Cape Fear Historical Society. Following lunch, guests will enjoy a walking tour of EUE / Screen Gems Studios, where Dawson’s Creek and One Tree Hill are currently filmed.

The cost for this tour is $45. Please see Registration Form to sign up.

56^{th} Annual
Highway Geology Symposium
Wilmington, North Carolina

Tuesday, May 3^{rd}, 2005

11:00 am-5:30 pm	TRB and HGS Registration
5:30 pm-7:30 pm	Welcome reception – Sponsor introductions – Visit with Exhibitors Student Poster Session

TRB Mid-Year Meeting Agenda

1:00 pm-3:00 pm Technical Presentations:

Aggregates for use in highway construction
Rick Meininger, FHWA

Geologic aspects of aggregate quality
Evan Franseen, Kansas Geological Survey

Slow gamma-ray logging of deposits for aggregate quality
Nelson Shaffer, Ned Bleuer, and Marni Dixon, Indiana Geological Survey

Measurement of particle shape, form, and texture characteristics
Eyad Masad, Texas A&M University

3:00 pm-3:30 pm **Break**

3:30 pm-5:00 pm Technical Presentations:

Development of the "Petrographic Examination" method for evaluation of aggregates
Fred Shrimer, Golder Associates

The micro-Deval test to assess aggregate quality
Stephen Senior, Ontario Ministry of Transportation (tentative)

Chemical reactions of aggregates
Stephen Lane, Virginia Transportation Research Council

Wednesday, May 4th 2005

7:00 am-12:00 pm HGS Registration

9:00 am-4:00 pm Guest Trip

6:00 pm-8:30 pm Diner Cruise

HGS Agenda

General Session
Room: Grand Ballroom

8:00 am-8:15 am *Welcoming Remarks*
Njoroge Wainaina, NCDOT
State Geotechnical Engineer
John Pilipchuk, NCDOT
56th Annual HGS Chairman

8:15 am-8:50 am *Geology of North Carolina*
Tyler Clark – Chief Geologist and
NC Geological Survey Manager

NOTE: Technical Session IA – Concurrent with Technical Session IB

Technical Session IA - Moderator: Kevin Miller – NCDOT
Room: Magnolia and Dogwood

9:00 am-9:20 am *Construction Monitoring, Sinking, and Inspection of Dredged Caisson Foundations US 82 Mississippi River Bridge, Greenville, Mississippi*
John F. Szturo, HNTB

9:20 am-9:40 am *Conceptual Designs and Cost Estimates: a Critical Step in Managing Unstable Slopes along Washington State Highways*
Steve Lowell Washington State DOT
William Gates, Kleinfelder
Lynn Moses, Washington State DOT
Chad Lukkarila, Kleinfelder
Brendan Fisher, Kleinfelder
Tom Badger, Washington State DOT
Norman Norrish , Wyllie & Norrish Rock Engineers

9:40 am-10:00 am *Highway US-34 Cut Slope Stabilization, Mt. Pleasant, Iowa*
Lok M. Sharma, P.E., Terracon Consultants, Inc.
Robert Stanley P.E., Iowa DOT

10:00 am-10:20 am **Break – Cape Fear Ballroom Exhibitor Area**

Technical Session IB - Moderator: Neil Roberson – NCDOT
Room: Camellia and Azalea

9:00 am-9:20 am *Stratigraphic Interpretations of Limestone, Geophysical Surveys, and Borehole Data Identify Potential Impact on Highway and Guide Future Quarry Expansion*
W. Burleigh Harris, University of North Carolina at Wilmington
Thomas J.Douglas, L.G., P.E., NCDOT

9:20 am-9:40 am *Rock Slope Stabilization, Decew #2 Generating Station, St Catharine's Ontario*
David F. Wood, David F. Wood Consulting Ltd. and Daniel Journeaux, Janod Contractors Ltd.

9:40 am-10:00 am *The Use of Graded Solid Rock for Rock Pad and Rock Embankment Construction along Highways in Karst Areas of East Tennessee*
Harry Moore, Tennessee DOT

10:00 am-10:20 am **Break – Cape Fear Ballroom Exhibitor Area**

NOTE: Technical Session IIA – Concurrent with Technical Session IIB

Technical Session IIA - Moderator: Dennis Li – NCDOT
Room: Magnolia and Dogwood

10:20 am-10:40 am *A Hybrid Rock Fall Protection System Along The Canadian Pacific Railway, Near Field B.C.*
A.J. Morris, P. Geol., Canadian Pacific Railway

10:40 am-11:00 am *3D Interpretations of Subsurface Soil and Rock Conditions*
Marc Fish New Hampshire DOT

11:00 am-11:20 am *Determining Soil and Rock Stiffness With MASW - Investigations of the 2004 I-40 Landslide and other Projects*
Ned Billington, L.G., Schnabel Engineering
David Hering, L.G., P.E., North Carolina DOT

11:20 am-11:40 am *Exhuming Rock Reinforcement*
Ken Fishman, McMahon & Mann Consulting Engineers, P.C.
Richard Lane, New Hampshire DOT
Andrew Salmaso, Janod Contractors Entrepreneur

11:40 am-12:00 pm *Condition Assessment of Thirty-Year Old Rock Reinforcements*
Ken Fishman, McMahon & Mann Consulting Engineers, P.C.
Richard Lane, New Hampshire DOT
Jim Bojarski, McMahon & Mann Consulting Engineers, P.C.

12:00 pm-1:10 pm **Lunch Buffet in the Grand Concourse and Ballroom**

Technical Session IIB - Moderator: Clint Little – NCDOT
Room: Camellia and Azalea

10:20 am-10:40 am *A Coarse Aggregate Paradox for Indiana Highway Pavements, Less is Better.*
Terry R. West, Purdue University
Joan O'Brien, Purdue University

10:40 am-11:00 am *Specification of Excavated Rock for Embankment Use*
Donald V. Gaffney, Michael Baker Jr., Inc.

11:00 am-11:20 am *Innovative Aggregate Resource Evaluations Using Electrical Resistivity Imaging*
J. Brant Gill, H.B.Sc., Golder Associates

11:20 am-11:40 am *Micro-Deval Abrasion Resistance of Aggregates and its Correlation with Performance in LA Abrasion and Sulfate Soundness Tests*
Prasad Rangaraju, Ph.D., P.E., Clemson University

11:40 am-12:00 pm *Determination of a Rock Bulking Factor for Highway Construction*
Stephen A. Senior, P. Eng., Ministry of Transportation Ontario

12:00 pm-1:10 pm **Lunch Buffet in the Grand Concourse and Ballroom**

Technical Session III - Moderator: Jody Kuhne – NCDOT
Room: Grand Ballroom

1:10 pmam-1:30 pm *Providing Structural Support and Reducing Long-Term Settlement in the Soft Silts and Clays Above the Cooper Marl. Ashley Phosphate Road and Route 52 Flyover, Charleston, SC*
Jeffrey J. Bean, P.E., Layne GeoConstruction
Robin Cheng, P.E. , Layne GeoConstruction

1:30 pm-1:50 pm *Repair of Voids Above Jack-and-Bore Pipeline Installations Under a Divided Highway*
Jeffrey R. Keaton, AMEC Earth & Environmental
Jeffrey Geraci, Moore & Tabor
Brian Stutzman, AMEC Infrastructure

1:50 pm-2:10 pm *"Too Little Too Late" or When To Include A Geologist In Highway Projects*
Albert Meijboom, Engineering Tectonics, P.A.
Barry Nelson, Engineering Tectonics, P.A.

2:10 pm-2:30 pm *Preliminary Findings on the September 16, 2004 Debris Flow at Peeks Creek, Macon County, North Carolina*
Rebecca S. Latham, North Carolina Geological Survey
Richard M. Wooten, North Carolina Geological Survey

2:30 pm-2:50 pm *An Overview of the North Carolina Geological Survey's Geologic Hazards Program – Phase I*
Richard M. Wooten, North Carolina Geological Survey
Jeffrey C. Reid, North Carolina Geological Survey
Rebecca S. Latham, North Carolina Geological Survey
Michael A. Medina, North Carolina Geological Survey
Randy Bectechtel, North Carolina Geological Survey
Timothy W. Clark, North Carolina Geological Survey

2:50 pm-3:10 pm **Break – Cape Fear Ballroom Exhibitor Area**

Technical Session IV- Moderator: Dean Argenbright – NCDOT
Room: Grand Ballroom

3:10 pm-3:30 pm *Landslide Investigation and Mitigation Along US 160 Between Durango and Mancos Colorado using Lightweight Fill, Ground Anchors, and Rockery Buttresses.*
Ben Arndt, P.E., P.G., Yeh and Associates
Richard Andrew, P.G., Yeh and Associates
Shan-Tai Yeh, P.E., Yeh and Associates

3:30 pm-3:50 pm *Ten Year Performance of a 400-foot High Rock Cut in Coal Measures Rocks*
James M. Sheahan, P.E. HDR
David L. Knott, P.E. HDR
Stanley L. Hite, P.E., Virginia DOT

3:50 pm-4:10 pm *Geotechnical Challenges Associated with US-59; Lawrence, KS to I-35 Near Ottawa, KS*
Carrie Denesha, MS, Kansas DOT
Robert Henthorne, PG, Kansas DOT

4:10 pm-4:30 pm *Geology, Landslides and Retaining Structures, on Arizona SR 89A, Jerome Arizona*
Nick Priznar, Arizona DOT
Paul Lindberg, Arizona DOT
J.J. Liu, Arizona DOT

4:30 pm-4:50 pm *Geophysics and Site Characterization K-18 over the Kansas River*
Neil M. Croxton, P.G., CPG, Kansas DOT

4:50 pm-5:10 pm *Geophysical Methods for Site Characterization of Offshore Highway Structures*
Richard E. Sylwester, Golder Associates

5:10 pm-5:40 pm Field Trip Overview

6:00 pm Henrietta III loads for the Cape Fear River Dinner Cruise. The cost for the cruise is $40.

Thursday, May 5th, 2005

8:30 am-5:00 pm Geology Field Trip - Wilmington Area

6:00 pm-7:00 pm Social Hour and Exhibits - Cash Bar

7:00 pm-10:00 pm Annual Banquet and Program
Guest Speaker – David Fischetti, P.E., DCF Engineering
“Moving the Cape Hatteras Lighthouse”

Friday, September 10th 2004

6:45 am-8:00 am Steering Committee Meeting

Technical Session VI - Moderator: Moderator: Lee Stone – NCDOT
Room: Grand Ballroom

8:00 am-8:20 am *Geotechnical Challenges of the Mon/Fayette Expressway Project, PA 51 TO I-376, Near Pittsburgh, Pennsylvania*
Lawrence J. Artman, II, P.G., HDR Engineering, Inc.
Kenneth M. Heirendt, P.G., Pennsylvania Turnpike Commission
Matthew L. McCahan, Pennsylvania Turnpike Commission

8:20 am-8:40 am *Performance of Flexible Debris Flow Barriers in Fire Burned Areas, State Route 18, San Bernardino County, CA*
Erik J. Rorem, Geobrugg North America, LLC

8:40 am-9:00 am *The Importance of Lateral Stress in Geotechnical Design ... but How Do We Measure It?*
Scott M. Mackiewicz, Ph.D., P.E., Kleinfelder and David J. White, Ph.D., Iowa State University

9:00 am-9:20 am *Peat Mapping Using Resistivity*
Paul Fisk, NDT Corp.
Keith Holster, NDT Corp.
Silas Nichols, FHWA
Peter Connors, Massachusetts Highway Department

9:20 am-9:40 am *Geotechnical Management Systems Where Do We Go From Here?*
Thomas E. Lefchik P.E., FHWA
Kirk Beach, Ohio DOT

9:40 am-10:00 am *MASW – From Detailed Investigations to Regional Surveys Along Roadways: Advantages and Limitations*
Lynn Yuhr, P.G., Technos Inc.

10:00 am-10:20 am **Break – Cape Fear Ballroom Exhibitor Area**

Technical Session VII - Moderator: Shane Clark – NCDOT
Room: Grand Ballroom

10:20 am-10:40 am *I-40 Slope Repairs in Western North Carolina*
Nilesh Surti, P.E., NCDOT

10:40 am-11:00 am *I-40 Toe Scour Protection System*
Joseph Bigger

11:00 am-11:20 am *Assessing the Potential Environmental Impact of Acid Rock Drainage (ARD) and Metal Leaching (ML) for the Sea to Sky Highway Improvement Project Between Vancouver and Whistler, British Columbia*
Stephen Barrett, Golder Associates Ltd
Rens Verburg, Golder Associates Inc
Valerie Bertrand, Golder Associates Ltd
Cheryl Ross, Golder Associates Ltd
Jeff Fillipone, Golder Associates Ltd
Dave Munday, Golder Associates Ltd.

11:20 am-11:40 am *Building The Case for Soft Solutions: Coastal Erosion and the 2004 Hurricane Season in Florida*
Rowland Atkins, M.Sc., P.Geo

11:40 am-12:00 pm *Geophysical Applications for Bridge Design, North Carolina Outer Banks: Results of Marine Seismic and Resistivity Investigations in the Pamlico Sound*
Ronald Crowson, Geo Solutions Limited
David Mallinson, East Carolina University
Ron Kaufman, Technos, Inc
Thomas V. Admay, ECS Limited, Inc.

12:00 pm Concluding Remarks – Adjournment

Construction Monitoring, Sinking, and Inspection of Dredged Caisson Foundations
US 82 Greenville, Mississippi River Bridge
Greenville, Mississippi – Lake Village, Arkansas

John F. Szturo R.G.
HNTB Corporation
715 Kirk Drive
Kansas City, Missouri 64105

56th Annual Highway Geology Symposium

May, 2005

Wilmington, North Carolina

Introduction

The new U.S. 82 bridge currently under construction over the Mississippi River near Greenville, Miss., will be the longest cable-stayed bridge on the Mississippi River and the second longest in the U.S. when completed in 2006. The new bridge will replace a 1940s structure that bears the dubious distinction of being the most struck bridge on the Mississippi River. The existing bridge was constructed in 1940 at a cost of $4.5 million while the new Bridge will cost over $300 million

The new bridge is located near Greenville on the Mississippi-Arkansas border. At this location the Mississippi River drains approximately 1/3 of the U.S. The 13,700-ft bridge includes a steel composite cable-stayed span of 1,378 ft providing approximately 65 ft of vertical clearance over the navigation channel.

The main span of the bridge is supported by two tall towers founded on dredged caisson foundations up to 140 feet below the river bottom. In comparison, the Pier 37 caisson was equal to constructing and sinking a 28 story building below the river surface. The two caissons cost over $30 million each and required 92,400 cubic yards of excavation. The caisson also required 70,400 yards of concrete. The contractor faced many challenges in constructing and sinking these massive foundations over the course of two years.

Vicinity Map

It has been determined that one of the primary reasons for the large number of collisions at the existing Greenville Bridge is its location relative to an upstream bend in the river. The towboat pilots refer to the bend not as a curve, but as a left turn against the current. The navigation opening is located on the east side of the channel (the left descending bank) and in order to transit the opening, the pilots must begin maneuvering far in advance of the bridge. With up to 1,500 ft of barge out front and one of the swiftest currents along the entire river, managing control of the vessel becomes difficult. As the operators attempt to line up for the bridge, the current drives them toward the Arkansas bank of the river and toward the westernmost main pier.

Perhaps one of the most complicated tasks of the design was the geotechnical investigation. The Greenville Bridge is located in one of the swiftest sections of the Mississippi River. Further complicating the investigation is the water depth, which varies from 60 to 120 ft at the location of the tower piers. In order to provide a stable drilling platform, an offshore, jack-up drilling platform was brought in from the Gulf of Mexico. Anticipating a large, open-dredged caisson foundation, four borings were taken at each tower pier location at the approximate locations of the foundation corners. Samples were taken and laboratory tests conducted to establish the allowable bearing capacity of the material.

As a result of the geotechnical investigation, it was confirmed that a dredged caisson foundation was the appropriate foundation type for this location. Dredged caissons are a type of foundation where the method of construction is as much a part of the design as the bearing capacity itself. Because of the depth of water, it was decided a "floating" caisson was be necessary. In this method, the contractor is required to progressively construct a large perforated concrete box in the river and keep it afloat until it is tall enough to rest on the river bottom and extend above the water surface. Once safely on the bottom of the river, the contractor may then begin to excavate the material from within the caisson in order to reduce friction and cause the caisson to "sink" to its final bearing elevation. Finally, the contractor may seal the bottom of the excavation and start up with his construction process.

GEOLOGIC SETTING

The project is located in the Mississippi Alluvial Valley, an extensive lowland extending from the boot heel of Missouri, southward 600 miles to the Gulf of Mexico. The region is a vast floodplain with valley width in the project area of approximately 80 miles. Ground surface elevations generally range from 100 to 130 feet above sea level. The constructed levee system rises approximately 30 feet above the local ground surface.

The alluvial deposits of the valley are of irregular thickness and are made of two units, a lower layer of sands and gravels (substratum), and an upper layer of soft clayey and silty beds (top stratum). The lower layer makes up most of the alluvial mass and occurs closer to the surface at the margins of the valley. The upper fine grained layer, is more unpredictable in material and thickness, having been reworked and replaced by the dynamics of the river during recent geologic time.

The alluvial plain is located in a great structural down warping called the Gulf Coast Geosyncline. Down warping is a result of the accumulation of the marine sediments forming the Gulf Coastal Plain.

Indurated sedimentary deposits of Tertiary Age (Eocene - 35 to 55 million years before present) form the floor of the entrenched valley under the alluvium at the project location. These deposits are characterized as deltaic marine clays with scattered beds of sand or gravel. Cementation by calcification may also rarely be found.

Geologic units underlying the alluvial deposits within the project area belong to the Claiborne Group that is made up of Eocene Age Cockfield Formation. The unweathered sediments of the Cockfield Formation are mostly homogenous fat clays interbedded with dense sands and widely scattered thin zones of lignite and carbonaceous fine grained material. The deposits are generally dark gray to brown in color and contain fossil shell fragments. The sediments exhibit a very low permeability and are considered to be very strong with a low compressibility.

Typical Undisturbed Sample of Cockfield Clay
Summary of Inspection

Caisson Construction, Sinking, and Inspection

Approval process
The general inspection of the caisson included; monitoring the position and geometry of the caisson during sinking, examining excavated material to match borings and design assumptions, continuous bottom soundings, witness of the jetting, witness of the airlift cleaning, viewing the sonar soundings, collaborating during the divers inspection and inspecting the seal placement.

Summary:

- Samples of the earth material above and at founding elevation were observed throughout the excavation and inspection process and deemed to meet the bearing requirements for the maximum load combination of 705 Kn/m² (7.3 tsf). The material examined during excavation was in general agreement with the boring logs included in the plans.

- The combination of witnessing the final jetting and airlifting, along with viewing the sonar images and review of diver inspection and communication, judged the bottom to meet the minimum area of 90 percent clean and sound with no more than 1 percent of debris in any one area.

- The bottom was generally, sound, level, and clean of loose material.

- Surfaces of the cutting edge, working chamber and caisson body were undamaged and clean. The perimeter cutting edge and lower divider edges were in contact with the bottom except for a few areas at Pier 38.

- Founding surface was generally equal to, or below the final elevation of the cutting edge.

Caisson Sinking Operations
Scour protection is the first order of construction with caisson sinking. Without some type of protection, as the caisson nears the river bottom, uncontrolled scour can be detrimental to beginning level and plumb construction. Past experience indicates as the caisson reaches the river bottom, scour in the order of tens of feet could occur.

Willow mattresses have traditionally been used for this purpose, however in this case, a unique partnership was formed with the US Army Corps of Engineers to provide and install articulated concrete mattresses typically used for bank protection and stabilization.

Installing Articulated Concrete Mattress Scour Protection

The sinking of the caissons began with float-in of the Pier 37 cutting edge. The beginning of a caisson begins with the cutting edge, prefabricated off-site and floated to the site. The cutting edge consists of a 12 feet high structural steel framework which the reinforced concrete caisson will be constructed upon. The wells in the cutting edge are fitted with steel caps to maintain buoyancy during float in and initial sinking.

A honeycomb like reinforced concrete structure is then constructed in various lifts on the cutting edge until it reaches the river bottom. The air domes are then removed by divers. Caisson construction and advancement then alternated between adding lifts of reinforced concrete and sinking. Typically 30 to forty feet of reinforced concrete is cast between periods of excavation. The addition of the weight of the concrete is necessary to overcome the side friction developed during sinking.

The method of excavation through and beneath the caisson employed 3 and 4 cubic yard clamming buckets from three cranes positioned on the east, west, and downstream sides of the caisson.

Float-in of Cutting Edge

Approximately 21 meters (69 feet) at Pier 37 and 13 meters (43 feet) at Pier 38 of alluvial sand and gravel were excavated before encountering the Tertiary Age Cockfield Formation at approximately elevation -9.0 meters (-29.5 feet) below sea level. The formation consists predominately of stiff, gray, slightly silty clay with layers, lenses and mixtures of silty sand, sandy silt, clayey silt, and cemented layers. The caissons were advanced through the Cockfield formation to founding elevation primarily by clamming, with some jetting.

After clamming the excavation to near the founding elevation, jetting of material between the dredge wells and along the deep cutting edges was employed to advance the caisson to final grade and generally level the bottom.

The Pier 37 caisson reached the final founding grade 1 year and 4 months after float in at an elevation of -31.9 meters (-104.7) on January 16, 2004, 0.20 meters (0.7’) below the planned founding elevation. Pier 38 reached its founding grade 1 year and 5 months after float in at a depth of -23 meters (-75.5). Pier 38 founding elevation was 30 feet higher but took one month longer to obtain.

Challenges During Construction

The contractor had difficulty advancing the Pier 38 caisson near elevation -18.9 meters (-62 feet). The cutting edge remained at this elevation while the clamming undercut the cutting edge to elevation -23 meters (-75 feet) or slightly below. It was not known if the caisson was friction bound or stopped on hard material at the edges or between the dredge wells.

In an attempt to remove the material standing between the dredge wells which the caisson was resting on blasting was considered. It was learned little information existed on construction blasting at water depths of nearly 200 feet. Blasting consultants were called on to perform analysis on the possibility of damage to the caisson and the effectiveness of the blasting as a means of dislodging the material.

A test blast was conducted by the contractor utilizing Dynagel as the explosive agent in an attempt to move the unexcavated material below and between the dredge wells. Test charge bundles of 2, 4, 8, 12 16, 24, and 32 pounds were lowered to near the bottom of the excavation approximately 4.5 meters (15 feet) below the cutting edge elevation in dredge well number 14 and detonated. The charges were placed in the water adjacent to the unexcavated material. Soundings of the bottom and elevations of the caisson were taken before and after the blasts. No material had apparently been loosened nor had the vibrations helped to "shake" the caisson down.

Subsequent sonar surveys on January 15, 2004 revealed the test blasting did not successfully move any material. The surveys also indicated the previous clamming below the interior cutting edges was off-center of the dredge wells toward the outbound sides of the caisson in nearly all cases.

As a result of the sonar findings, the contractor elected to "chop" the sides of the excavations with open clam buckets. A considerable amount of material was observed to be removed by this process.

After clamming the excavation to near the founding elevation, the caisson was not advancing. Jetting of material between the dredge wells as well as under and along the deep cutting edges was employed to attempt advancing the caisson to founding grade. On February 16, 2004, the caisson suddenly dropped 3.6 meters (12 feet) to within 1 meter (2 feet) of founding elevation. Again on March 13, 2004 the caisson descended 0.55 meters (1.8 feet) to the final founding grade of elevation -23.0 meters (-75.5) equal to plan grade.

After the drop and while completing the final jetting, a sand and gravel mixture, similar to the upper alluvial material between elevation - 6 meters (-20 feet) and - 30 meters (-98 feet) partially filled the dredge wells along the west and southwest lower side of the caisson side (dredge wells 4, 5, 6, 7). Further work ceased until the source of the "blow in" was determined.

Divers were called in to examined the connection of the temporary sheet pile follower cofferdam and the permanent concrete caisson body and determine if the connection had been breeched or sheets pulled apart allowing the sand and gravel to enter the dredge wells. The divers did not find a breech in the sheets and found them to all be intact and seated in the lower steel beam. .

The inflow of sand and gravel material was clammed out of dredge wells 4 and 6 and sound foundation material found below the sand and gravel material, thus, the granular material was not thought to part of the Tertiary Cockfield Formation. It was then determined the eastern four sets of 6 dredge wells (7 through 24) would be airlifted clean and have the seal concrete placed before the west 6 dredge wells (1 through 6) were airlifted.

The caisson excavation remained stable during the airlifting and seal placement in the east three fourths (dredge wells 7 through 24) of the caisson. The west 6 dredge wells (1 through 6) were then airlifted, and seal concrete immediately placed. The bulk of the sand was removed by clamming in the west 6 dredge wells. During the clamming the bottom of excavation was confirmed as hard silty clay of the Cockfield Formation by retaining and examining samples from the clam buckets.

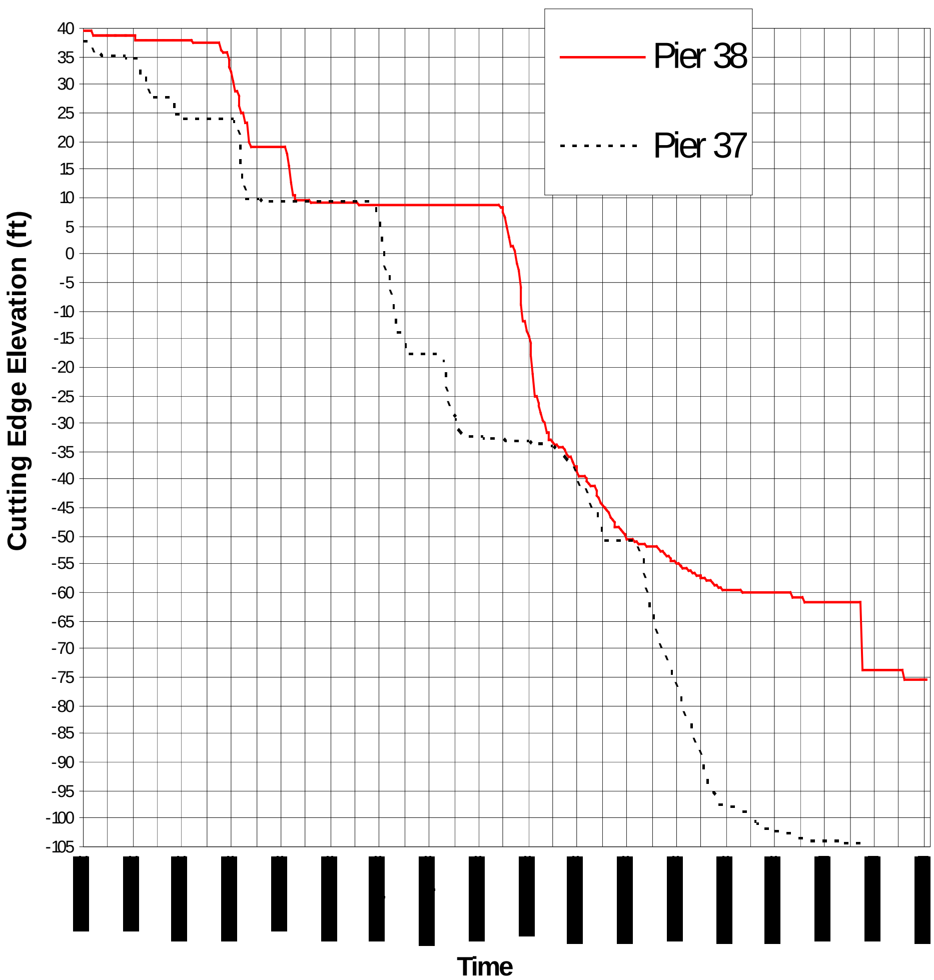
The most likely source of the “blow in” was later determined during the final dive inspection as divers reported small to large gaps under the deep interior cutting edges. The path or cause of the inflow of sand most likely was caused by the deep undercut and gaps at the outside cutting edge and deep interior walls

Clamming in between lifts of concrete at Pier 37

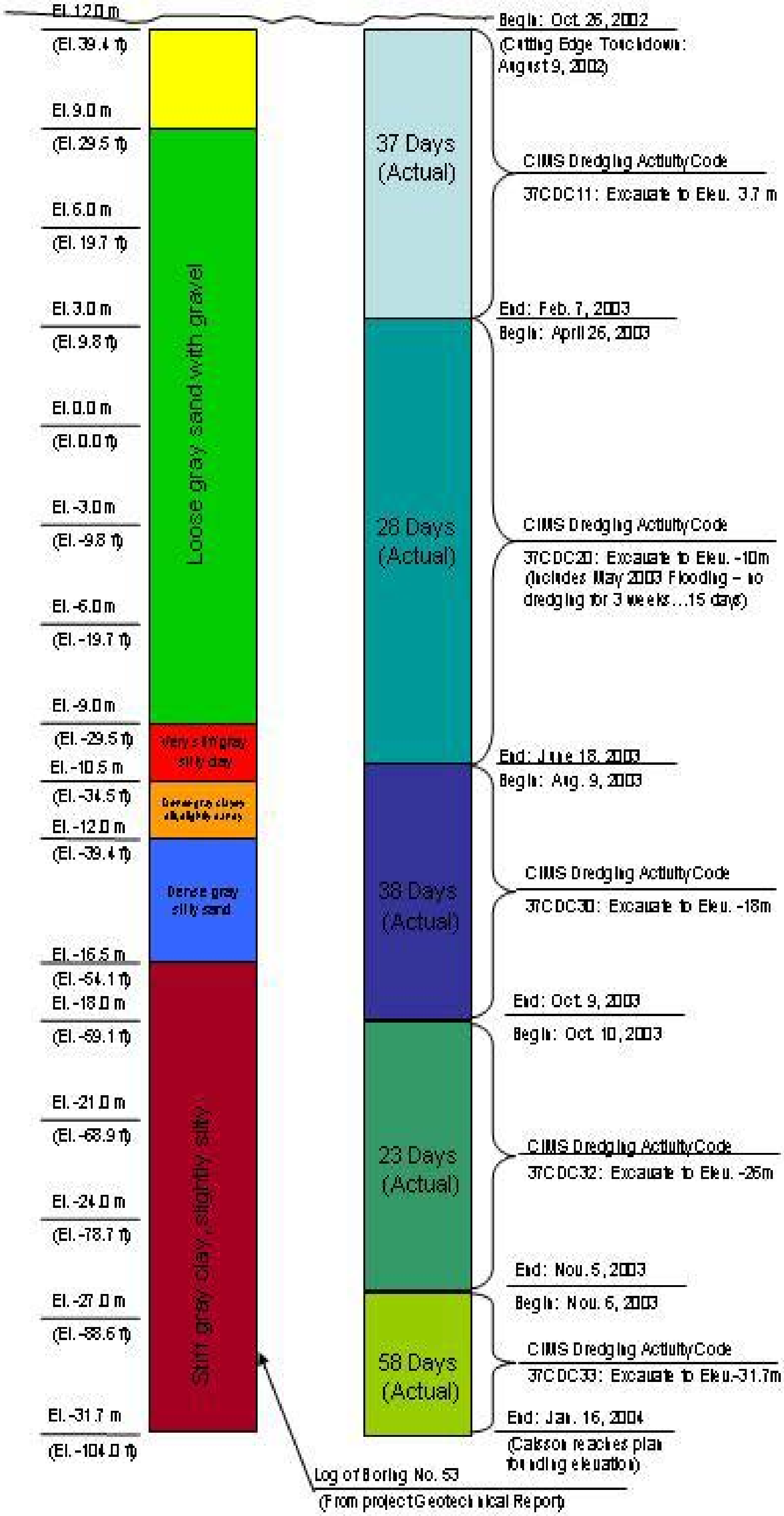
New US 82 Greenville Bridge

Dredged Caissons at Pier 37 and Pier 38

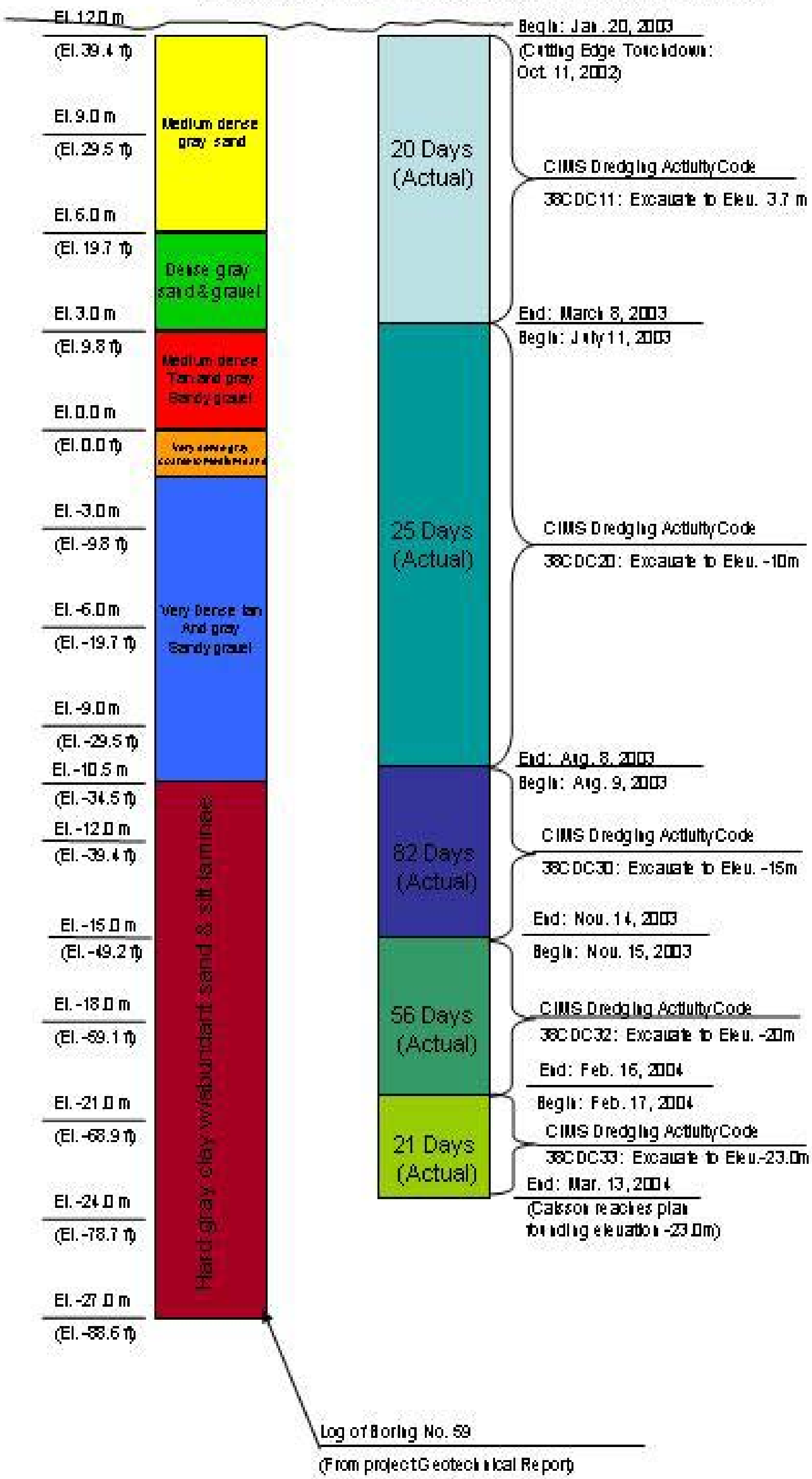
"CAISSON SINKING PROGRESS"



Dredged Caisson Excavation Progress – Pier 37



Dredged Caisson Excavation Progress – Pier 38



Final Jetting
Jetting by high pressure water was employed as the caissons neared plan founding elevation. The jetting completed the excavation under the interior cutting edges and generally leveled the bottom.

The jetting operation generally employed 2 – 10 hour shifts per day. The operation consisted of two jets, one 10 – inch diameter and one 12 – inch diameter. Both jets were fed high pressure water from two Conmaco model 5TUT-16 pumps for each jet (four total). Two flexible hoses fed each jet. Right angle bends and extensions at the bottom of each pipe were reduced to one – inch nozzles at the end of each jet. Pressure readings of 280 psi were normal at the pump discharge. Contractor personnel rotated the jets at the top of the follower caisson near the cutting edge elevation until no obstructions were felt.

Airlifting
After completion of jetting, final clean-out was accomplished using two airlifts consisting of one 14 – inch diameter and one 24 – inch diameter pipe powered with compressed air. Airlifts work as vacuums to clean the bottom of mud, loose and loose material. The airlifts are powerful enough in sandy laminated material to "peel" and excavate material. The ends of both airlifts were angled to reach the areas between the open dredge wells in the working chamber.

The airlift was directed to cover the entire bottom area of the caisson and was accomplished until clean water was visible. As one step of verifying cleanliness of the founding surface, the airlifting was witnessed on a full-time basis by inspection personnel. The area was deemed clean when the water ran clear.

1" Jet

Operating Jet in Dredge Well

Offset 24-inch Airlift Foot

Sonar Inspection
The Engineers plans and specifications had designated the final inspection of the caisson as to cleanliness, level, and general bottom condition be performed by television camera. Television was to provide the bulk of the inspection due to the deep water conditions necessitating deep water diving. Spot dive inspection would confirm the TV and check items of concern discovered by TV.

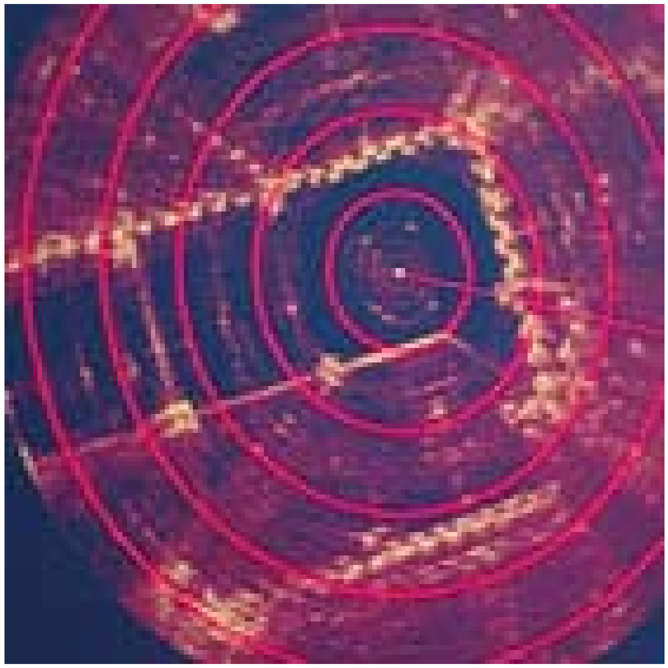
The contractor desired to begin inspection as soon as possible after airlifting and elected not to attempt to clear the water or allow it to settle for adequate clarity for use of a television camera.

As an approved alternative to the television camera inspection, the interior and bottom of the caisson were examined using sonar methods. This method had never been used to inspect a caisson and consisted of technology used by the offshore petroleum industry to inspect drill platforms and locate pipelines. After a satisfactory onsite demonstration, the sonar was approved for use.

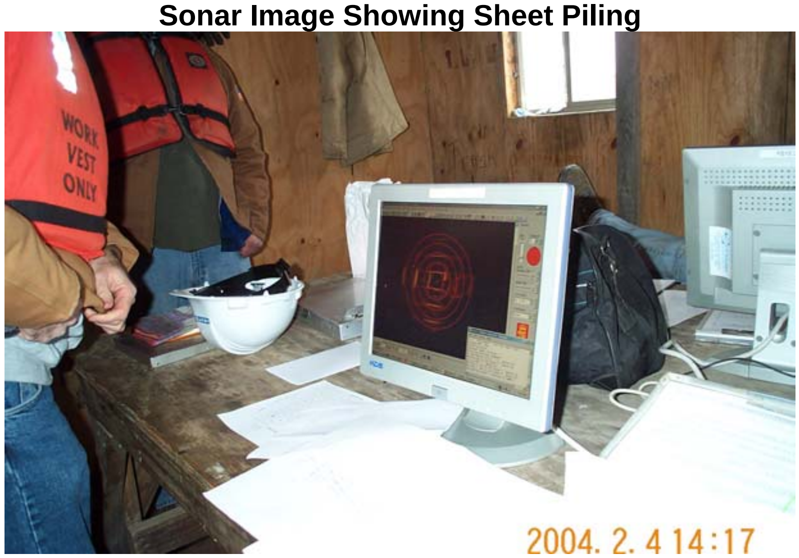
The sonar was furnished and operated by C & C Technologies of Lafayette, Louisiana and consisted of a Simrad MS 1000, 360° scanning sonar. The sonar had to be held level and at known elevations. Subsequently, the sonar was lowered through the dredge wells with dual crane lines or dual winches to predetermined elevations near the caisson bottom and working chamber.

The sonar was viewed and notes recorded in real time by inspection personnel. The live sonar image (movie) was recorded as an audio-video file. Digital snapshots of the sonar were also taken and recorded. The sonar was found to be an effective tool in the inspection process able to indicate small detail.

Simrad MS 1000 Sonar Head



Sonar Image Showing Sheet Piling



Real Time View of Sonar Image

Dive Inspection

After airlift, jetting, and sonar observation, noted areas were designated for dive inspection. The inspection was performed by Onyx Special Services Inc. of Appleton, WI. Prior to inspection a pre-dive meeting was held with the dive crew and the owners inspection staff. Inspection personnel also relayed pertinent information to the divers before each dive, monitored the diver communications and held post-dive interviews.

Due to the depth of water, mixed gas was used by the divers. The divers also used a heated water system to maintain body temperature throughout the dive. Umbilical cords containing a safety line, hot water, cable for television and two way communications were attached to each diver. Total bottom time for each diver was 30 minutes. The divers were required to decompress in a chamber for several hours after each dive.

The divers used a helmet mounted camera to record the dive although visibility was 6 inches or less. For the most part the dives were considered black water, zero visibility dives. Video tape was used to record both the camera and dive communications.

Divers performed a general inspection of the caisson for cleanliness, bottom topography, soundness of founding material, amount of loose debris, and amount of material remaining along cutting edges. General dimensions of items were taken by the divers and "pneumo" readings taken which provide depths below water. These reading were used to provide topography and dimensions.

Dive Communications and Control Center

Diver Prepared to Enter Pier 38 Caisson

Seal Concrete Placement

Seal concrete was placed using standard methods for underwater concrete placement. The contractor utilized "rabbit" plugs or steel plates and rubber gaskets at the end of the tremie covered with plastic bags and taped to seal out water. The 12" diameter tremie was then lowered to the bottom charged with concrete. Once charged, the tremie was lifted slightly to break the seal and release the concrete. The end of the tremie was constantly maintained a sufficient depth below the surface of the fresh concrete.

Concrete was furnished to the caisson by transit trucks unloaded at the dock into four - 4 cubic yard buckets on barges. Tugs brought the barges to the caisson where the buckets where hoisted to the tremies by cranes and dumped.

A total of 7459 cubic yards of concrete was placed for the Pier 37 caisson seal, or 272 cubic yards more than the estimated plan quantity of 7188 cubic yards. The extra concrete was due to the undercut of the founding surface to bring the caisson to plan elevation. The seal placement took 5 days averaging just under 1500 yards placed per day.

A total of 7418 cubic yards of concrete was placed for the Pier 38 caisson, seal or 230 cubic yards more that the estimated plan quantity of 7188 cubic yard. The seal placement took 6 days.

Placement of Seal Concrete

Acknowledgements

Owners
Mississippi Department of Transportation

Contractor
Massman-Traylor Brothers Joint Venture

Subconsultants
C & C Technologies
Onyx Dive Services

Conceptual Designs And Cost Estimates: A Critical Step In Managing Unstable Slopes Along Washington State Highways

By

Steve Lowell[1], William Gates[2], Lynn Moses[3], Chad Lukkarila[4], Brendan Fisher[5], Tom Badger[6], Norman Norrish[7]

ABSTRACT

The Washington State Department of Transportation (WSDOT) has developed a proactive and rational approach for identifying, categorizing, and prioritizing for mitigating unstable slopes along their 6,835-mile highway system. Early evaluation and scoping of unstable slopes as it relates to the geologic problem, mitigation and cost issues are critical in planning, budgeting and prioritizing unstable slopes for mitigation. In 1993, WSDOT developed the Unstable Slope Management System (USMS) to address known slope hazards adjacent to WSDOT's highway system. The objectives of the program are to (1) rationally evaluate more than 2500 unstable slopes, (2) perform early scoping, conceptual designs and cost estimation, (3) conduct cost-benefit analysis of unstable slopes, and, (4) prioritize the mitigation of known unstable slopes according to the expected benefits. Utilizing a matrix-based numerical rating system, the USMS includes not only rockfall hazards but also landslide, settlement, and erosion problems. Most of the 2,500 unstable slopes have been rated and entered into the USMS. Presently WSDOT Geotechnical Division and their consultants are performing early scoping of the slopes by developing conceptual designs and preliminary cost estimates for mitigation. Because of the large number of sites, WSDOT based prioritization of the slopes for evaluation on (1) highway functional class, (2) USMS numerical rating, and (3) average daily traffic (ADT). Senior-level engineering geologists field inspect the unstable slope and develop a problem definition and conceptual design for each slope. Each unstable slope conceptual design slated for mitigation includes location, problem definition, problem correction and cost estimate supported by detailed field notes. Once the conceptual design has been completed, cost estimates for the work are developed. Unit costs are based on average construction bid tabulations for similar type work in the last five years. The conceptual designs and cost estimates to mitigate the slope hazard are then entered into the USMS and become a permanent record in the database. The WSDOT Geotechnical Division uses the cost estimate for each unstable slope in the cost-benefit analysis for mitigation of the slope hazard. Cost-benefit analysis for slope stabilization considers anticipated cost of traffic impacts resulting from a slope failure and the annual maintenance costs over 20 years divided by the cost to mitigate the slope hazard. Because of limited funding, only those slopes with cost-benefit ratios above 1.0 are considered for mitigation. Since the USMS program has begun, WSDOT can demonstrate accurate and conservative results between the conceptual design, engineer's estimate and the contractor's low bid. Typically, cost estimates in the conceptual design have been accurate and conservative, and higher than the Engineer's estimate and the Contactors low bid. The results of the USMS are a rational and proactive program for mitigating geologic hazards along the Washington State highway system.

INTRODUCTION

In the mid 1990's, a new capital improvement project programming approach was implemented for WSDOT's highway construction program. This new approach involved prioritizing and programming projects (priority programming) based on the extent which they addressed highway deficiencies along WSDOT's highway system. One of the deficiencies identified for programming in the highway preservation program is the proactive stabilization of known unstable slopes. The funding level for this unstable slope program was set at $300 million dollars over a 20-year program life (Lowell and Morin, 2000).

WSDOT has internally developed a comprehensive management system that would address the goals of the priority program approach and would:

- Rationally evaluate all known unstable slopes along WSDOT's highway facilities utilizing a numerical rating system developed by WSDOT that rates both soil and rock instabilities.

- Develop an unstable slope ranking strategy, based on highway functional class, which would address highway facilities with the greatest needs.

- Provide for early unstable slope project scoping, conceptual designs, and cost estimates that could be used for cost benefit analysis.

- Prioritize the design and mitigation of unstable slope projects, statewide, based on the expected benefit.

NUMERICAL RATINGS OF UNSTABLE SLOPES

To accurately prioritize individual slopes within the statewide inventory, a wide variety of unstable slopes must be rated in a systematic manner based on consistent and measurable criteria. WSDOT developed a numerical slope rating system that evaluates risk factors assigned to the highway facility modeled after similar hazard rating systems (Wyllie, 1987; Pierson et al., 1990). WSDOT's numerical rating system (Figure 1) is unique in that it considers both soil and rock instabilities within the same matrix, and the numerical ratings are consistent for both types of unstable slopes. WSDOT's numerical rating system addresses the type and severity of slope hazard or failure in only one rating category while the remaining categories are dedicated to establishing risk factors to the highway facility (WSDOT, 1995). This numerical rating system assigns points, varying from 3 to 81, to eleven risk categories, and the exponential scoring system quickly distinguishes increasing importance or hazard potential. The higher the numerical rating for an individual slope generally relates to higher overall risk to the highway facility.

Unstable Slope Rating Form

WSDOT, Geotechnical Services, Unstable Slope Management Unit

REGION	
SR	
BEG MP	
END MP	
Side, *L for Left* *R for Right or L,R*	
Functional Class	

RATED BY (initial)	
DATE	

SPEED, *posted (mph)*	
SIGHT DISTANCE, *estimated (ft)*	
DECISION SIGHT DISTANCE, *(ft)*	

RATING CRITERIA *(place an "X" to select a criteria from each catergory)*

CATEGORY		3	9	27	81	POINTS
PROBLEM TYPE	SOIL or	CUT or FILL SLOPE EROSION	SETTLEMENT or PIPING	SLOW MOVING LANDSLIDES	RAPID LANDSLIDES or DEBRIS FLOWS	
	ROCK *Rockfall / Catchment*	MINOR / GOOD	MODERATE / FAIR	MAJOR / LIMITED	MAJOR / NONE	
ADT avg daily traffic	*fill in value*	<5K	5-20K	20-40K	>40K	
Truck ADT	*fill in value*					
PDSD % of decision sight dist.		ADEQUATE 100%+	MODERATE 80-99%	LIMITED 60-79%	VERY LIMITED <60%	
IMPACT of FAILURE On ROADWAY	*fill in value (ft)*	< 50'	50' - 200'	200' - 500'	> 500'	
ROADWAY IMPEDENCE		SHOULDER ONLY	1/2 Roadway	3/4 Roadway	FULL roadway	
AVERAGE VEHICLE RISK		<25%	25-50%	50-75%	>75%	
PAVEMENT DAMAGE		MINOR Not Noticeable	MODERATE Driver Must Slow	SEVERE Driver Must Stop	EXTREME Not Traversible	
FAILURE FREQUENCY		0/5 YR	1/5 YR	1/YR	1+/YR	
MAINTENANCE COSTS ($/year)		< 5000	5-10K	10-50K	>50K	
ECONOMIC FACTOR detours		NO Detour Required	SHORT Detours, < 3mi	LONG Detours Detours, > 3mi	SOLE ACCESS No Detours	
ACCIDENTS (in last 10 years)	*fill in value*	0 TO 1	2 TO 3	4 TO 5	>5	
TOTAL POINTS						

Figure 1. USMS rating form includes both hazard and risk factors to evaluate and numerically rank unstable slopes (WSDOT, 1995).

A primary goal of priority programming is to address transportation deficiencies or needs in those areas that have the highest investment. Early in the development of the USMS, it was recognized that the *"worst first"* approach by total inventory would not maximize the investment of limited construction dollars. To ensure, to the greatest extent possible, that construction dollars were being spent in those areas that had the highest return on the investment, the unstable slope inventory was grouped based on highway functional class. Under this programming scenario unstable slopes along interstate, and principle arterials are being mitigated first, followed by lower volume facilities. Within each highway functional class, the slopes are ranked in descending numerical order, so that the highest risk slopes within the functional class are considered first. Based on this ranked list of unstable slopes problem definitions, conceptual mitigation design, and cost estimates are developed.

CONCEPTUAL DESIGN PROCESS

As part of the process, WSDOT senior-level geotechnical staff or their consultants conducts a field review of the unstable slope to collect information associated with the slope problem and to develop a conceptual slope mitigation design. The Conceptual Design process includes a detailed problem definition of the unstable slope, a conceptual design for mitigation, and the geotechnical cost estimates for the conceptual mitigation.

Problem Definitions

Defining the geotechnical problem for each unstable slope is critical in the USMS. Figures 2a and 2b are examples of WSDOT's consultant team (Kleinfelder) defining some unstable slope problems along SR 97 near Blewett Pass, Washington. As part of the process, WSDOT incorporates field information and other pertinent data associated with the slope problem to develop a conceptual slope mitigation design. Figure 3 is an example of the level of detail required in the field notes that support the conceptual design.

Figure 2a: Kleinfelder team employing a laser range finder to define the size of the unstable slope problem on SR 97 near Blewett Pass, Washington.

Figure 2b: Kleinfelder team mapping fractures on a rock face on SR 97 near Ruby Creek.

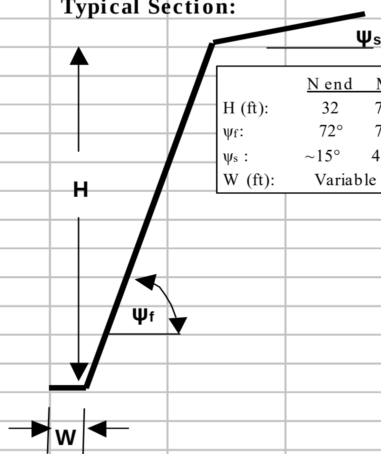
USMS - FIELD NOTES
REGION: **NW** SR: **011**
MP: BEG **11.3R** END **11.45R**
Slope#: **2575**

Prepared by: N. Norrish, P.E.
Date: January 13, 2005

Site Measurements:

Impact of Failure: 640 ft slope length
Sight Distance:
S bound N end: 265ft
N bound N end: 180ft
N bound middle: 205ft
N bound S end: 220ft
Highway Trend: 342° N end 297° S end

Typical Section:



	N end	Mid	S end
H (ft):	32	71	87
ψ_f:	72°	73°	60°
ψ_s :	~15°	46°	36°
W (ft):	Variable 1 to 4		

Rock Mass Characterization:

Rock Type: Fresh to slightly weathered, fine grained, tan brown, medium strong sandstone (2 - 10 ft beds) interbedded with slightly weathered, dark gray to black, weak to very weak, locally carbonaceous shale/siltstone (1 - 2 ft beds)

Structures: Beds 35°/353° planar, smooth, clean 30' + (S end)
Joints 62°/209° planar, rough, clean 10'
Beds 34°/015° planar, rough, clean 30' + (N end)
Joint 75°/232° curved, rough, clean 20' (N end - defines slabs)

Site Photographs: 11/4/04

North end of rockslope

South end of rockslope

Types of Instability:

1. Differential weathering.
2. Rockfalls controlled by joints in sandstone wher overhangs created by weathering.
3. Planar

Special Notes:

1. Approximately 90 ft of impact length at S end has been stabilized. (rock bolts, drain holes, shotcrete)
2. Masonry wall on outboard side at mid cut with guard rail on top of wall.

Mitigation Alternatives:

1. Re-slope to improve alignment. Sandstone is favorable for controlled blasting. Shale layers will require shotcrete protection to minimize weathering.
2. Remove trees from slope face and crest, scale, spot rock bolts, shotcrete shale layers, slope drape.

Figure 3: Example of detailed field notes adapted to a Microsoft Excel file developed by Norm Norrish with Wyllie & Norrish Rock Engineers. At each unstable slope, at least two photos displaying both approaches and the slope are obtained and uploaded to the USMS record for that site.

Depending on the complexity of the problem, the field review may take several hours per site, typically, WSDOT budgets about four hours per site. Typical information collected during the field review includes:

- Problem type – Verify the problem type identified in the unstable slope rating. Is the slope a soil or rock slope problem? If it is a soil slope problem? Is it a slow-moving landslide or fast-moving debris flow? If it is a rockslope problem, is it a case of erosion and raveling or major rockfall?

- Problem extent and size – Outline in detail the extent of the problem area, including slope length, height, inclination of slope face and backslope, and impact on the highway facility etcetera.

- Identification of the factors that are the major contributing factors to the slope instability; for example, weak foundation soils, inadequate drainage, structurally controlled rockmass, etc.

- Catchment area adequacy – If the unstable slope is a rock slope, identify if the existing catchment area is adequate based on the slope height, inclination and rock block size. The ODOT/FHWA "Rockfall Catchment Area Design Guide," (Pierson, et al, 2001) is often used to evaluate the adequacy of the catchment area. Figure 4 is an extreme example where the catchment ditch was inadequate to contain a large rockslide along SR 20 near Falls Creek in the North Cascades, Washington.

Figure 4: Rockslide at Falls Creek overtopping catchment area and highway along SR 20 North Cascades, Washington.

- Maintenance concerns and slope history – Maintenance personnel often have worked in the same area many years. Coordination with the local maintenance crews to discuss the slope stability problem that has occurred at the site is invaluable. Maintenance personnel often know the failure frequency of the slope, the extent of the clean up, and the estimated annual cost for maintenance.

- Identification of environmental constraints at the slope location that may influence the conceptual design and/or the design's feasibility. These could include endangered species, wetlands, anadromous fish windows, etc.

Conceptual Designs

By defining the slope problem in detail, an appropriate conceptual slope mitigation design can be developed. Mitigation strategies focus on avoidance, containment or stabilization. The conceptual design supported by the field notes outlines the mitigation strategies for the unstable slope. Based on the extent of the unstable slope problem, quantities of engineering materials (e.g. landslide buttresses, debris flow barriers, rock bolts, wire mesh slope protection, retaining walls, etc) can be estimated. The key components of the conceptual design include a geotechnical investigation and design, the mitigation/stabilization strategy, and other concerns (such as traffic control), during construction.

The conceptual design recommendations include a cost estimate for the geotechnical investigation and the geotechnical stabilization elements of the conceptual design. The costing factors are based on actual bid histories that have been compiled by the WSDOT Geotechnical Division.

Figure 5 is a photo from WSDOT's SR View (an external web-based video log of Washington State highways) displaying an unstable rockslope located on SR 2 at MP 91.38 in Tumwater Canyon, Washington. Figure 6 is an example of the conceptual design developed for this unstable rockslope after it was uploaded to the USMS database.

Figure 5: Photo from WSDOT SR View at MP 91.38 SR 2 displaying location of conceptual design (WSDOT, SRweb, 2002)

Cost Estimating

The conceptual design and the geotechnical estimating factors are entered into the WSDOT USMS database where the WSDOT regional program managers can access this information and complete the project cost estimate. Additional costs that are not necessarily associated with the geotechnical aspects of the slope mitigation/stabilization are incorporated into the estimate for each individual unstable slope. The Regions consider the following items:

- Conceptual design geotechnical recommendation costs
- Mobilization
- Traffic control
- Right-of-way
- Surfacing and paving
- Preliminary engineering

Unstable Slopes

Conceptual Design

Slope Last Updated 07/25/2001

Slope Inventory Data					
Region	**State Route**	**Begin Mile Post**	**End Mile Post**	**Side**	**Posted Speed**
North Central	002	91.200	91.500	L	50
Functional Class	**Maint. Area**	**Maint.Section**	**County**	**Problem**	**Numerical Rating**
2	1	2	Chelan	Rockfall	552

Problem Definition

Western portion consists of a slope up 80 feet high containing bouldery colluvium. Eastern portion consists of 60 to more than 100 foot high slope with up to 20 feet of bouldery colluvium overlying bedrock. Raveling of the bouldery overburden as well as planar and wedge type failures from the bedrock portion are the primary rockfall failure modes. Block size and slope configuration limit the ditch effectiveness in containing rockfall. Eastern portion of section has been partially remediated due to a December 1996 failure.

Problem Correction

Slope stabilization will require scaling, installation of rock bolts/dowels, and placement of both wire mesh and modified cable net. Scaling will require full roadway closures for short duration, work windows. Crane support for bolt/dowel installation will likely not be possible and still provide a single lane for traffic. The following work items and estimated quantities have been identified:

Estimating Factors

Geotechnical field exploration and design				30000.00
Debris Cleanup	1000	Cubic Yards	30.00 /cu.yd.	30000.00
Modified Cable Net	30000	Square Feet	8.00 /sq.ft.	240000.00
Rock Bolts/Dowels	600	Linear Feet	100.00 /lin.ft.	60000.00
Slope Scaling	200	Hours	300.00 /hr.	60000.00
Wire Mesh (w/anchors)	15000	Square Feet	4.00 /sq.ft.	60000.00
			TOTAL	$ 480000.00

Figure 6: Conceptual design for an unstable slope along SR 2 in the Tumwater Canyon, Washington that has been uploaded to the USMS database (WSDOT, 2001).

- Construction engineering
- Sales tax
- Contingencies

With the addition of these costing factors the Region completes a scoping estimate for each identified unstable slope, and enters the estimate into the USMS. Once entered into the USMS, the Geotechnical Division completes the process by conducting a benefit cost analysis. Figure 7 is an example of a scoping estimate developed by the North Central Region for the unstable rockslope based on the unstable slope reflected on Figure 5 and the conceptual design described on Figure 6 along SR 2, MP 91.2 to 91.5 in Tumwater Canyon, Washington.

BENEFIT COST ANALYSIS

The two most reliable and easily determinable impacts resulting from a slope failure along a highway facility are the cost associated with traffic delays, and the annual maintenance costs factored over the life of the program (20 years). Several simplifying assumptions were made to estimate costs associated with traffic delays. First, based on experience, a typical traffic delay in the event of a slope failure was assumed to be 24 hours. Secondly, a factor needed to be considered in terms of the amount of the roadway that would be impacted, since this has a bearing on traffic flow through the area. The roadway impedance rating addresses this factor and then applies reduction factors in the calculation of traffic delay costs. For example, if the roadway impedance rating indicated that only the highway shoulder would be impacted, then only 25 % of the total calculated traffic delay cost is used. If the roadway impedance rating indicated that all lanes of the highway facility would be impacted, then the total cost of the delay (100%) was used. Similar reduction factors were developed for other roadway impedance ratings. Life-cycle maintenance costs are established based on the estimated annual costs that have been generated by the regional maintenance personnel, and multiplied by the 20-year program life. These two factors (traffic delay and maintenance costs) are evaluated against the cost of mitigating the unstable slope to establish a benefit-cost ratio. In special cases, consideration of other known and quantifiable economic impact costs can be included in the benefit-cost analysis. Typically, this is done for lower volume highway facilities or high cost slope mitigations, where the ramifications of a slope failure can have severe socio-economic impacts. Based on the benefit-cost analysis, the unstable slopes are sorted by descending benefit-cost to form a prioritized list of slope stabilization projects. Because of funding limitations, only those unstable slopes that have a benefit-cost ratio of one or larger are considered for funding within the Unstable Slope program.

Washington State Department of Transportation

NORTH CENTRAL REGION SCOPING ESTIMATE

Prog. Item No.	Work Item No.	SR No.	From SR MP	To SR MP	Date/cost index	Date
		2	91.20	91.50		
Functional Class NHS						
Project Descript.			Nature of Work **Slope stabilization**			

I. RIGHT OF WAY				$0
II.CONSTRUCTION				
1. Grading/Draining				
Clear & Grub, Demolition		Channel ex.		
Exc/Emb.	$0	OSC Materials Lab.		
Remove Sidewalk		Scoping Estimate	$480,000	
Riprap		Miscellaneous:		
Drainage	$0			$480,000
2. Structures				
Conc. Bridges		Walls		
Steel Bridges		Riprap		
Tunnels		Miscellaneous:		$0
3. Surfacing/Paving				
Surfacing Type: CSTC	$0	Paving Type: ACP	$0	
Surfacing Type: Ballast	$0	Paving Type:		
Planning	$0	Shoulder Type:		
		Miscellaneous:		$0
4. Roadside Development				
Fencing				
Seeding, Fertilizing & Mulching		Planting		
Temporary Water, Pollution Control		Roadside Cleanup	$0	$0
5. Traffic Services				
Curb & Gutter				
Concrete Sidewalk		Guide Posts		
Signals		Lane Markers		
Illumination	$0	Channelization/Curb		
Signing	$0	Traffic Control	$0	
Temporary Striping		Total Traffic Control	$60,000	$60,000
6. Misc. 10.00%				$54,000
7. Construction Subtotal (Lines 1 thru 6)				$594,000
8. Mobilization 8.00% of Line 7				$47,520
9. Subtotal - Line 7 and 8				$641,520
10. Sales Tax of line 9				$0
11. Agreements (Utility, etc.)				
12. Subtotal - Lines 9 thru 11				$641,520
13. Constr/Engr 10.00% of line 12 &		contingencies 5.00% of line 12		$96,228
14. State Force Work				
15. CONSTRUCTION TOTAL Lines 12, 13 and 14				$737,748
III. PRELIM. ENGINEERING		8.00% of line 15		$59,020
IV. TOTAL ESTIMATED COST		Lines 1, 15 and III		$796,768

North Central Region Page 1

Figure 7: Scoping estimate developed for the unstable slope described on Figure 6 along SR 2 in the Tumwater Canyon by WSDOT's North Central Region (WSDOT).

ACCURACY OF CONCEPTUAL DESIGN COST ESTIMATES

The conceptual design estimates are used not only for the benefit-cost analysis, but also to program the project for design and construction. Because on time and on-budget project delivery is a top priority within WSDOT, an analysis of performance measurement of the Unstable Slopes program was recently completed. To measure performance, the conceptual design cost estimates were compared to the Engineer's final design estimate and the Contractor's low bid for twenty-six unstable slopes projects representing forty-seven unstable slopes. The projects that were selected for the analysis were projects that had no major scope changes from conceptual designs to final design. In addition, to insure consistency, a senior-level engineering geologist or geotechnical engineer completed the conceptual designs along a set of guidelines as opposed to an individual without the technical background. Figure 8 provides a graphical representation of the results of this comparison study. Based on the study the following summary can be made:

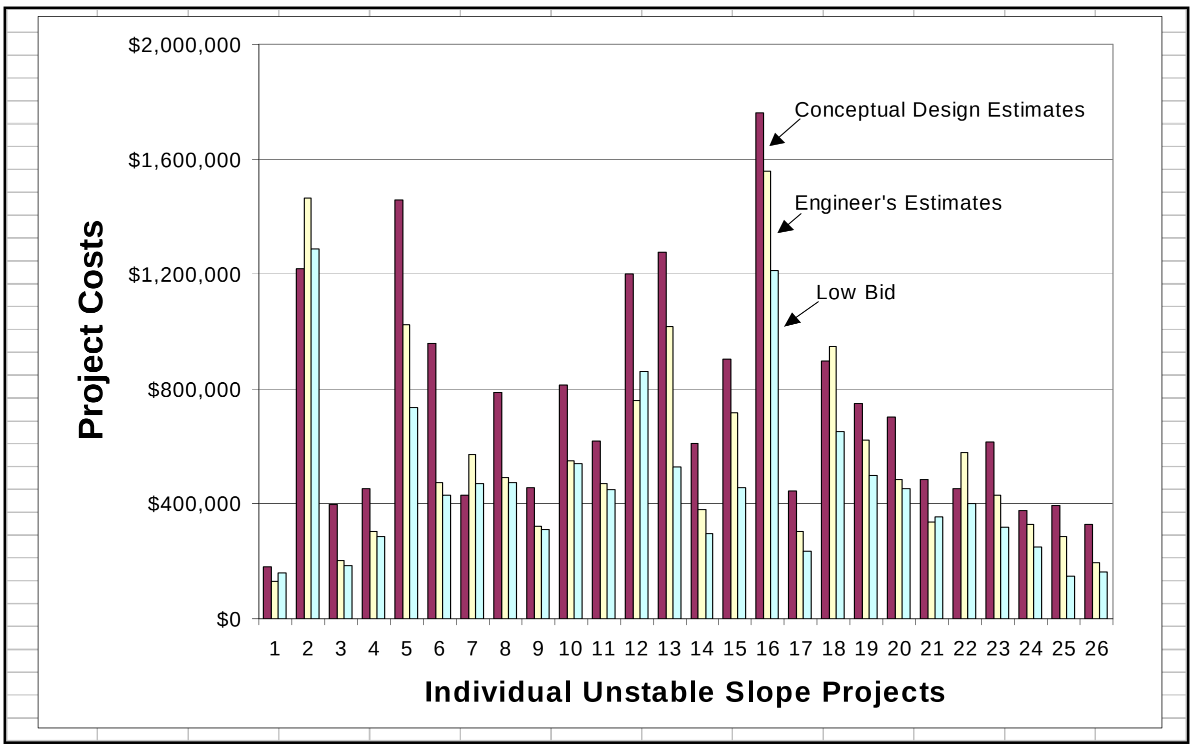


Figure 8: Graphical representation comparing estimates from the conceptual design, Engineer's estimate and Contractor's low bid. The results demonstrate that the conceptual design estimates are generally more conservative than the final cost estimates.

- Out of the twenty-six projects analyzed, twenty-two of the conceptual design estimates overestimated the actual construction costs. It should be noted that the conceptual design estimates also contain the cost for the engineering design.

- Four project conceptual design estimates underestimated the Engineer's estimate.

- Two project conceptual design estimates underestimated the first low bid from a competitive bidding process.

The results of this comparative study demonstrate that the conceptual design estimates are reasonably accurate and appropriately conservative.

Photos displayed on Figure 9a, 9b and 9c are examples of recently completed Unstable Slope projects throughout the State of Washington.

SUMMARY AND CONCLUSIONS

The approach that WSDOT has developed for identifying, categorizing, and prioritizing unstable slope mitigation is a proactive and rational approach to address a variety of geologic (slope) hazards along state highways. WSDOT has established that early evaluation and scoping of unstable slopes are critical to plan, budget and prioritize mitigation work on a statewide basis. When WSDOT developed the USMS, they had four objectives in the program: (1) rationally evaluate more than 2500 unstable slopes, (2) perform early scoping, conceptual designs and cost estimation, (3) conduct cost-benefit analysis of unstable slopes, and, (4) prioritize the mitigation of known unstable slopes according to the expected benefits. Most of the 2,500 unstable slopes have been rated and entered into the USMS. Presently, WSDOT's consultants are reviewing these slopes, developing conceptual designs and uploading them to the USMS database as a permanent record.   WSDOT maintains an updated database of construction bid tabs for conceptual design estimating. The WSDOT Geotechnical Division uses the cost estimate for each unstable slope in the cost-benefit analysis to select and prioritize slopes for mitigation. The benefit-cost analysis compares the maintenance and delay costs over a 20-year program life to the cost of mitigating the slope hazard. Because of limited funding, WSDOT considers only those slopes with cost-benefit ratios above 1.0 for programming. Since the USMS program has begun, WSDOT can demonstrate accurate and conservative results between the conceptual design, Engineer's estimate and the Contractor's low bid, meeting one of WSDOT's strategic objectives of delivering projects on-time and on-budget.

Figure 9a: WSDOT conducting an emergency repair of an unstable slope problem along SR 11 Chuckanut Drive in south Whatcom County, Washington.

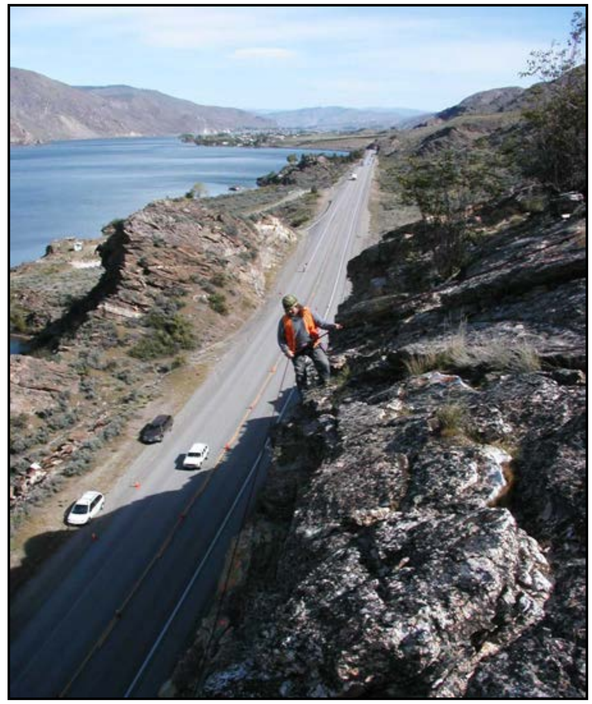


Figure 9b: Kleinfelder team member preparing to map a rockslope using rappelling techniques on SR 97 north of Wenatchee, Washington.

Figure 9c: Installation of horizontal drains and recently completed soldier-pile wall on an unstable slope above the Bogachiel River along SR 101on the Olympic Peninsula.

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HIGHWAY US-34 CUT SLOPE STABILIZATION
MT. PLEASANT, IOWA

Lok M. Sharma, P.E.[1] and Robert Stanley, P.E.[2]

INTRODUCTION

Generally, the economics of highway alignment improvements involve weighing the costs of relocation against long-term advantages for the transportation vehicles. A realignment or relocation may involve geometric constraints, additional right-of-way, large cuts and fills and other governing factors such as user needs, road side developments and politics. The following paper describes a highway relocation project that required large cuts through a portion of the realignment. The Highway US-34 relocation project lies west of the City of Mount Pleasant, Henry Count, Iowa as shown in Figure 1.

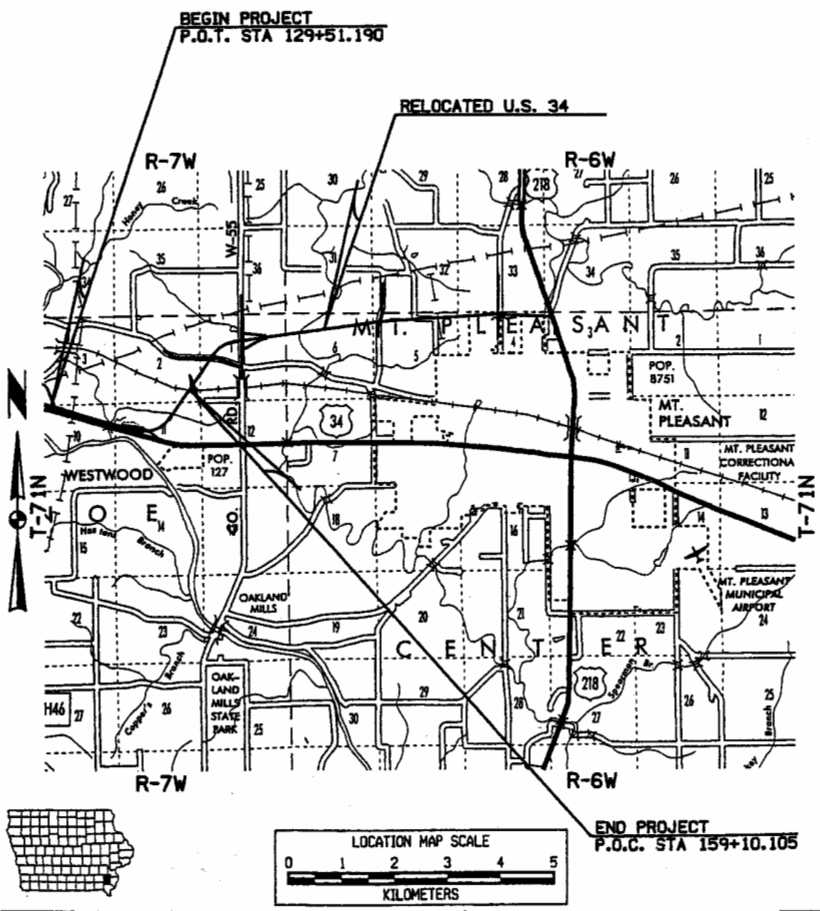

Figure 1. Project Location

For about a 300 meter stretch, the backslope cuts along the west bound lane involved excavation of 3 to 6 meters of soil mantle followed by 10 to 13 meter cuts in interbedded shale and limestone bedrock. For about 80 meters distance the cuts in the soil mantle could not be sloped back to stable slopes due to right-of-way constraints. The Iowa Dept of Transportation (IowaDOT) design document required a soil nail retaining wall in the near vertical cuts in the soil mantle (referred to as Upper Soil Nail Wall). The cut below the soil mantle was designed to be 1/4H: 1V with a 3 meter horizontal bench at the toe of the Upper Soil Nail Wall. The soil nail retention system was a design build bid item in the contract.

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[2] Soils Design Engineer, Soils Design, Iowa Department of Transportation

Before constructing the Upper Soil Nail Wall, the designed cut slope in the bedrock was pre-split and blasted to avoid damage to the Upper Soil Nail Wall. The pre-split holes were drilled at 1/4H:1V cut face in the bedrock. After completion of the Upper Soil Nail Wall in the soil mantle, excavation of the blasted rock started. During the initial excavation the upper shale units encountered in the bedrock were, unexpectedly, highly weathered and potentially unsuitable for a 1/4H: 1V slope. IowaDOT became concerned about the long-term stability of the steep rock face after the removal of the blasted debris. Initial global stability checks indicated that unsupported ¼H: 1V slope in the heterogeneous bedrock presented an unacceptable level of safety against slope failure. The interbedded sequence of weak shales and jointed limestone were susceptible to differential weathering and posed likely slope instability. Extensive slope stability analyses using limit equilibrium methods of analysis indicated that after excavating the blasted rock debris in front of the rock cut, the cut slope would have an unacceptable factor of safety for global failure. The factor of safety could be improved if portions of the blasted rock debris were left in place to form a rock toe buttress. Leaving the entire blasted rock debris was not possible due to the widening requirements of the roadway.

This paper describes how the slopes were analyzed using both Limit Equilibrium Method (LEM) and FLAC numerical analysis method and remedial of rock slope stabilization measures that were designed and implemented.

SITE CONDITIONS

IowaDOT had performed soil explorations for the project. At the location of the steep cut the soil mantle for the Upper Soil Nail Wall was extensively explored by performing borings and testing of collected soil samples. The borings were not extended to the full depth of the cut. However the bedrock units were tagged with series of borings and test pits along the descending top of the cut. A representative subsurface profile through the area is shown in Figure 2. Based on the limited exploration, cut slopes were designed at 1/4H: 1V in the bedrock. The cuts were to be pre-split along the face and blasted for excavation. Figure 2 also identifies various numbers assigned to the rock layers.

The surficial landforms in the area are predominantly drift plains of south eastern Iowa derived from glacial, wind, river and marine environment of the geologic past. The soil mantle overlying the bedrock at this site primarily was comprised of silty lean clays and glacial deposits overlying weathered shale and limestone bedrock. The bedrock units are of Mississippian Geologic age. The bedrock formations at the site include the St. Louis limestone formation followed by shales and limestone of Warsaw and Keokuk formations. Figure 3 show a geological profile at the project site.

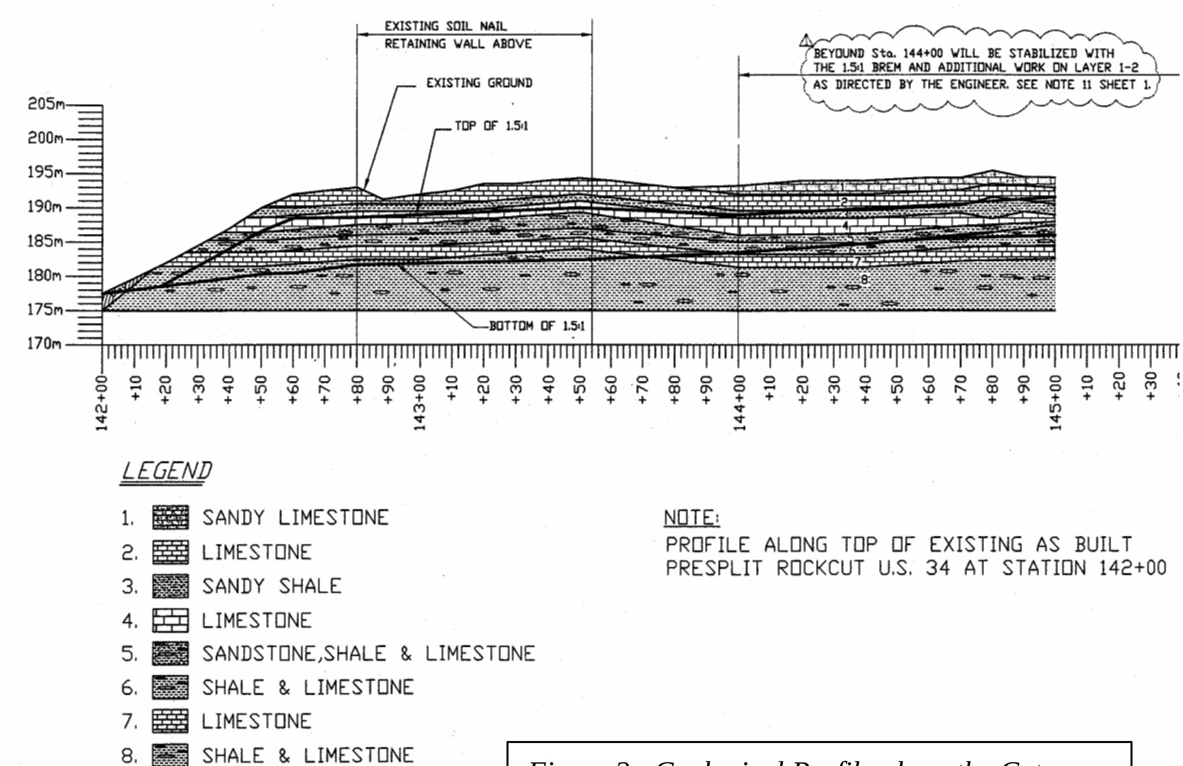
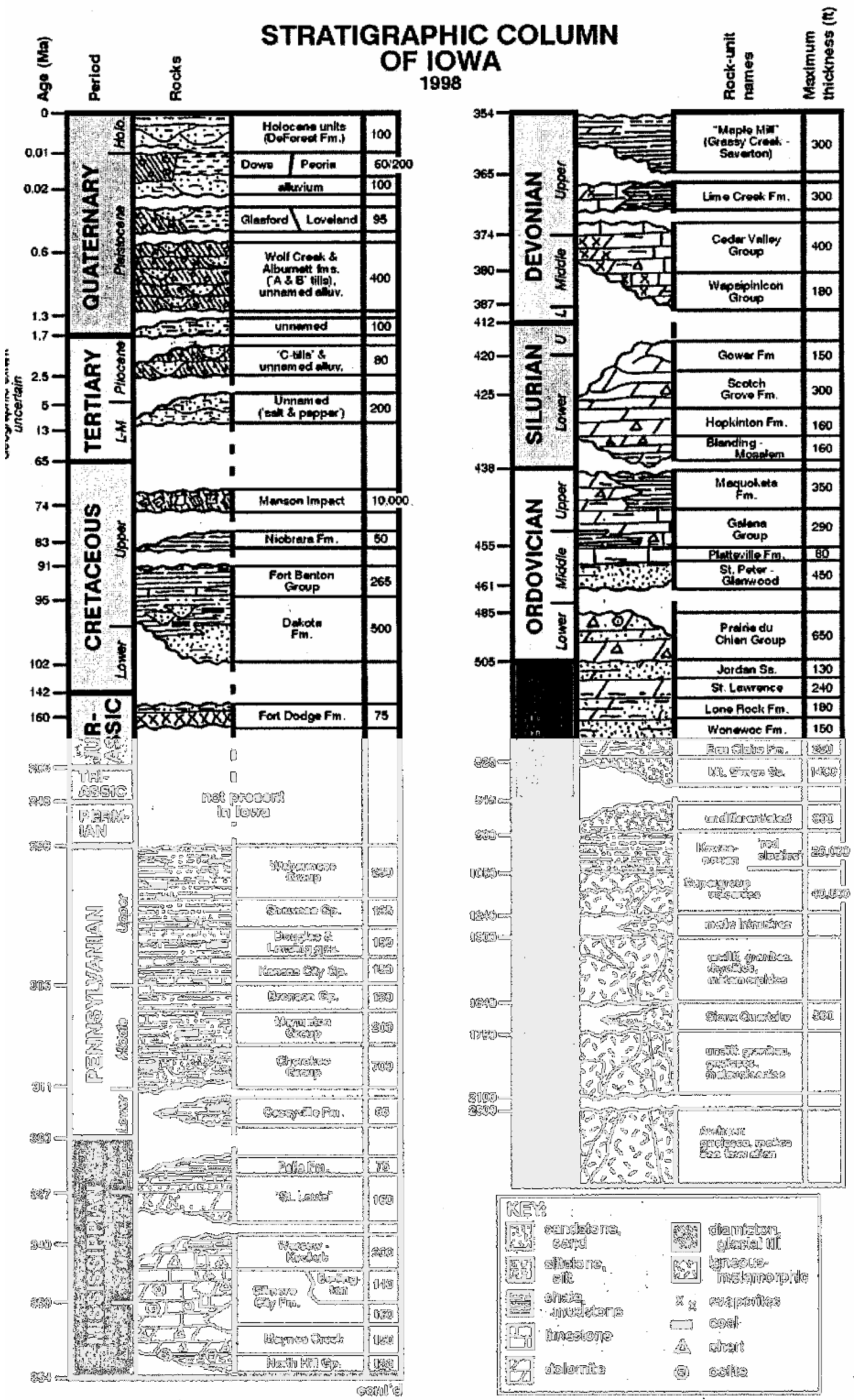


Figure 2. Geological Profile along the Cut

Field Mapping

After pre-splitting and blasting of the bedrock units, the blasted debris was left in place. The debris was planned to be excavated after the Upper Soil Nail was completed. The Upper Soil Nail Wall was completed in the summer of 2004 and the excavation of the blasted rock debris was started soon after the soil nail wall completion. During excavation of the blasted rock it was discovered the rock strata contained numerous layers of weak shale that would likely affect the global stability of the steep slope and the Upper Soil Nail Wall above. The weathered nature of some of the more competent rock layers presented potential for rock fall on to the roadway. In order to assess the rock strata in the rock cut face, the pre-split face was exposed at several locations with a backhoe and various strata of rock units were identified and correlated. The resulting stratigraphy was used for the design. The St. Louis limestone formation at this location is comprised of interbedded layers of sandstone, limestone and shale. The exposed rock layers were field mapped and Rock Mass Rating (RMR) of the rock units were estimated. Representative photographs of the rock strata are shown in Photographs 1 and 2. The subsurface profile shown in Figure 2 was adjusted to reflect the field mapping.



Iowa Geological Survey Bureau 109 Trowbridge Hall Iowa City, IA 52242-1319 Phone: 1-319-335-1575 Fax: 1-319-335-2754

Figure 3. Stratigraphic Column of Iowa

GEOTECHNICAL ANALYSES

Based on our review of field mapping, estimated RMR and limited test data, we estimated properties of various rock layers as listed in Table1. The layers 1a and 1b are primarily in the overburden soils. Layers 2 to 8 are the various shale and limestone/sandstone units as identified in the field mapping. The global stability of the slope was analyzed using STABL5 slope stability program and FLAC (Fast Lagrangian Analysis of Continua) numerical analyses. The STABL program uses limit equilibrium approach, while FLAC simulates the behavior of natural geological structures as well as man made structures built of soil, rock and other materials. The constitutive model for the material behavior in our FLAC analyses was based on Mohr-Coulomb behavior.

The limit equilibrium method of slope stability analysis is based on the principals of statics and remains a useful too for stability analysis. The limit equilibrium method of slope stability analysis does not satisfy displacement compatibility when the material behavior tends to be elastic-plastic. The FLAC modeling presents a stresses and strains base analysis and satisfies the issue of displacement compatibility.

Photograph 1. Showing weathered Jointed Limestone Bedrock

Photograph 2. Showing layers of Bedrock Units

FLAC Analysis

The FLAC slope stability analyses were performed for two critical sections at Station 142+80 and Station 143+00. Figures 4 and 5 provide the geometry of these sections. The Figures also show assumed ground water table across the slope and distribution of finite difference zones (grid). Each finite

Table 1 – Rock Properties and parameters used in the FLAC analysis

Rock properties and parameters used in the FLAC analysis													
Layer number	Description	dry density	Wet density below water table	Young's modulus	Poisson's ratio	$\frac{E}{3(1-2v)}$ Bulk modulus	$\frac{E}{2(1+v)}$ Shear modulus	Cohesion	Friction angle	Tmax = $\frac{c}{\tan\phi}$ Tension		Dilation	Constitutive Model
		ρ	ρ_w	E		K	G	c	ϕ	Tmax	T used	ψ	
		kg/m^3	kg/m^3	Pa	v	Pa	Pa	Pa	degree	Pa	Pa	degree	
2	Highly weathered limestone (top - 5.5m)	2200	2800 (for model simplification)	2.95E+09	0.3	2.5E+09	1.1E+09	2.3E+05	40	2.7E+05	1.4E+05	0	Mohr-Coulomb
3	Soft shale (second from top)	2200		1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.0E+04	0	Mohr-Coulomb
4	Massive limestone	2400		2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05	0	Mohr-Coulomb
5	Sandstone, shale & limestone	2300		1.00E+09	0.3	8.3E+08	3.8E+08	1.0E+06	35	1.4E+06	7.1E+05	0	Mohr-Coulomb
6	Hard shale with limestone strings	2300		1.00E+08	0.4	1.7E+08	3.6E+07	2.4E+05	25	5.1E+05	2.1E+05	0	Mohr-Coulomb
7	Massive limestone	2400		2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05	0	Mohr-Coulomb
8	Hard shale with limestone (bottom layer to greater depth)	2300		1.00E+08	0.35	1.1E+08	3.7E+07	5.0E+05	35	7.1E+05	3.6E+05	0	Mohr-Coulomb
	Rock fill at toe	2000		1.00E+07	0.35	1.1E+07	3.7E+06	0.0E+00	34	0.0E+00	0.0E+00	0	Mohr-Coulomb
For soil retained by upper wall													
1-A	Firm Shale	2000	dry	1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.00E+04	0	Mohr-Coulomb
1-B	Glacial Till	2000	dry	1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.00E+04	0	Mohr-Coulomb

difference zone is 0.5 m by 0.5 m square except along the slope where FLAC adjusted different shaped zones to fit the geometry.

Actual measured rock properties of the materials in the slope were not available. The values of properties and parameters were estimated based on the field mapping, estimated RMR, available information in the literature and our experience with similar rock types. The rock properties in Table 2 and 3 were used in the stability of the rock slope at station 142+80. The properties in Table 4 were used in the analysis of slope at Station 143+00. The cross-section configurations were decided on the basis of leaving the blasted rock debris starting at a slope of 1-1/2H: 1V at the roadway edge and meeting the 1/4H: 1V pre-split face of the rock cut; from thereon the rock pre-split face left at 1/4H: 1V slope.

The rock slopes at both sections were analyzed with dry as well with water table as shown in the sections in Figures 4 and 5.

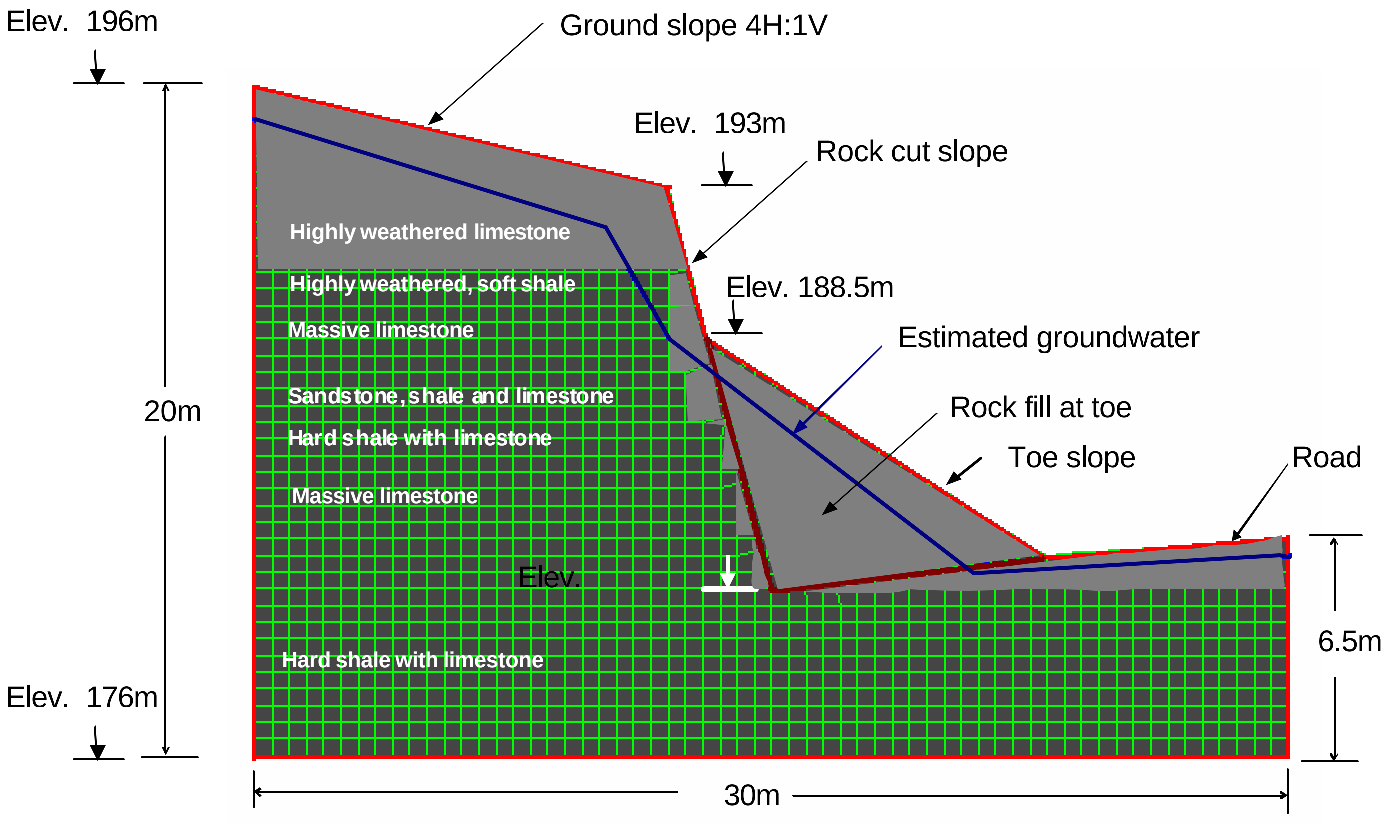


Figure 4. Rock slope at Station 142+80

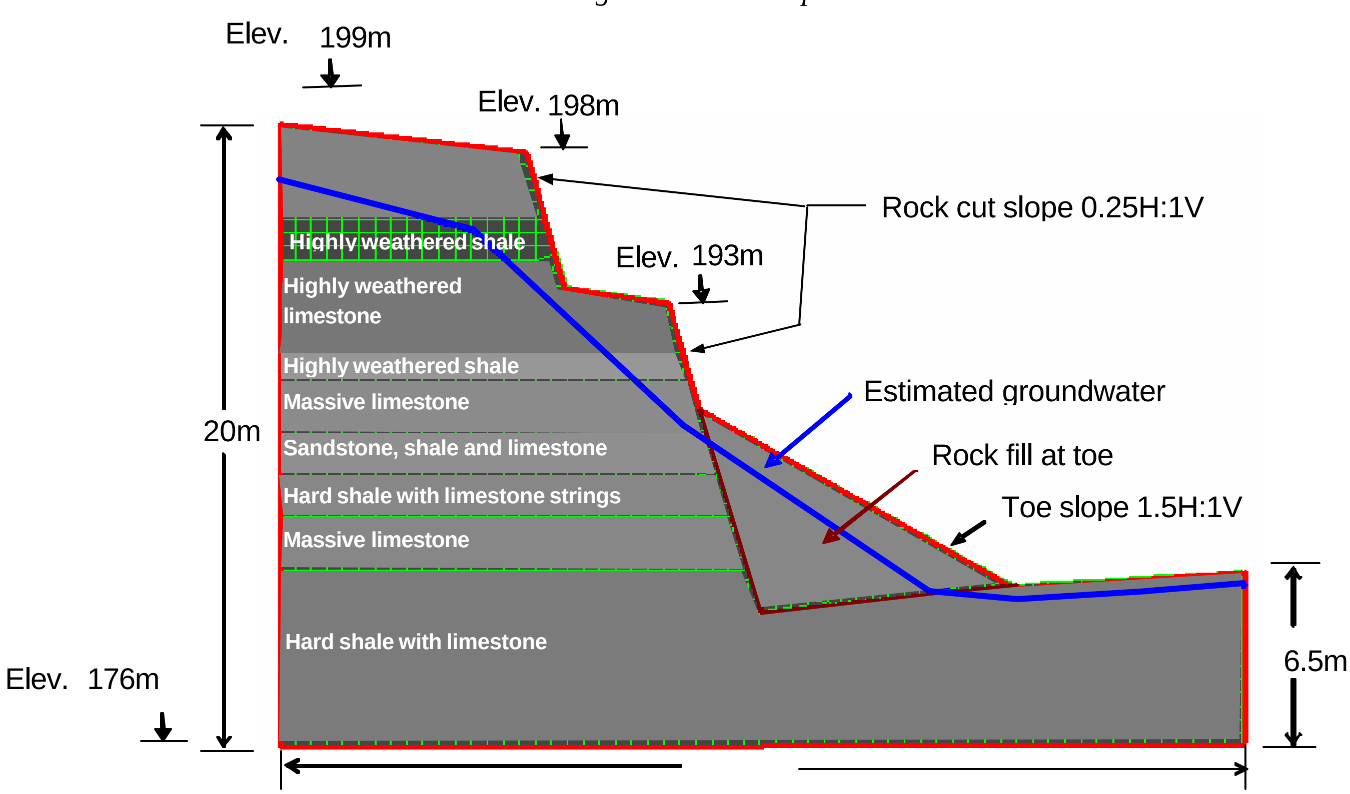


Figure 5. Rock slope at Station 143+00 (with the upper soil nail wall)

Table 2. Rock parameters used in Analysis Case 1

Station 142+80

Layer (from top)	Elevation	Thickness	dry density	Wet density below water	Analysis Case 1							
					Elastic parameters				Mohr-Coulomb palsticity parameters			
					Young's modulus	Poisson's ratio	Bulk modulus	Shear modulus	Cohesion	Friction angle	Tension	
			?	?sat	E		K	G	c	F	Tmax	T
	m	m	kg/m^3	kg/m^3	Pa	?	Pa	Pa	Pa	degree	Pa	Pa
1. Highly weathered limestone	190.5-196.0	2.5 to 5.5	2200	2800 (for model simplification)	2.95E+09	0.3	2.5E+09	1.1E+09	6.7E+04	40	8.0E+04	1.0E+03
2. Highly weathered, soft shale	189.5-190.5	1	2200		5.00E+06	0.4	8.3E+06	1.8E+06	2.4E+03	18	7.4E+03	5.0E+02
3. Massive limestone	187.5-189.5	2	2400		2.85E+10	0.3	2.4E+10	1.1E+10	6.7E+06	40	8.0E+06	2.0E+03
4. Sandstone, shale & limestone	186.0-187.5	1.5	2300		1.00E+09	0.3	8.3E+08	3.8E+08	6.7E+05	35	9.6E+05	1.0E+03
5. Hard shale with limestone strings	184.5-186.0	1.5	2300		1.00E+08	0.4	1.7E+08	3.6E+07	1.0E+04	25	2.1E+04	1.0E+03
6. Massive limestone	182.5-184.5	2	2400		2.85E+10	0.3	2.4E+10	1.1E+10	6.7E+06	40	8.0E+06	2.0E+03
7. Hard shale with limestone (bottom layer to greater depth)	176.0-182.5	6.5	2300		1.00E+08	0.35	1.1E+08	3.7E+07	1.0E+05	35	1.4E+05	1.0E+03
Rock fill at toe	-	-	2000		1.00E+07	0.35	1.1E+07	3.7E+06	0.0E+00	34	0.0E+00	0.0E+00

Table 3. Rock parameters used in Analysis Case 2

Station 142+80

Layer (from top)	Elevation	Thickness	dry density	Wet density below water	Analysis Case 2							
					Elastic parameters				Mohr-Coulomb palsticity parameters			
					Young's modulus	Poisson's ratio	Bulk modulus	Shear modulus	Cohesion	Friction angle	Tension	
			?	?sat	E		K	G	c	F	Tmax	T
	m	m	kg/m^3	kg/m^3	Pa	?	Pa	Pa	Pa	degree	Pa	Pa
1. Highly weathered limestone	190.5-196.0	2.5 to 5.5	2200	2800 (for model simplification)	2.95E+09	0.3	2.5E+09	1.1E+09	2.3E+05	40	2.7E+05	1.4E+05
2. Highly weathered, soft shale	189.5-190.5	1	2200		1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.0E+04
3. Massive limestone	187.5-189.5	2	2400		2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05
4. Sandstone, shale & limestone	186.0-187.5	1.5	2300		1.00E+09	0.3	8.3E+08	3.8E+08	1.0E+06	35	1.4E+06	7.1E+05
5. Hard shale with limestone strings	184.5-186.0	1.5	2300		1.00E+08	0.4	1.7E+08	3.6E+07	2.4E+05	25	5.1E+05	2.1E+05
6. Massive limestone	182.5-184.5	2	2400		2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05
7. Hard shale with limestone (bottom layer to greater depth)	176.0-182.5	6.5	2300		1.00E+08	0.35	1.1E+08	3.7E+07	5.0E+05	35	7.1E+05	3.6E+05
Rock fill at toe	-	-	2000		1.00E+07	0.35	1.1E+07	3.7E+06	0.0E+00	34	0.0E+00	0.0E+00

Table 4. Rock parameters used in Analysis Case 3

Station 143+00

Layer (from top)	Elevation	Thickness	dry density	Wet density below water	**Elastic parameters**				**Mohr-Coulomb palsticity parameters**			
					Young's modulus	Poisson's ratio	Bulk modulus	Shear modulus	Cohesion	Friction angle	Tension	
			?	?sat	E		K	G	c	F	Tmax	T
	m	m	kg/m^3	kg/m^3	Pa	?	Pa	Pa	Pa	degree	Pa	Pa
1. Glacial till	195.5-199.0	2.5 to 3.5	2000		1.00E+07	[illegible]	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.00E+04
2. Highly weathered shale	194.0-195.5	1.5	2000		1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.00E+04
3. Highly weathered limestone	190.5-194.0	2.5 to 3.5	2200		2.95E+09	0.3	2.5E+09	1.1E+09	2.3E+05	40	2.7E+05	1.4E+05
4. Highly weathered, soft shale	189.5-190.5	1	2200		1.00E+07	0.4	1.7E+07	3.6E+06	2.3E+04	25	4.9E+04	1.0E+04
5. Massive limestone	187.5-189.5	2	2400	2800 (for model simplification)	2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05
6. Sandstone, shale & limestone	186.0-187.5	1.5	2300		1.00E+09	0.3	8.3E+08	3.8E+08	1.0E+06	35	1.4E+06	7.1E+05
7. Hard shale with limestone strings	184.5-186.0	1.5	2300		1.00E+08	0.4	1.7E+08	3.6E+07	2.4E+05	25	5.1E+05	2.1E+05
8. Massive limestone	182.5-184.5	2	2400		2.85E+10	0.3	2.4E+10	1.1E+10	1.0E+06	40	1.2E+06	6.0E+05
9. Hard shale with limestone (bottom layer to greater depth)	176.0-182.5	6.5	2300		1.00E+08	0.35	1.1E+08	3.7E+07	5.0E+05	35	7.1E+05	3.6E+05
Rock fill at toe	-	-	2000		1.00E+07	0.35	1.1E+07	3.7E+06	0.0E+00	34	0.0E+00	0.0E+00

Initially, the slope at Station 142+80 was analyzed using the properties detailed in Table 2. The results of analysis indicated a number of zones within the highly weathered, soft shale layer located at elevation 189.5 to 190.5 reached yield state. This resulted in tension failure in the weathered limestone strata overlying the shale layer. Review of this result suggested increasing the strength (cohesion, friction angle and tensile strength) and stiffness (Young's Modulus) parameters of the shale layer and modifying cohesion and tensile strength of other layers as shown in Table 3. The slope at Station was analyzed with the values of Table 3 and the values in Table 4 were used to analyze slope at station 143+00.

Analyses were performed with plane strain condition. The vertical sides of the models (ends) were considered to be fixed against x-displacements and free in y-displacements. The base of the finite difference model was considered free in x-displacements and fixed in y-displacements.

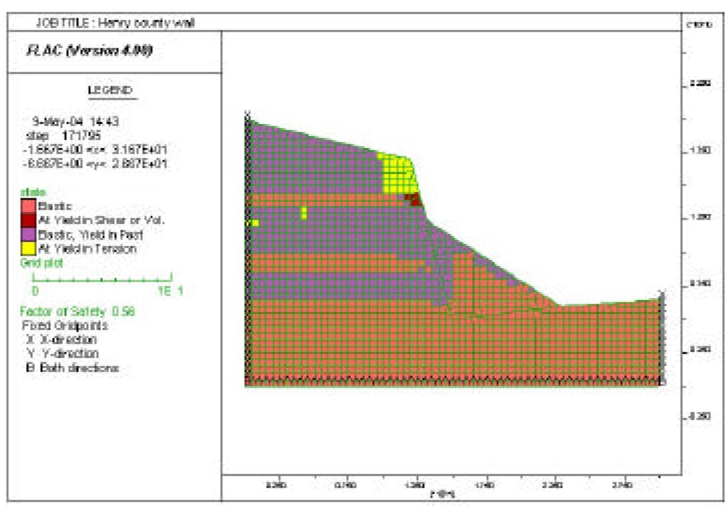


Figure 6 – Case 1 (no water table) F.0.S.=0.56

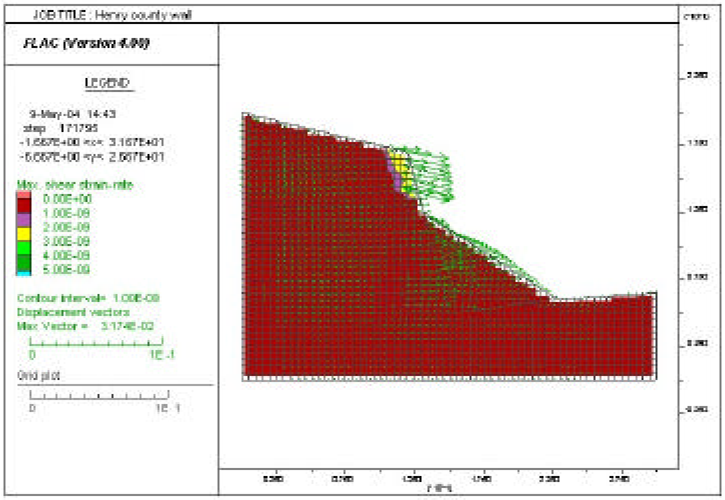


Figure 7 – Case 1 (no water table) Displacement Vectors

Analysis results from FLAC were plotted for the elastic/plastic states and displacement/velocity vectors, which are presented in Figures 6 to 17. Figures 6 through 9 are the results from Case 1 when the rock slope of Station 142+80 was analyzed using properties in Table 3. Figures 10 through 13 are the results of analysis (Case 2) of the same slope using values in Table 4. Figures 14 through 17 present the results of analysis for Case 3 when slope at station 143+00 was analyzed using parameters in Table 4.

Case 1 presented factor of safety of 0.58, which is very low and indicated unstable slope. A number of elements showed either shear failure or tension failure. Since the rock slope in the field had been exposed for some months and was still standing, the low factor of safety obtained in the analysis was unrealistic that led us to discard the estimated values in Table 2. With the values in Tables 3 and 4 the planned rock slope at both the sections appeared to be stable but with a low factor of safety of 1.04 to 1.06.

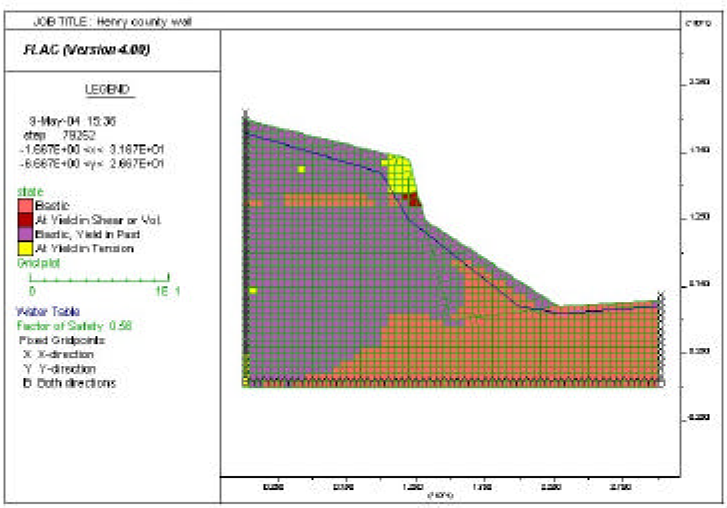


Figure 8 – Case 1 (with water table) F.0.S.=0.56

The STABL program analysis indicated the modified slope configuration with rock debris at the base has a minimum factor of safety of global failure of 1.7, as shown in Figure 18 (in STABL analysis the failure surface was forced beyond the reinforced zone of the Upper Soil Nail Wall). The FLAC model showed some elastic yield of the weak shale units and thus a much lower factor of safety of 1.06. The elastic yielding is critical in the shale unit layer No. 3 due to its softness and lack of confinement at the face.

Also, the shale is likely to deteriorate at a faster rate due to weathering. Thus, we concluded that the shale layers need to be stabilized to provide for confinement and long-term strain softening.

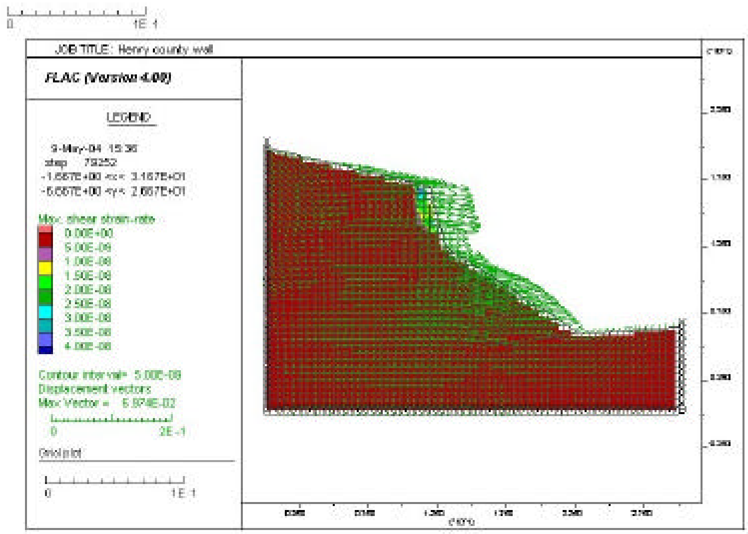


Figure 9 – Case 1 (with water table) Displacement Vectors

From the analyses it was established that the global stability of the modified slope had an adequate factor of safety. The exposed rock face, however, is vulnerable to toppling failure due to elastic yielding of the shale layers. Therefore the exposed 1/4H: 1V rock slope needed some stabilization. Rock nail reinforcement system with shotcrete facing was adopted to check the yielding and weathering of the rock face layers.

ROCK NAIL STABILIZATION

The analyses discussed above dictated that the exposed 1/4H: 1V in the rock face above the debris fill needed to be stabilized for the following reasons:

- To provide confinement of the shale layers and prevent relaxation and softening with time
- To protect against differential weathering of the exposed different rock strata
- Softening and differential weathering in the shale strata could lead to toppling failure of the overlying limestone strata and the Upper Soil Nail Wall.

The geometry thus considered for rock face stabilization was the Upper Soil Nail Wall with a 3 m horizontal bench followed by a 1/4H: 1V rock slope to a distance where a 1-1/2H: 1V slope in the rock debris fill at the toe of the cut would meet.

The method chosen to design the stabilization measures is consistent with the "Soil Nail Wall" concept, although in this case it is applied to the weathered limestone/shale bedrock. The methodology presented in FHWA-SA-96-069 titled "Manual for Design and Construction Monitoring for Soil Nail Walls" was used. The summary of the rock nail stabilization is shown below. The facing was designed based on the assumption that a finite wedge of rock could destabilize and impose loading on the shotcrete facing and the rock nail.

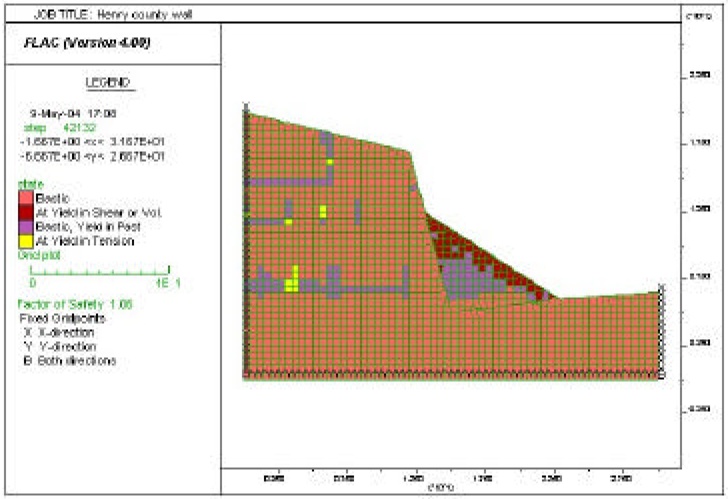


Figure 10 – Case 2 (no water table) F.0.S.=1.06

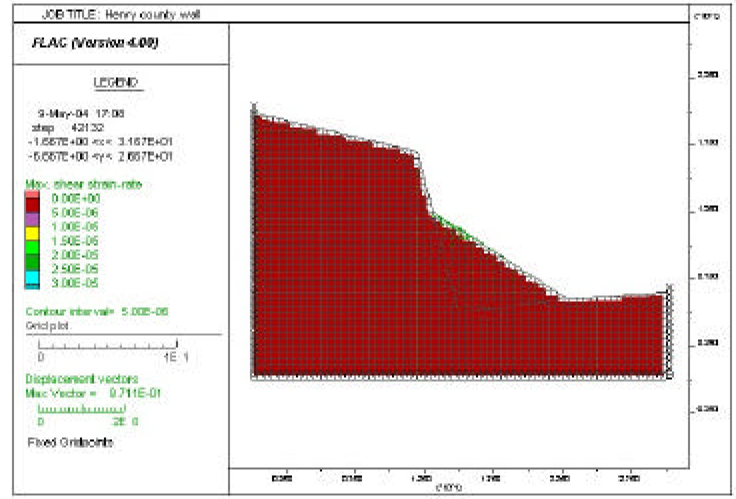


Figure 11 – Case 2 (no water table) Displacement Vector

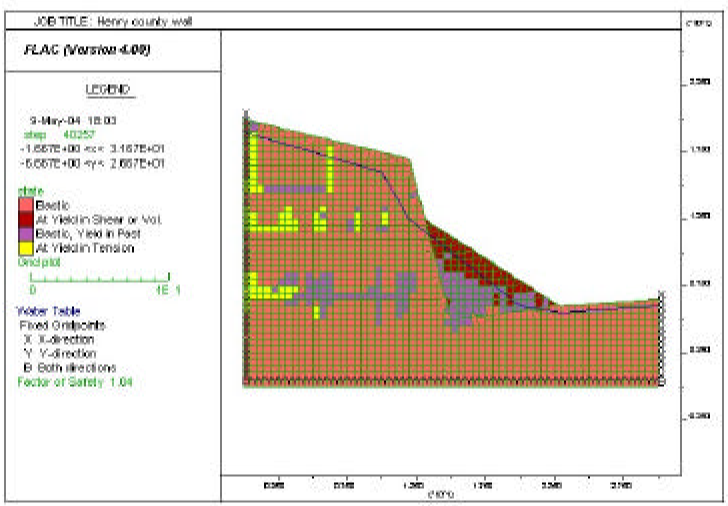


Figure 12 – Case 2 (with water table) F.0.S.=1.04

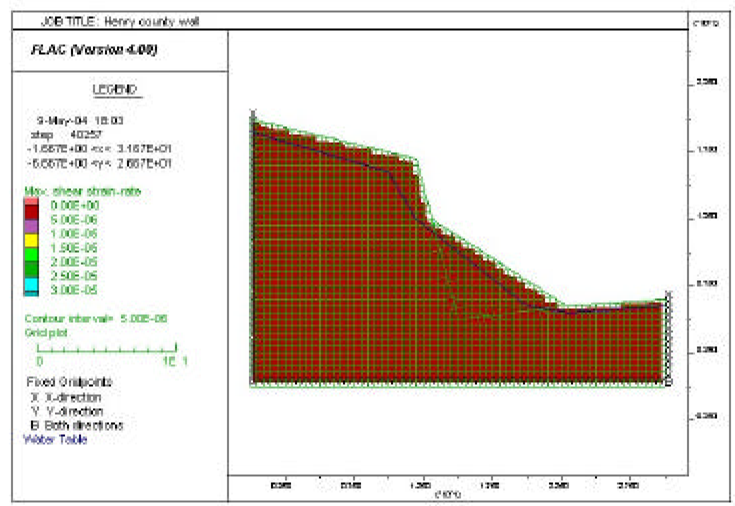


Figure 13 – Case 2 (with water table) Displacement Vector

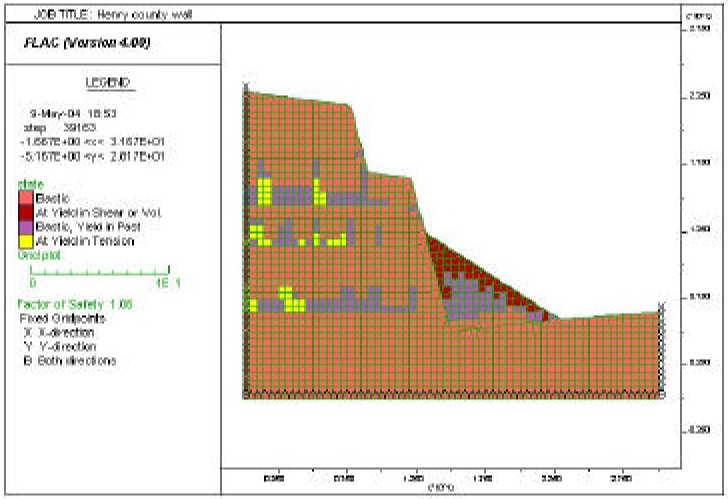


Figure 14 – Case 3 (no water table) F.0.S.=1.03

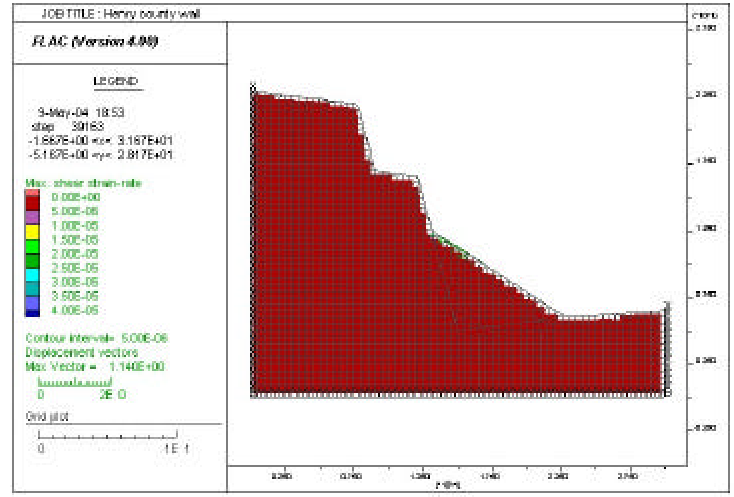


Figure 15 – Case 3 (no water table) Displacement Vector

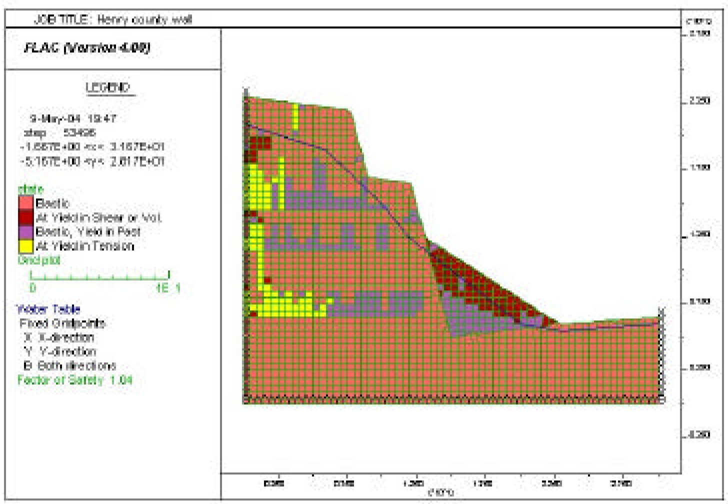


Figure 16 – Case 3 (with water table) F.0.S.=1.04

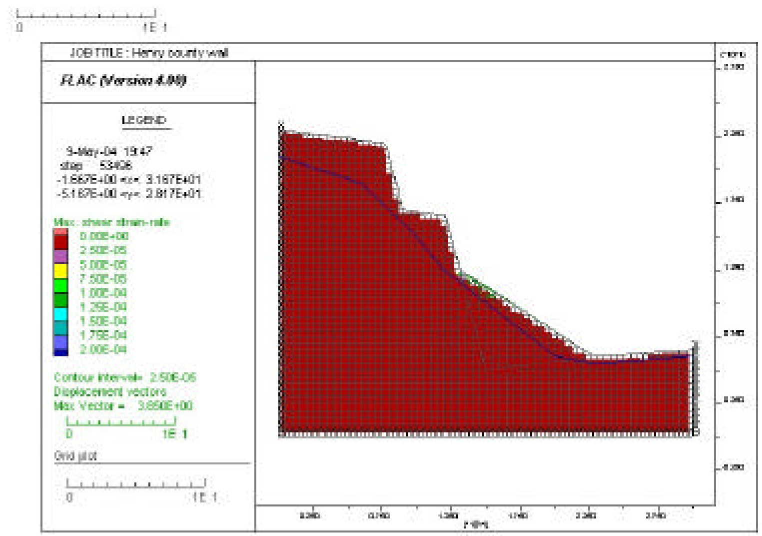


Figure 17 – Case 3 (with water table) Displacement Vector

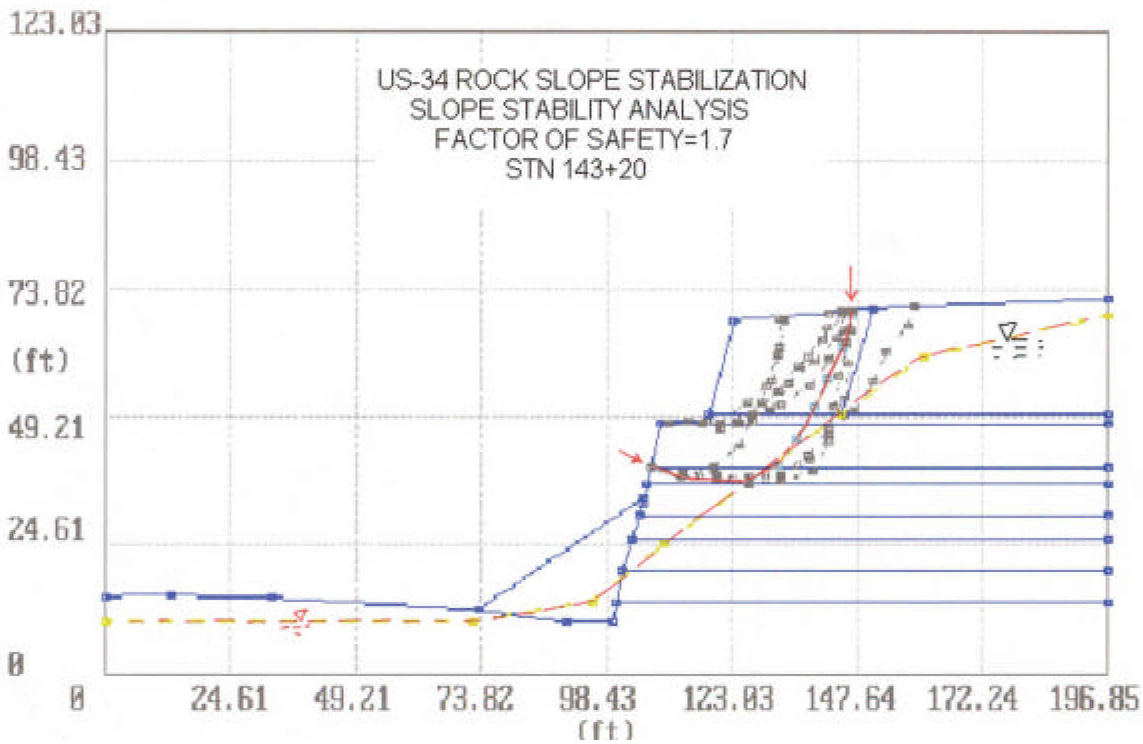


Figure 18 Global Stability Analysis at St. 143+20

Nails

Nail Spacing (H x V)	2.5 m x 2.5 m
Nail Length	10 m
Nail Diameter	30 mm
Drill Hole Diameter	125 mm

Temporary Shotcrete

Bearing Plate Dimensions	200 mm x 200 mm
Plate Studs	125 mm x 19 mm
Stud Heads	32 mm x 9.5 mm
Welded Wire mesh	(2) 102 x 102-MW19xMW19
Waller Bars	2 Continuous No. 13
Vertical Bars	2 No.13; 75 mm long @150 mm o/c
Shotcrete Thickness	100 mm

Permanent Cast-In-Place Facing (CIP)

Finish	Textured stone masonry
Horizontal Bars	No. 13 at 300 mm o/c
Vertical Bars	No. 13 at 300 mm o/c
CIP Thickness	200 mm
CIP Strength	28 Mpa

The rock nail stabilization was carried out in two stages. The first level bench to install the first row of nails was excavated to 1.25 m below the level of the rock nail. After first row nail and temporary shotcrete completion, excavation for the next row of nails was similarly repeated. The bottom of shotcrete was taken to about 1.25 meter below the top of the 1-1/2H: 1V slope of the rock fill debris. Typical rock nail stabilization is shown in Figure 19 and 20. FLAC analysis was used to check the stresses in the nails. Figure 21 shows the magnitude of stresses at different nail levels. Photograph 3 shows the stabilized work nearing completion.

Photograph 3. Showing the Upper Soil Nail Wall and the Rock Nail Stabilization

CONCLUSIONS

The limit equilibrium method of slope stability is a useful tool and provides representative results for global analysis of slopes. However, it can not adequately model stress relaxation and yielding in layers reaching the elastic-plastic state due to unloading or stress increase. The use of FLAC analysis modeled the steep rock slope and allowed a displacement based analysis of the layered rock slope. With appropriate soil and rock properties selection, lower bound solutions to the steep slope stability were obtained indicating low factor of safety due to yielding of the softer shale layers.

Rock nail stabilization with shotcrete facing and toe rockfill buttress has provided adequate factor of safety against failure. The FHWA soil nail design methodology was found to be applicable for rock nails as the stresses checked with FLAC modeling were within acceptable levels.

ACKNOWLEDGEMENT

The writers are indebted to many co-workers at IowaDOT and Terracon for their efforts, support and input into the successful completion of this project and this paper. The efforts of Dr. Binod Sapkota of Terracon for assistance in FLAC analysis are greatly appreciated.

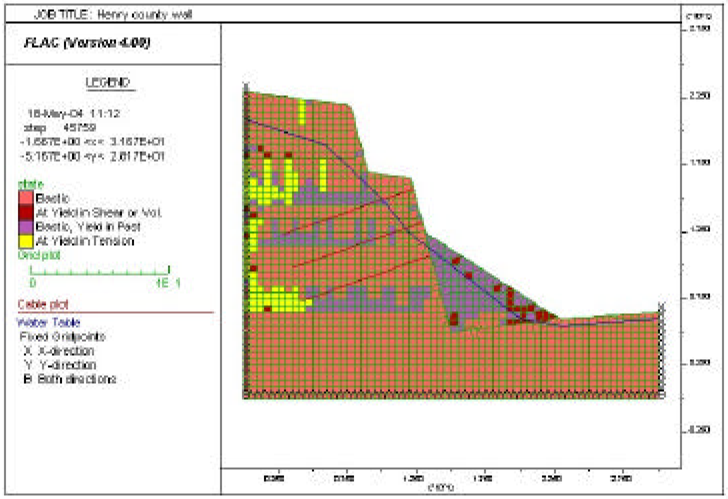


Figure 19 – Rock Nail Stabilization Geometry

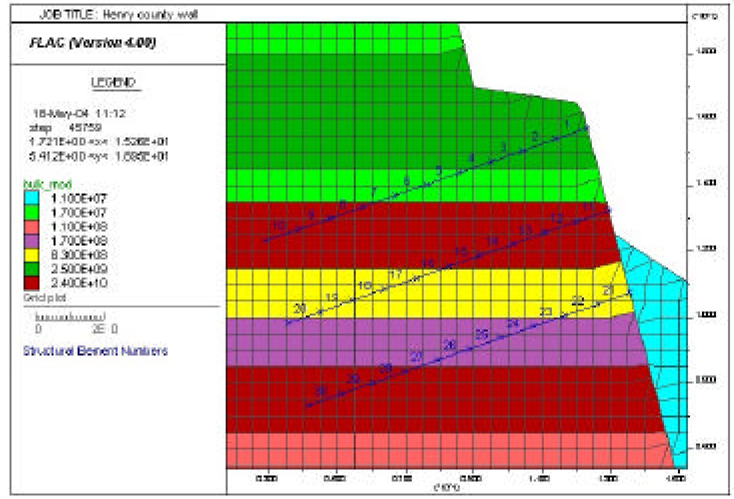


Figure 20 – Rock Nail Stabilization Structural Elements

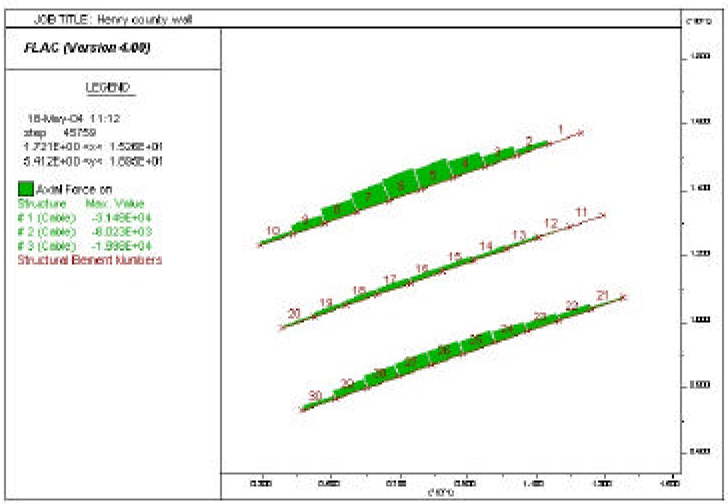


Figure 21 – Axial Force on the Rock Nails

References:

Federal Highway Administration (1996) “Manual for Design and Construction Monitoring of Soil Nail Walls,” FHWA-SA-06-069, B.J. Byrne, D. Cotton, J. Porterfield, C. Wolschlag, G. Ueblacker.

Fast Lagrangian Analysis of Continua (FLAC) ver 4.10, Itasca Consulting Group, Inc., 2002.

Terracon Consultants, Inc., Report on “US-34 Relocation Project, Henry County, Iowa, Cut Slope Stabilization,” June 2004.

STRATIGRAPHIC INTERPRETATIONS OF LIMESTONE, GEOPHYSICAL SURVEYS, AND BOREHOLE DATA IDENTIFY POTENTIAL IMPACT ON HIGHWAY AND GUIDE FUTURE QUARRY EXPANSION

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ABSTRACT

The proposed expansion of a limestone quarry toward U.S. Highway 258 in Onslow County, NC, necessitated an investigation of its potential impact because current quarrying activities have been associated with aquifer dewatering and concomitant sinkhole development. Sinkholes and other solution features occur in the quarry but also in areas beyond the quarry perimeter. The Eocene Castle Hayne Limestone underlies the area and is in turn overlain by surficial sediments (overburden) consisting of organic rich sand, clay, and silt. Overburden thickness varies in the area and generally is greater in the vicinity of the quarry than north along the New River (two miles) where the Castle Hayne is exposed. In the vicinity of the quarry, the Castle Hayne Limestone consists of two units that are lithologically similar but display different sizes of solution features. The lower unit (A) is greater than 98 ft in thickness and comprised predominantly of medium to coarse shell fragments in medium to thick composite sets of cross beds. Separated by a disconformity is an upper unit (B) comprised predominantly of lower lime mud overlain by medium to coarse shell fragments ranging from being absent to almost 35 ft in thickness. The lower Castle Hayne unit underlies the entire quarry area whereas the upper unit is absent in the central part of the quarry and in the central area of the proposed expansion. Sinkholes and/or solution features occur in both Castle Hayne units and may be associated with a rectilinear set of fractures that trend NNE-SSW and NNW-SSE. Karstic features (sinkholes) occur along these fractures and major sinkholes tend to concentrate at fracture set intersections.

Lime mud in the lower part of unit B serves as an aquiclude restricting the downward movement of water into unit A of the Castle Hayne Limestone. Based on observations in the active quarry, solution features and sinkholes that develop above this zone are in the more porous and permeable upper part of the Castle Hayne (Unit B) and are larger (up to 10 ft in size) than sinkholes in the lower unit (Unit A). Solution features in the lower unit (A) of the Castle Hayne Limestone are smaller (1-3 ft in size) than those in unit B.

DC resistivity surveys made along roads adjacent to the area of proposed quarry expansion verified the stratigraphic relationships determined from core study. The surveys also indicated potential areas of sinkhole development along the roads that needed monitoring during quarry expansion. Ground penetrating radar (GPR) surveys along U.S. 258 were inconclusive.

INTRODUCTION

In December 2003 a proposed expansion of a quarry in Onslow County necessitated an evaluation of its potential impact on adjacent highways because nearby sinkholes had been attributed to quarry operations (Fig. 1). NCDOT was interested in minimizing the effects of the quarry expansion on NC roadways. Several methods were used to study the local geology and recommendations were made concerning the quarry expansion.

Figure 1. A- Sinkhole on Duffy Field Road that developed in December 2003, view west. **B-** Below ground view of sinkhole illustrating its lateral extent. The sinkhole is developed in Unit B of the Castle Hayne Limestone; overburden is at the top of the photo. Note the roadway asphalt.

Onslow County is located in eastern North Carolina in the Atlantic Coastal Plain Province, an area consisting of relatively low relief and unconsolidated sediments. The area consists of a seaward dipping wedge of Mesozoic-Cenozoic sediments and rocks that are older west along the fall line and younger east along the coast. Sediments and rocks of the Atlantic Coastal Plain Province are separated into depositional basins and intervening highs beginning in the north with the Salisbury embayment in Virginia and Maryland and ending with the Peninsular arch in

Florida. South of the Norfolk arch in Virginia is the Albemarle embayment in the northeastern part of North Carolina. Here about 10,000 ft of Mesozoic-Cenozoic sediments and rocks occur near Cape Hatteras, which is the thickest onshore coastal plain section in North Carolina. South of the Albemarle embayment is the northwest-southeast trending Cape Fear arch onto which the thick Albemarle embayment section thins. In the Cape Fear area, just south of Wilmington, the coastal plain is about 1500 ft thick. Between the two areas is a northwest-southeast trending transition zone known as the Neuse hinge with the area north called the Albemarle block and the area south called the Onslow block (Harris and Laws, 1997).

The quarry is located about 10 miles northwest of Jacksonville and three miles south of Richlands in the Catherine Lake, North Carolina 7.5-minute quadrangle, Onslow County, just west of Union Chapel and south of Duffy Field Roads (Figs. 2, 3).

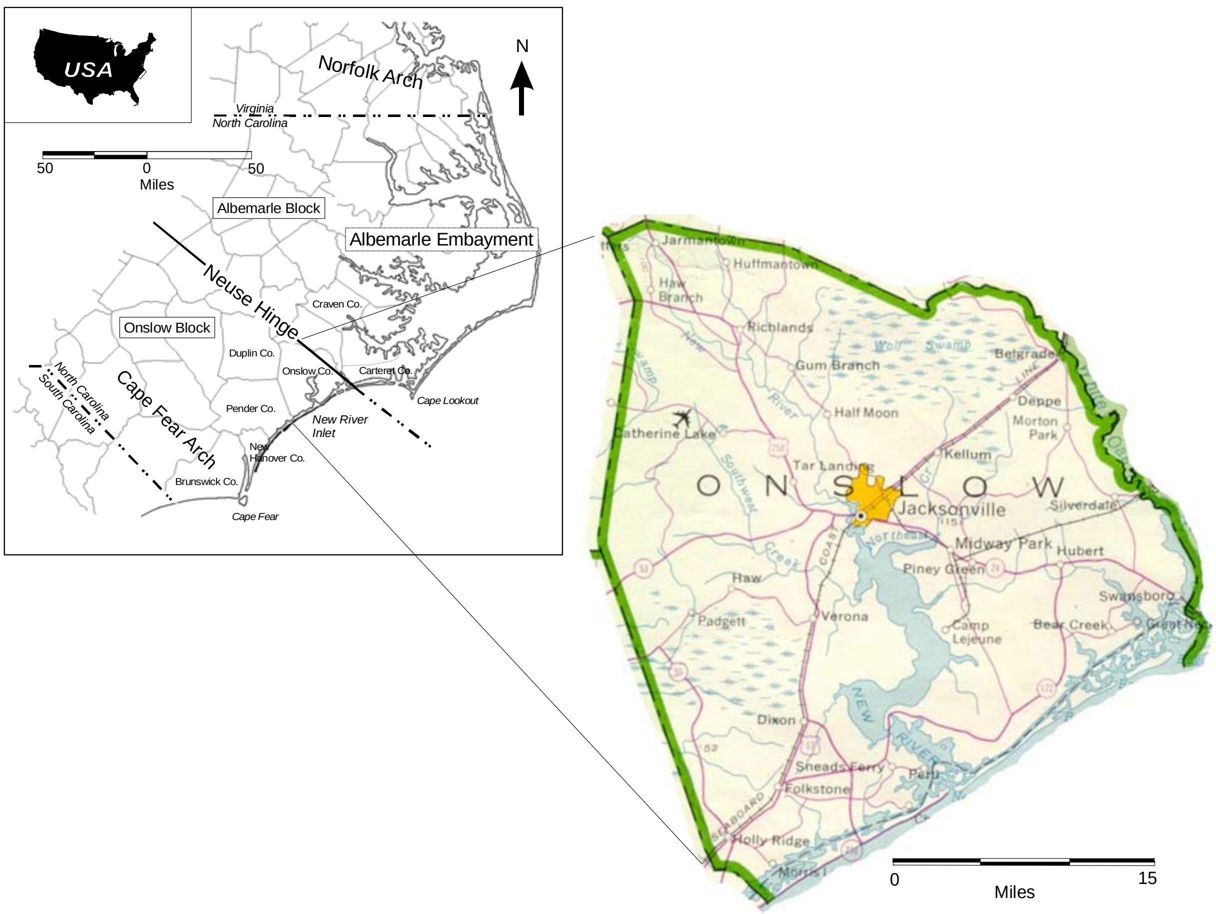

Figure 2. Location of Onslow County, North Carolina with respect to major geologic features of the North Carolina Coastal Plain.

This paper provides a model for integrating stratigraphic and geophysical data to determine the potential impact of quarry expansion on U.S. Highway 258 and adjacent roads. The concern is that as the quarry expands to the north sinkholes will develop. Another concern is if they do develop, what areas of the major four-lane highway have the greatest possibility for sinkhole hazards. This investigation examined the possible relationship between quarry activities and sinkhole development in the area and identifies potential sites that need monitoring concomitant with quarry expansion.

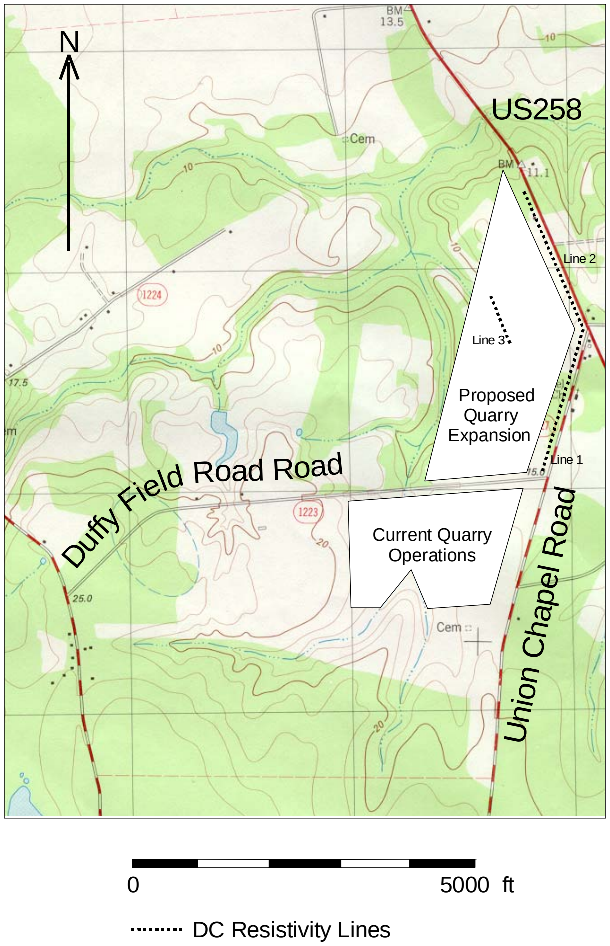

Figure 3. The active quarry is located south of Duffy Field Road and west of Union Chapel Road. The proposed expansion is to the north of Duffy Field Road and brings the active quarry adjacent to U.S. Highway 258, about two miles southeast of Richlands, NC. The map illustrates the location of resistivity surveys; GPR data were also collected along the same lines.

METHODS

To determine the relationship between sinkholes and quarry operations the geology of the active quarry and nearby outcrops on the New River were studied. Exploratory cores from the area of the proposed quarry expansion were also studied. In addition, topographic maps, historical records and aerial photographs of the area were examined to identify pre-quarry sinkholes or other nearby features attributed to limestone dissolution. Table 1 lists the cores used in this study and stratigraphic units recognized in each.

Ground penetrating radar and DC resistivity surveys were conducted around the quarry in Onslow County in May 2004 by Geophex, Ltd. (Report, May 2004). Over 5700 linear ft of GPR data were collected along U.S. 258 extending a 2002 survey northwest approximately 1100 ft. Approximately 425 ft of GPR data were collected west of Union Chapel Road within the proposed area of quarry expansion (Fig. 3). The GPR data were collected using either a RAMAC 100 MHz or 250 MHz antenna with an integrated X3M control unit manufactured by MALA Geoscience. GPR data were processed using RADAN© software from GSSI, Inc. The GPR unit was pulled behind a pick-up truck traveling at approximately 2 miles per hour, with signal pulses triggered every 0.2 ft. Distance was monitored by a calibrated survey wheel attached to the radar antenna. Four thousand forty one ft of linear resistivity data were collected along U.S. 258 and Union Chapel Road (Fig. 3). Seven hundred twenty ft of resistivity data were also collected west of Union Chapel Road within the proposed area of quarry expansion (Fig. 3). DC Resistivity data were collected using an AGI SuperSting R8 IP Earth Resistivity/IP Meter with a 56-electrode cable. All data were collected using dipole-dipole geometry with a 12.8 foot spacing and were processed with RES2DINV© software from Geoelectric Imaging, Inc.

Table 1. Quarry cores examined, unit thickness and age at hole termination (TD).

Core	Overburden Thickness (ft)	Castle Hayne Unit B Thickness (ft)	Castle Hayne Unit A Thickness (ft)	Unit at TD
A-92	12	Abs.	86	K or P
B-92	2	Abs.	96	K or P
C-92	18	25	65+	E
E-92	18	30	55+	E
D-92	5	31	57	K or P
F-92	15	33	45	K or P
R-92	16	Abs.	82+	E
A-10-93	24	Abs.	74	K or P
A-11-93	5	34	64	K or P
A-13-93	12	Abs.	95	K or P
A-14-93	29	Abs.	84+	E
A-15-93	28	31	54+	E
A-16-93	15	Abs.	98+	E

E = Eocene P = Paleocene K = Cretaceous

GEOLOGY

Geologic units recognized in this area of Onslow County include the Cretaceous Peedee Formation, possible Paleocene Beaufort Group sediments, the Eocene Castle Hayne Limestone and surficial sediments (Fig. 4). Below, each are discussed with particular attention given to the Castle Hayne Limestone as this unit is mined in the quarry and is associated with sinkhole development in other parts of the North Carolina Coastal Plain (Brunswick, Duplin, Jones and Pender Counties).

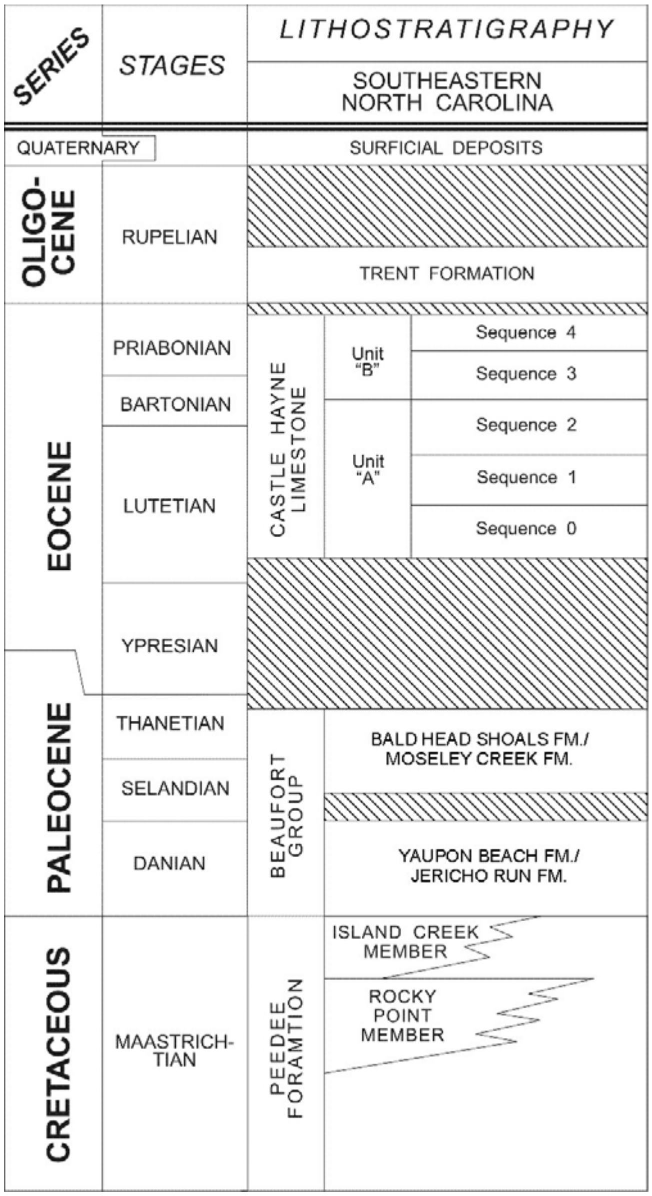

Figure 4. Upper Cretaceous and Paleogene lithostratigraphic units that occur on the Onslow block in NC. In the vicinity of the quarry in Onslow County the Peedee Formation and Castle Hayne Limestone are the only units identified.

Peedee Formation

The youngest Cretaceous unit recognized in North Carolina is the Upper Cretaceous (Maastrichtian) Peedee Formation. The Peedee Formation is disconformable on the Donoho Creek Formation of the Black Creek Group. The predominant lithology of the Peedee is dark gray to green, argillaceous, calcareous very fine-to-fine quartz sand, but occasionally, well-lithified, thin bioturbated calcareous beds occur in outcrop. In southern Brunswick County, however, the unit contains a moderately indurated, medium light gray to olive gray, very fine-to-fine sandy foraminiferal wackestone to sandy wackestone (Harris et al., 1986). The upper part of the Peedee has been divided into two members, the lower Rocky Point and the upper Island Creek (Dockal et al., 1998). The Rocky Point Member is a well-cemented sandy molluscan-mold grainstone to calcareous cemented quartz arenite to loose quartz sand. The contact between it and the underlying very fine to fine sand of the Peedee is gradational (Harris, 1978).

Disconformably overlying the Rocky Point is the Island Creek Member, or the youngest Maastrichtian unit recognized in North Carolina. The Island Creek consists of well-sorted, very fine to fine grained, poorly indurated, bioturbated, argillaceous, dolomitic quartz wacke. The calcareous nannofossil assemblage in the unit indicates that it correlates to the latest Maastrichtian (Fig. 4).

The Peedee Formation has been considered to be Campanian to Maastrichtian in age, and is assigned to the *Exogyra costata* zone. Recent work, however, by Self-Trail et al. (2002) and Harris et al. (2004) indicates that the Peedee Formation is Maastrichtian in age and the Campanian-Maastrichtain boundary lies between the underlying Donoho Creek Formation of the Black Creek Group and the Peedee.

The Peedee Formation represents deposition in an outer neritic, open shelf environment in the lower part, grading upward into an inner neritic environment for the Rocky Point and Island Creek Members. The Island Creek, however, is disconformable on the Rocky Point and probably represents a slight deepening event. The Island Creek contains faunal elements suggesting normal marine salinity and some that have a wider tolerance range. An inner neritic, low energy environment is indicated by the small size and delicate nature of most faunal elements (Dockal et al., 1998).

Beaufort Group

The Beaufort Group consists of four formations: the Danian Jericho Run and Yaupon Beach Formations, and the Thanetian Moseley Creek and Bald Head Shoals Formations. The Yaupon Beach and Bald Head Shoals Formations are only recognized offshore of Brunswick County near Cape Fear. The Jericho Run and Moseley Creek Formations are only recognized near Kinston, in Lenoir County (Harris and Laws, 1994) or western Craven County (McLaurin and Harris, 2001. Although Beaufort Group sediments have not been recognized in western Onslow County they occur in the northeastern part of the county. Consequently, the possibility exists that they may occur in the quarry area.

Castle Hayne Formation

The Castle Hayne Limestone occurs throughout eastern North Carolina between the Cape Fear and Neuse Rivers. Miller (1912) named the unit for exposures in the vicinity of Castle Hayne, New Hanover County, but a type section was not designated. Baum et al. (1978) designated the Martin Marietta quarry, three miles northeast of Castle Hayne, the lectostratotype and recognized three lithologic units: lower phosphate pebble conglomerate (fossiliferous packstone), a middle bryozoan grainstone and an upper bryozoan-sponge packstone. Ward et al.

(1978) identified three members of the Castle Hayne Limestone, the lower New Hanover Member (= Baum et al., phosphate pebble conglomerate), the Comfort Member (= Baum et al., middle bryozoan grainstone and an upper bryozoan-sponge packstone). Baum et al. (1978) identified a younger and different formation that Ward et al. (1978) identified as the Spring Garden Member of the Castle Hayne Limestone.

In order to alleviate some of the confusion over Castle Hayne nomenclature, Zullo and Harris (1987) identified five depositional sequences in the Castle Hayne Limestone, each separated by phosphatized and glauconitized disconformable surfaces. They designated the sequences from oldest to youngest, 0 though 4. Sequence 0 was only recognized in an outlier in Duplin County and is a sandy, bryozoan packstone to grainstone; it ranges up to 15 ft in thickness. Although the unit has not provided age diagnostic fossils, it is presumed to be Eocene based on lithologic similarity to overlying units that have provided age diagnostic species. Sequence 1 is widespread throughout southeastern North Carolina varying in thickness from a few inches to over 10 ft. This sequence consists of sandy phosphate pebble conglomerate, sandy calcarenite, dense sandy molluscan packstone, sandy cross-bedded bryozoan grainstone and bryozoan-molluscan packstone. These latter two lithologies are the common rock types in sequence 1. Sequence 1 contains the age diagnostic megafossils *Protoscutella mississippiensis rosehillensis* Kier, *Cubitostrea lisbonensis*? and nannofossils (Worsley and Laws, 1986) that suggest a middle Eocene (Lutetian) age for the unit.

Sequence 2 of the Castle Hayne Limestone has a similar distribution to sequence 1, but is more continuous and usually thicker. Sequence 2 is disconformable on sequence 1 of the Castle Hayne, older Paleocene units, or the Cretaceous Peedee Formation, and the disconformity is usually solutioned, phosphatized and glauconitized. Sequence 2 varies in thickness from about 3 ft to near 40 ft and consists of lithologies similar to those in sequence 1. As in sequence 1, sequence 2 also contains a large percentage of quartz sand. Age diagnostic megafossils in sequence 2 include *Protoscutella conradi*, *Cubitostrea sellaeformis* and the upper range of the pectinid *Chlamys clarkeana*. Worsley and Laws (1986) identified a calcareous nannofossil flora and fauna representative of the upper middle Eocene (Bartonian). Sequence 3 of the Castle Hayne Limestone is the most complete Eocene depositional sequence exposed in the North Carolina Coastal Plain. This sequence is widespread north of the axis of the Cape Fear arch in New Hanover, Pender, Onslow, Jones and southwestern Craven Counties. Sequence 3 is disconformable on sediments of sequences 1 and 2, but updip may overlie Paleocene or Cretaceous sediments. Lithologic units in sequence 3 are phosphate pebble bearing grainstone, bryozoan grainstone, molluscan-bryozoan grainstone, and bryozoan-sponge packstone-wackestone. Units attributed to sequence 3 differ from sequences 1 and 2 in their lower content of quartz sand, greater overall thickness and abundance of micrite (lime mud). Sequence 4 of the Castle Hayne Limestone has a restricted and discontinuous distribution and is only known from deposits in northern New Hanover County and Craven County.

Because similar rock and sediment types occur in all Castle Hayne Limestone sequences, and only one or two sequences are present at any single locality it is often difficult to distinguish one from another without the presence of mega- or microfossils. Therefore, in this paper, the Castle Hayne is grouped into two informal units identified as A and B.

Unit A – Unit A consists of lower Castle Hayne sequences 0, 1 and 2. These sequences are grouped into this rubric because of the ubiquitous presence of several percent quartz. Unit A is typically well-indurated, cross-bedded, cyclic in nature; it often has a patchy distribution. An abundant and diverse bryozoan fauna characterizes the unit; age-diagnostic megafossils and microfossils are commonly absent.

Unit B – Unit B consists of upper Castle Hayne sequences 3 and 4. These sequences are grouped together because they commonly have only a trace of quartz. However, these characteristics can only be used in certain areas as north along the Neuse River, quartz sand is a common component of the upper part of Unit B. Unit B is typically soft and poorly indurated in the lower part and better indurated in the upper part.

Overburden

Overburden in this report is defined as any post-Castle Hayne sediment at or near the land surface. The main sediments recognized include sand, clay, silt and organic-rich materials. These sediments may be of any post-Eocene age and represent deposition in a variety of marine to non-marine environments. Thicknesses vary from greater than 50 ft south of the quarry to less than 10 ft north of the quarry at the New River. Because of the highly irregular upper surface of the Castle Hayne, overburden thickness can change rapidly over a short distance.

RESULTS

Units Present and Lithology

Two lithologic units of the Castle Hayne Limestone occur in the active quarry, cores and outcrops north of the quarry on the New River. The lower unit (A) is the thickest and most widespread Castle Hayne unit in the area and the main rock mined in the quarry (Figs. 5, 6). Unit A consists of fossiliferous limestone (grainstone and packstone) and has varying degrees of hardness. Fossiliferous packstone to grainstone occur in a cyclic pattern with packstone (micrite-rich) occurring at the base and grading upward into grainstone (micrite-poor) in each cycle. Temporally and spatially the limestone is poorly consolidated (identified as a marl or soft limestone in core logs) or well-indurated (identified as limestone in core logs). Well-indurated limestone often occurs directly atop the underlying Cretaceous and/or Paleocene? sand/clay and forms the lower part of the quarry face. It has a thickness that ranges from less than 50 ft. (Core F-92) to over 98 ft. (Core Hole A-16-93). Small solution cavities (1-3 ft) occur in this unit, and are primarily enlarged laterally along bedding planes.

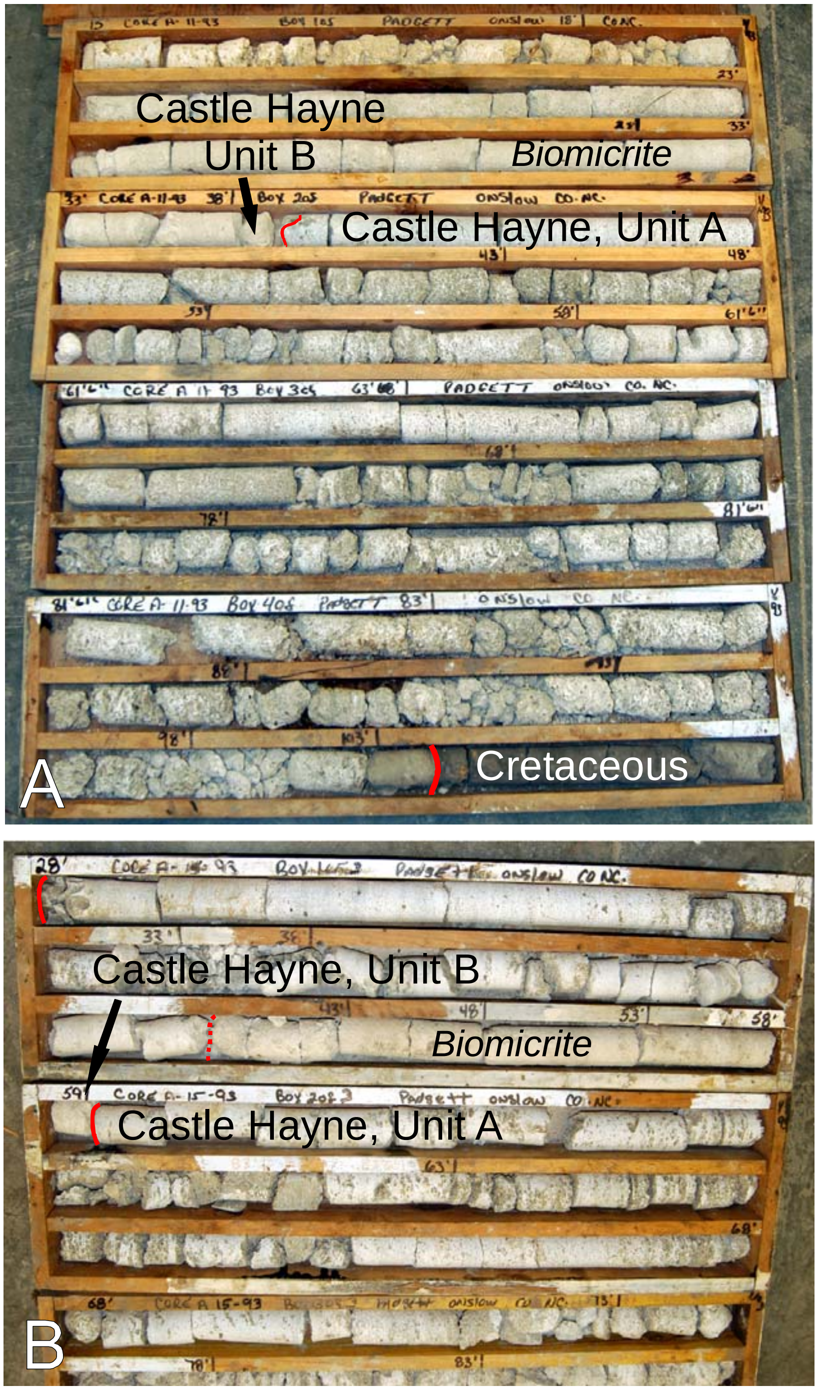

Figure 5. A. Castle Hayne Limestone units A and B in core A-11-93 from the west side of the proposed expansion of the quarry. **B.** Castle Hayne Limestone units A and B in core hole A-15-93 from the east side of the quarry. Note in unit B the lower tan biomicrite and the upper gray porous limestone. The top of the Castle Hayne is illustrated at 28 ft by a bored surface. For core hole locations see Figure 8 and Table 1.

The upper limestone unit (Unit B) is disconformable on the lower limestone (Unit A) and separated from it by an irregular phosphatized and glauconitized crust (Fig. 5). Unit B forms the upper part of the quarry face on the eastern and western sides of the active pit. The lower part of this upper unit is a soft fossiliferous micrite (mudstone/wackestone) and the upper part well-indurated limestone (wackestone/packstone/grainstone). This unit has a very irregular and karstic upper surface that is overlain by surficial sediments (overburden) of sand/silt/clay. Unit B contains large solution features (Fig. 7) that are larger (up to 10 ft? in diameter and height) than those in Unit A; they appear to be developed along NNE-SSW oriented fractures.

Figure 6. Unit A of the Castle Hayne Limestone, west quarry wall near sump. Note the cyclic nature of the unit based on variations in color, hardness and wall relief. Coffey and Read (2004) also recognized the cyclic nature of this part of the Castle Hayne Limestone at this locality. Quarry wall height is about 40 ft.

Figure 7. Unit B of the Castle Hayne Limestone on the east side of the quarry. Note the rounded nature of the rock and the large solution feature in the center of the picture below the person (outlined). The solution feature is partially filled with younger sand and clay (overburden).

Geologic mapping shows that both Castle Hayne units have a strike north and occur beneath U.S. 258; however, upper unit B is more prominent toward the east (Union Chapel Road) and west sides of the quarry. Unit A is prominent in the central part of the quarry. The distribution of Units A and B in the proposed area of the quarry expansion is illustrated in Figure 8. Of note is that Unit A, the older Castle Hayne unit, occurs below the overburden oriented in a north-south direction in the central area that is planned for quarry expansion. In addition, exposures on the New River several thousand feet to the northeast of the planned quarry expansion are also of Unit A of the Castle Hayne Limestone. Unit B occurs on the east and west sides of the planned expansion below the overburden and thickens to the east and west (Fig. 8). The pattern illustrated by the isopach of Unit B and the cross section below (Fig. 9) suggest that the active and planned quarry areas may be the site of a former larger tributary to the New River that has cut through Unit B into the top of Unit A. Figure 10 illustrates truncation of Castle Hayne Unit B by the overburden on the north wall south of Duffy Field Road in the active quarry.

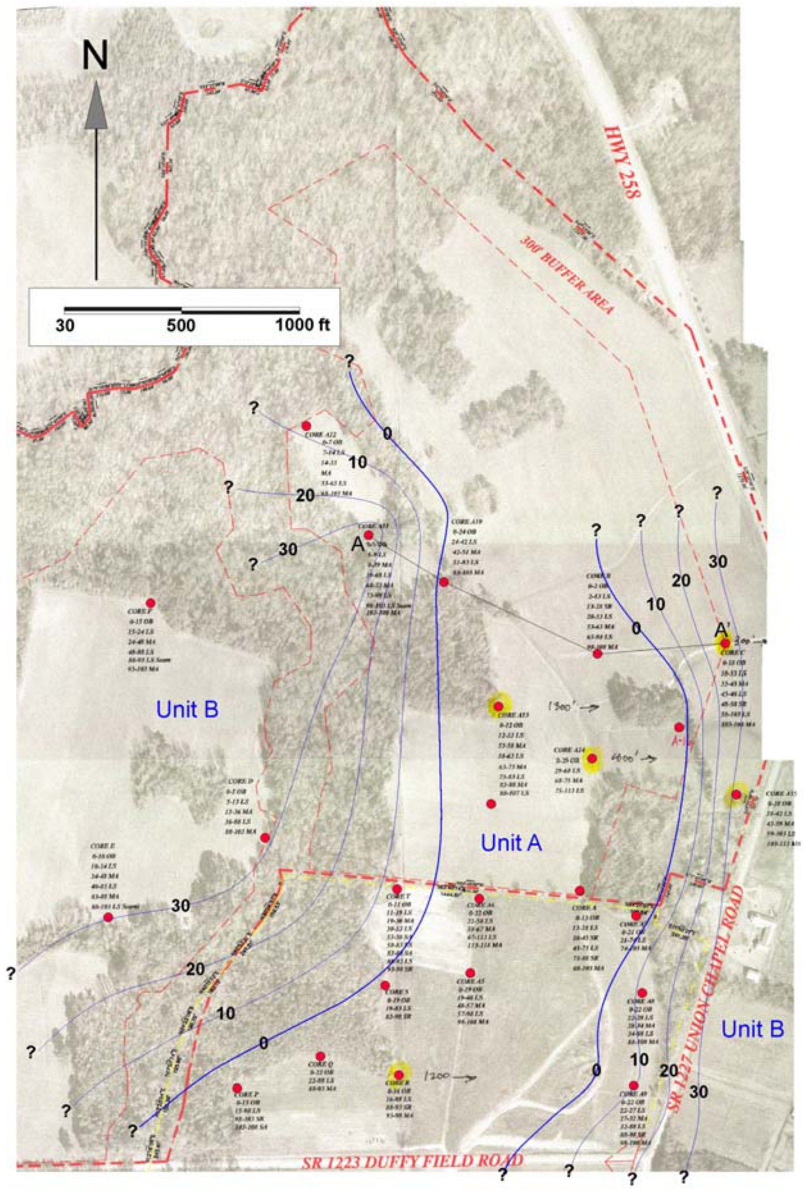

Figure 8. Isopach of Unit B of the Castle Hayne Limestone north of Duffy Field Road. Note that the lack of core hole control just south of U.S. 258 does not permit mapping the distribution and thickness of Unit B. Contours in ft.

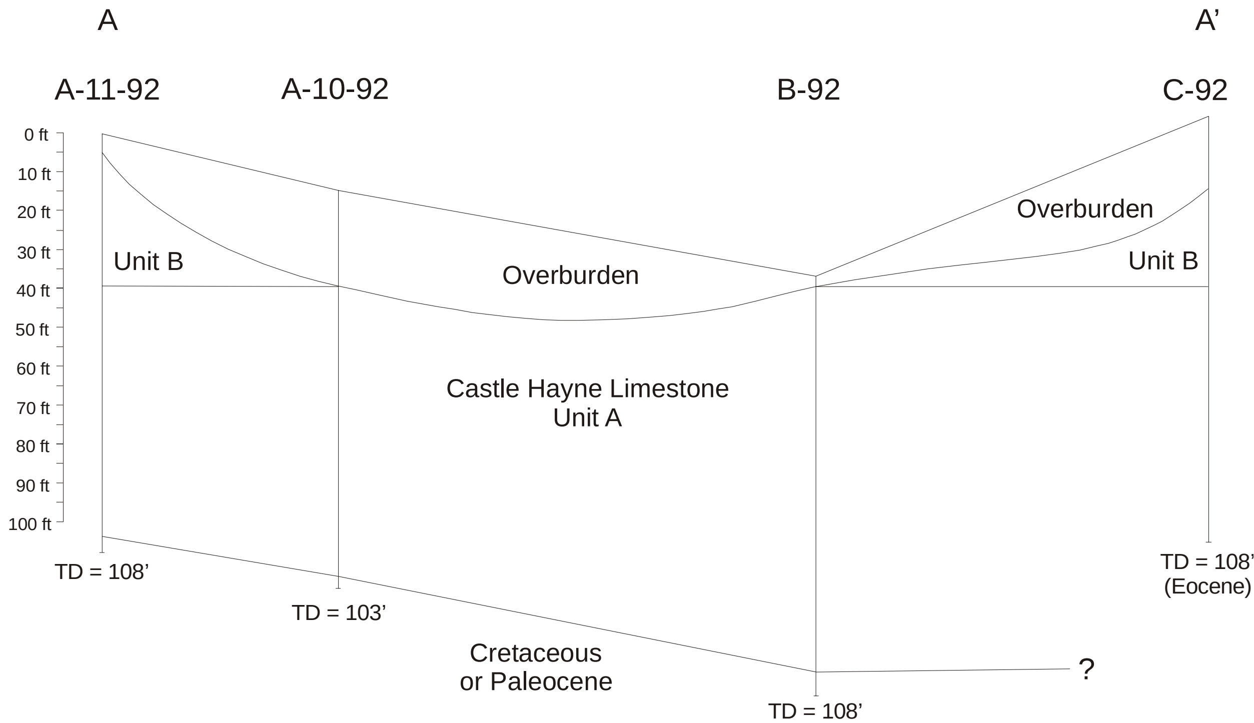

Figure 9. West to east stratigraphic cross-section illustrating the relationships of Unit A and Unit B of the Castle Hayne Limestone and the overburden. The line of cross section is shown in Figure 8. Vertical scale in feet. Datum is top of Castle Hayne Unit A.

Sinkhole Occurrence

Karstic features including sinkholes have been identified in the Castle Hayne Limestone in the quarry and in areas beyond the quarry perimeter (Union Chapel Road, Duffy Field Road, U.S. 258, Rhodestown Road, etc.). Sinkholes and karstic features occur in Unit A and Unit B of the Castle Hayne Limestone in the active quarry, but have different spatial characteristics depending upon the unit in which they occur. Solution features and the resulting sinkholes in Unit A are generally small (1-3 ft), parallel to bedding and concentrated in the upper parts of sedimentary cycles in porous and permeable grainstone, above less porous and permeable packstone/wackestone. Solution features and sinkholes in Unit B occur primarily in Castle Hayne sediments and rocks above the micrite (mudstone/wackstone) in the upper part of the unit. This lithology serves as an aquitard for water moving downward through the overburden into the fractured upper part of Castle Hayne (Fig. 11). Water that infiltrates through overburden into Unit B of the Castle Hayne does not enter the underlying middle to deeper aquifer of the Castle Hayne Limestone (Unit A) in areas where the calcareous mudstone/wackestone occurs. This water forms the shallow water table aquifer. Where Unit B is missing or the calcareous mudstone/wackestone is absent, water from the overburden moves into Unit A of the Castle Hayne. Based on lithology and weathering characteristics, larger, better connected solution features are more likely in Unit B of the Castle Hayne Limestone whereas smaller not as well connected solution features are more likely in Unit A of Castle Hayne.

A rectilinear set of fractures trending NNE-SSW and NNW-SSE occurs in well-lithified limestone of Unit A in the quarry (Fig. 12). Although it is not possible in the active quarry to map the distribution of karstic features (sinkholes), they are probably concentrated along these fractures. The possibility exists that fracture set intersections control the location of major sinkholes in both Units A and B of the Castle Hayne Limestone.

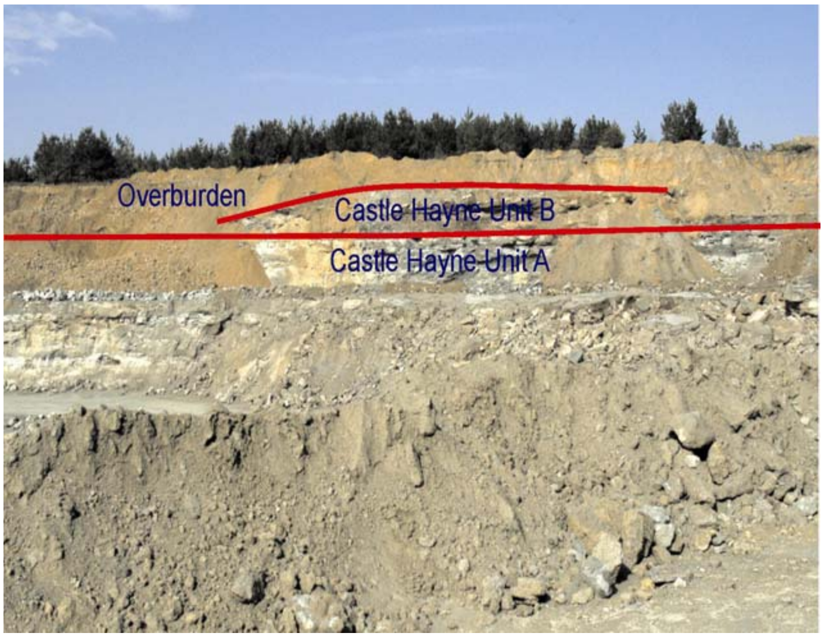

Figure 10. Truncation of Castle Hayne Unit B by overburden, north wall of active quarry, just south of Duffy Field Road.

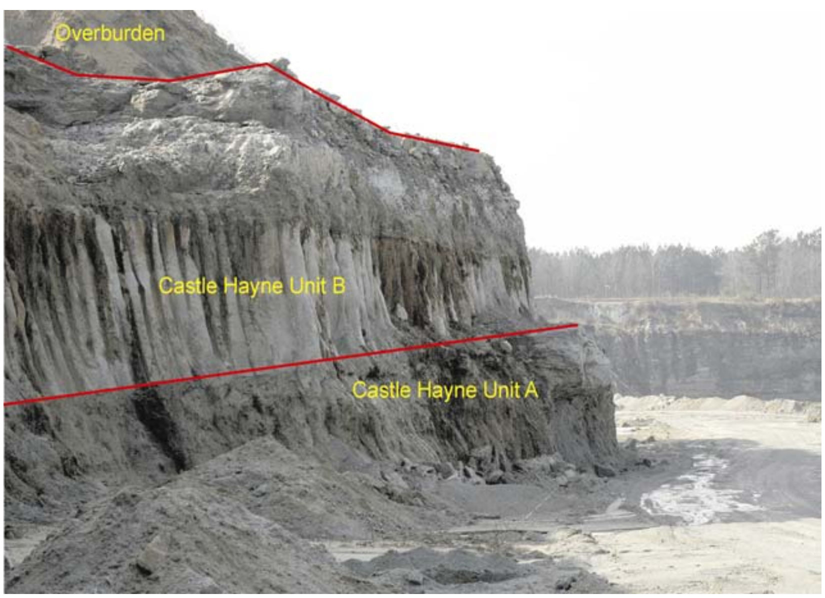

Figure 11. Castle Hayne Limestone Units A and B and overburden. Note the impermeable nature of the lower part of Unit B marked by water stains.

Geophysics

DC resistivity data from Union Chapel Road, U.S. Highway 258, and the proposed area of quarry expansion support and reinforce stratigraphic interpretations made from quarry, core and outcrop study. The southeast to northwest resistivity line along U.S. 258 (Fig. 13) indicates three distinctive resistivity layers. A lower high resistivity layer extends from the southeast to the northwest to about 1520 ft where it is disrupted and absent. An intermediate very low resistivity layer occurs above the high resistivity layer and extends from the southeast to about 840 ft where it ends. These two layers are interpreted to represent Units A (high resistivity) and B (low resistivity) of the Castle Hayne Limestone and suggests that Unit B is absent to the northwest beyond about 840 ft (Fig. 13). This also indicates that the Castle Hayne Limestone dips to the southeast supporting observations made in core study. The northwest end of the resistivity line along U.S. 258 suggests that Unit A of the Castle Hayne has undergone dissolution and overlying lower resistivity material has filled in the disrupted surface.

Figure 12. NNE-SSW and NNW-SSE rectilinear fracture set in Unit A of the Castle Hayne Limestone, west wall of the active quarry south of Duffy Field Road. Fracture set intersections may control the location of sinkholes. The view is to the south.

The southwest to northwest resistivity line along the west side of Union Chapel Road (Fig. 14) also delineates three resistivity layers. The lower high resistivity layer is similar to that recognized on the U.S. 258 line (Fig. 13) and is interpreted to represent Unit A of the Castle Hayne Limestone. This layer is fairly continuous along the line except around the 1200 ft position where lower resistivity material breaches the lower high resistivity layer of the Castle Hayne Limestone. This lower resistivity zone suggests the presence of sinkholes in Unit A of the Castle Hayne. This area is located near several small rounded depressions that occur along the east side of Union Chapel Road, which may represent the initial stages of sinkhole formation. In addition, the southwest end of the line along Union Chapel Road suggests another area of potential sinks as the high resistivity lower layer is also disrupted. This is also in the area where Duffy Field Road intersects Union Chapel Road and where sinkholes have been recognized (see Figs. 1, 3).

Overlying the lower high resistivity layer is a low resistivity zone that is interpreted to represent Unit B of the Castle Hayne Limestone. This unit appears to be present along the entire length of Union Chapel Road, north of Duffy Field Road, supporting the distribution of the Castle Hayne units shown on the isopach (Fig. 8). Differences in the resistivities observed on the lines along U.S. 258 and Union Chapel Road support the interpretation that micrite (lime mud) in the lower part of Unit B of the Castle Hayne retards the downward movement of water allowing the upper limestone to remain saturated thus reducing the resistivities.

Core hole A-15-93, located approximately 100 ft west of the resistivity line along Union Chapel Road, has been projected in to the line at 1380 ft. This core contains both Units A and B of the Castle Hayne Limestone with the boundary between the two located approximately in a transition zone between a lower high resistivity and upper lower resistivity (Fig. 14). Resistivity line 3, which was run in a field within the proposed area of quarry expansion, only records Unit A of the Castle Hayne Limestone collaborating the differentiation of the Castle Hayne Limestone mapped in the cores (Figs. 8, 15). In addition, the line also illustrates the southeast dip of the units (Fig. 15).

Ground penetrating radar (GPR) data along U.S. 258 show fairly regular and consistent reflectors with no distinct disruption. It is therefore difficult to interpret potential areas of sinkhole development based on the GPR data. At the northwest end of the GPR line along U.S. 258 there is thickening of what is interpreted as overburden. This thickening may reflect the area in the proposed expansion of the quarry where Unit B of the Castle Hayne Limestone is missing. If so, it probably represents a relict drainage valley that was a tributary to the New River to the north. Because the GPR data is inconclusive in providing recognition of the potential areas of sinkholes along U.S. 258, it is not illustrated in this paper.

SUMMARY

The following summarizes the findings of this study.

- The Castle Hayne Limestone underlies the Onslow Quarry and surrounding area; it is overlain by surficial sediments (overburden) that vary in thickness from 0 ft along the New River to greater than 20 ft in cores (A-10, A-14, A-15) from the proposed new quarry area; overburden thickness is greater to the south than the north.
- Sinkholes and solution features have been identified in the Castle Hayne Limestone in the quarry and also in areas beyond the quarry perimeter (Union Chapel Road, Duffy Field Road, U.S. 258, Rhodestown Road, etc.). Sinkholes beyond the perimeter of the quarry are interpreted to occur in the Castle Hayne Limestone.
- The Castle Hayne Limestone consists of two main lithologic units, a lower unit separated by a disconformity from an upper unit. The lower Castle Hayne unit underlies the entire quarry area whereas the upper unit is discontinuous occurring on the east and west sides of the proposed quarry expansion. The main area of the proposed quarry expansion north of Duffy Field Road will be into the lower Castle Hayne unit.
- Although both Castle Hayne units are lithologically similar, the lower part of the upper unit contains biomicrite that serves as aquiclude to downward movement of water.

Sinkholes and/or solution features occur in both Castle Hayne units. In the upper unit they are developed above the biomicrite and appear to be larger than sinkholes in the lower unit. Solution features in the lower unit are smaller based on quarry observations. The Castle Hayne unit, A or B, which occurs below the overburden is the main factor controlling sinkhole development and size.

- A rectilinear set of fractures that trend NNE-SSW and NNW-SSE occur in well-lithified limestone in the quarry. Sinkholes may occur predominantly along these fractures; however, no detailed sinkhole mapping or measurements of specific fracture orientations have been made. The possibility exists that fracture set intersections control the location of major sinkholes.
- The proposed location of the active quarry north of Duffy Field Road and closer to U.S. 258 will have an impact on water levels and the highway, with the possibility of sinkholes occurring concomitant with dewatering.
- A resistivity survey along U.S. 258 suggests that sinkholes are present northwest along the line as the continuity of a lower high resistivity zone is disrupted over the last 500 ft of the line. Consequently, the development of sinkholes is likely near U.S. 258 and beyond if previously formed solution cavities are present in the Castle Hayne Limestone in the subsurface.
- A resistivity survey along Union Chapel Road indicates two areas where sinkholes may be present because of disruption in the continuity of a lower high resistivity zone, one at the southwest end of the line and the other around 1200 ft.
- Information gathered in this study, along with additional information, was used to make recommendations concerning quarry expansion. The quarry operator met special conditions before modified permits were awarded allowing quarry expansion.

ACKNOWLEDGMENTS

The authors thank the Onslow County quarry operators for allowing access to the quarry, cores and other data from the area; special thanks is given to Steve Whitt for coordinating the efforts. Paul Thayer and Catherine Morris of UNCW are also thanked for reviewing earlier drafts of the paper and providing suggestions for its improvement.

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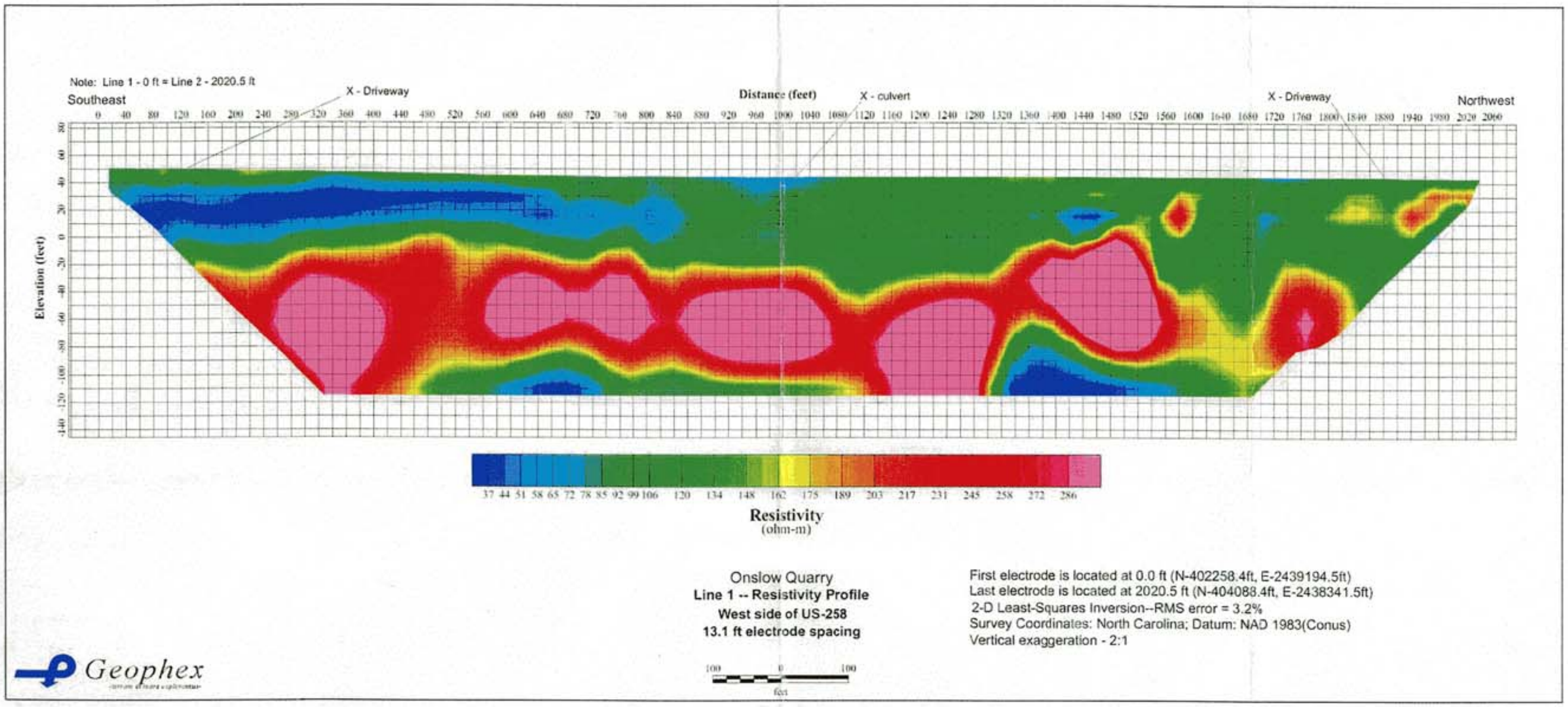

Figure 13. Southeast-northwest resistivity line 1 collected along U.S. Highway 258. Note the difference in resistivities between the lower part of the line and the upper part of the line. The differences are interpreted to represent Castle Hayne Limestone units A and B; note the southeast dip. The northwest end of the line displays resistivities that are significantly lower than those occurring to the southeast. The higher resistivities (>200 ohm-m) and the lateral continuity exhibited to the southeast are missing north of about 1320 ft. This area is interpreted to represent sinkholes where the overlying material has breached into the lower part of the Castle Hayne Limestone.

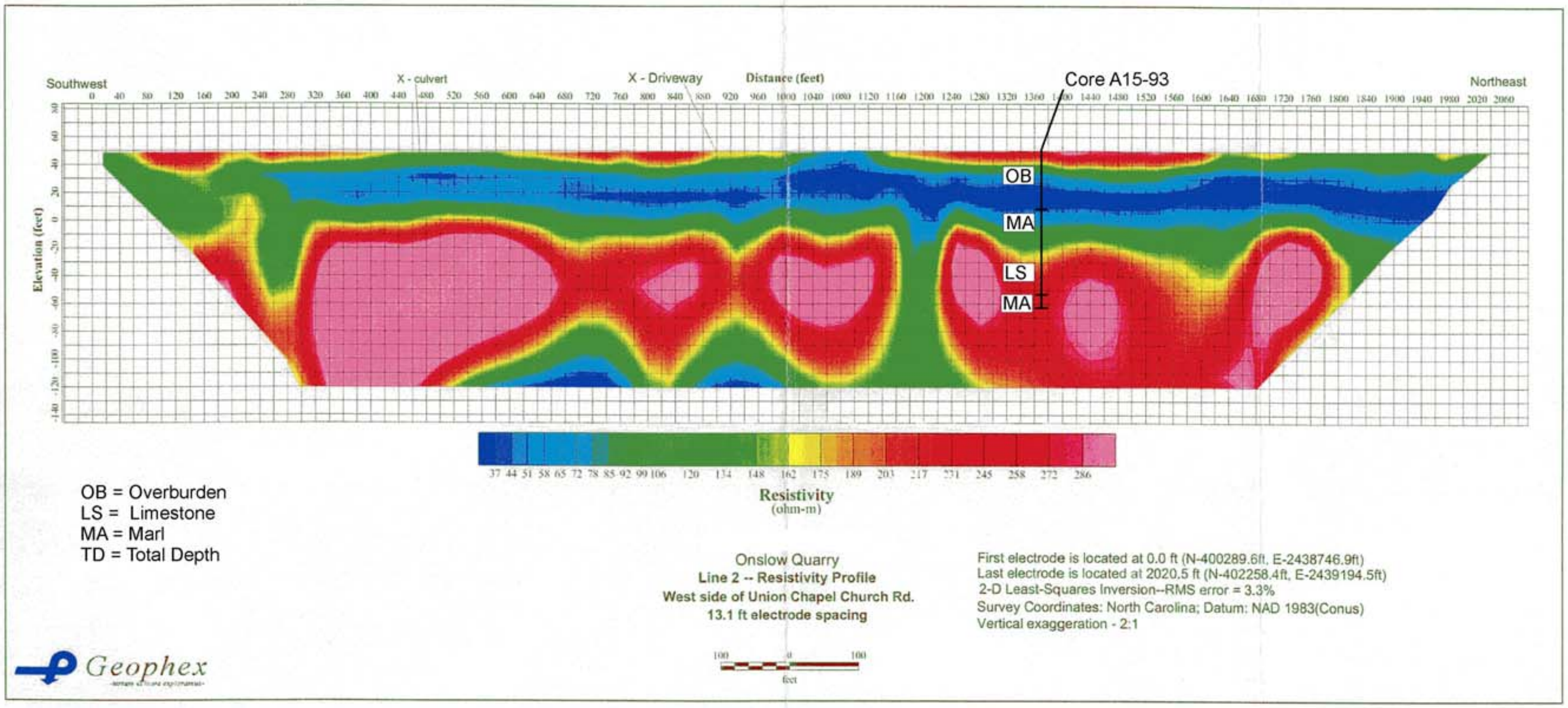

Figure 14. Southwest-northeast resistivity line 2 collected along the west side of Union Chapel Road. Note the differences in resistivities between the bottom and top of the profile indicating the presence of Castle Hayne Limestone units A and B. Core A15-93, which is projected into the line, indicates the division between the two units at approximately the first marl to limestone boundary. Also, note that on the southwest end (0-320 ft) and near the center of the line (1200') resistivities are significantly lower than those lateral (>200 ohm-m). These are areas interpreted to represent sinks where overlying material has beached the lower part of the Castle Hayne.

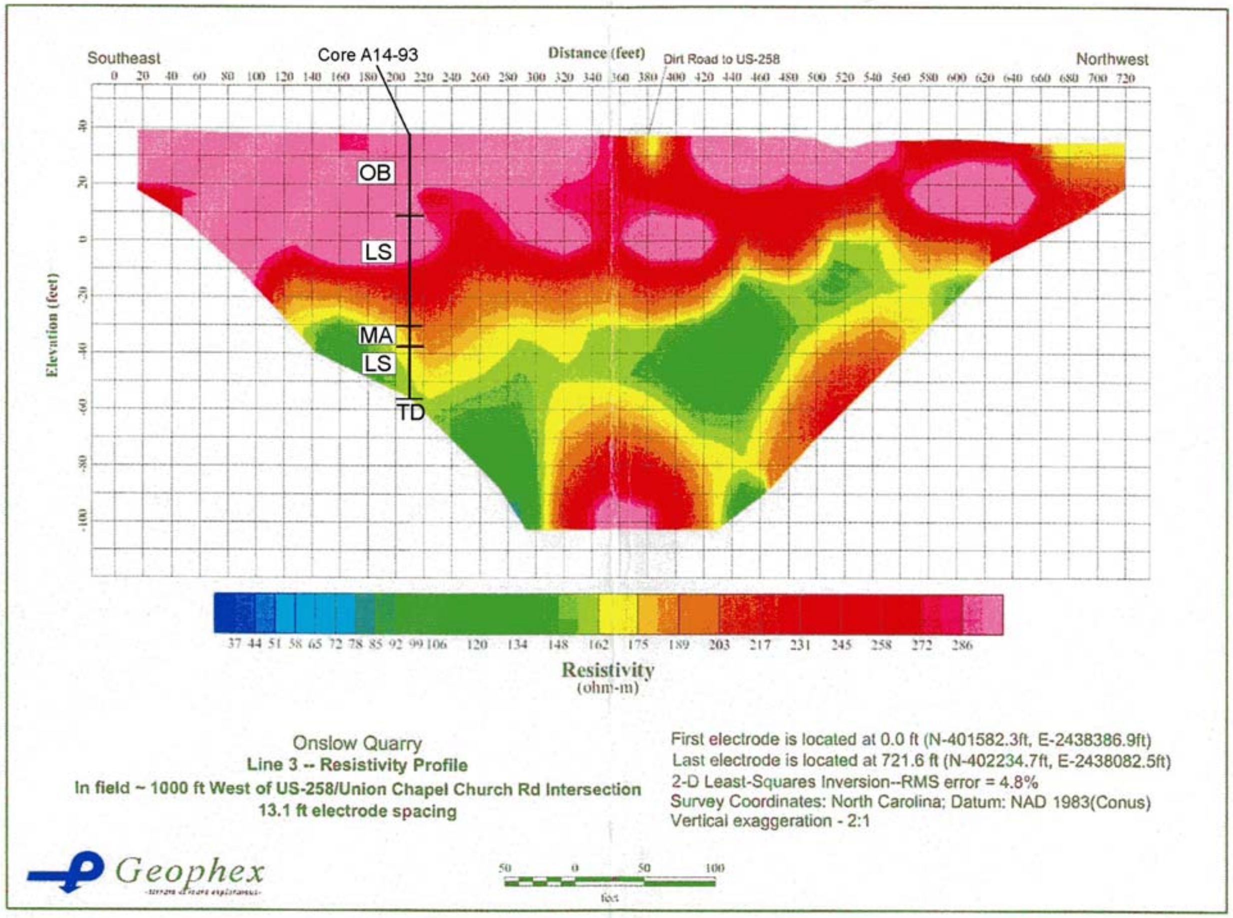

Figure 15. Northwest-southeast resistivity line 3 collected in the agricultural field west of Union Chapel Road where the proposed expansion of the quarry will occur. Note the southeast dip of the resistivities with the upper part of the profile having resistivities higher than the lower part of the profile. Core A14-93 provides, which ties to the line, indicates that Castle Hayne unit A underlies the overburden.

Rock slope stabilization, Decew #2 Generating Station, St Catharines, Ontario, CANADA

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ABSTRACT

The Niagara escarpment is known worldwide for the impressive Horseshoe Falls between Canada and the US as water rushes on its way down the Niagara River from Lake Erie to Lake Ontario. The Decew Falls generating stations were built along the escarpment, about 20 km to the west of Niagara Falls. Number 1 generating station was put into operation in August 1898 while the second hydroelectric power station was opened in the 1940s. The second plant was constructed in a large-scale excavation within the 80-metre high escarpment. Ongoing rockfall events have threatened the safety of plant workers and infrastructure at the Number 2 Generating Station, which houses a pair of turbines below twin penstocks. Ontario Power Generation called for Proposals in April 2004 to accomplish rock face stabilization measures primarily incorporating rockfall catchment fences and draped mesh.

Excavations in the sedimentary sequence for construction of the power house at the base of the escarpment had exposed rock material to weathering and degradation that led to numerous events over the years that deposited rockfall debris on the powerhouse roof and on both sides of the building. Janod put forward an alternative solution to controlling rockfall material and to protect the generating infrastructure. This incorporated very large ring nets hung from the crest of the slope and an intermediate bench, as well as double twist mesh to provide 100% coverage of the friable rock material exposed at the site. In order to fulfil their contractual obligations, Janod needed to have the design work checked and as-built drawings prepared.

Rock anchor pull tests were carried out to simulate the potential loading from a ring net installation prior to finalizing the design of the main anchors. The nets were hung in place using a helicopter in September 2004, followed by the placement of the rolls of double twist mesh. Horizontal cables were used across the base of the ring nets to act as braking elements to hold the nets and mesh in place should a large rockfall event take place – this would allow the ring net system to work in a similar way to a conventional rockfall catchment fence without the need for posts. Local treatments were also required for a number of isolated limestone blocks that were in a precarious condition near to the crest of the escarpment. The project was completed on time and on budget by the end of 2004.

INTRODUCTION

The Decew Generating Station is part of the hydroelectric power generating system developed from water falling over the Niagara Escarpment, see Figure 1. The Ordovician and Silurian rocks comprise a mixed sequence of detrital and carbonate rock types including shales, siltstones, sandstones and dolostones. The dolostones form the dominant cap-rock throughout the escarpment 'protecting' the underlying weaker materials and leaving a notable "cuesta" feature that extends for many hundreds of kilometres. Decew was developed in two phases, the latter construction leading to on-line generation in the early 1940s. In order to facilitate construction of the penstocks and powerhouse for Decew #2, a major slope-reshaping program was undertaken on the 80-metre high escarpment. Rockfall activity in the intervening years has led to concerns about safety of workers and operations around the turn of the century, and a plan was initiated to rehabilitate the rock face to provide a safer working environment.

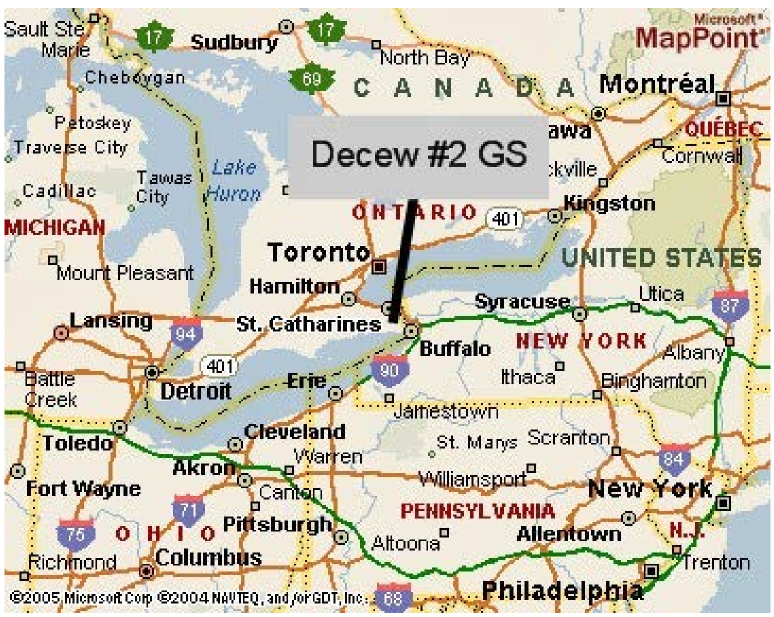

Figure 1. Site plan for Decew Generating Stations, St. Catharines, Ontario.

The Owner's design called for the installation of a number of rockfall catchment fences at different locations and elevations across the rock face. Specific energy absorption capacities were called for with fences of different heights in the various locations. Requests for Proposals were sent out in April 2004 that invited prospective proponents to submit proposals for stabilizing the rock slope and providing rockfall protection. Janod was awarded the project in June 2004 with an alternative solution using draped ring nets and double-twist galvanized mesh. Part of the bid required the use of professional rock engineering expertise to assist in the development of the project, to interpret the technical specifications and to prepare as-built drawings. David F. Wood Consulting Ltd. (Wood) provided these services.

This paper describes the project, provides information regarding the geology of the site, presents background information on hydro generation along the Niagara Escarpment, and gives details of the alternative design and construction methods. It concludes with commentary about the interpretation of the specifications and the overall success of the project.

BACKGROUND

The Decew Generating Stations are located in the city of St. Catharines, Ontario, some 21 kilometres (13 miles) due west of Niagara Falls. Twelve Mile Creek flows towards the northeast from the base of the escarpment having cascaded over what used to be known as DeCew Falls. The headwaters of the creek flow westerly from the area of the Welland Canal, through control structures on Lake Gibson and Lake Moodie before being split to pass through either Decew #1 or Decew #2 Generating Stations, see figure 2. The hydraulic head at the power stations is approximately 80 metres (over 260 feet).

Decew #1 Generating Station was put in service on August 25, 1898 with five (5) generating units controlled remotely from the main Sir Adam Beck II Generating Station near Niagara Falls. It generates approximately 23 Megawatts of electricity. Unit 1 of Decew #2 Generating Station was put in service October 1943; while Unit 2 entered into service in 1948, see Figure 3. These two (2) generating units are also controlled remotely from the main Sir Adam Beck II Generating Station near Niagara Falls, and they generate approximately 142 Megawatts of electricity. The total capacity of the generating systems in this area is approximately 2,000 Megawatts, as shown on Table 1.

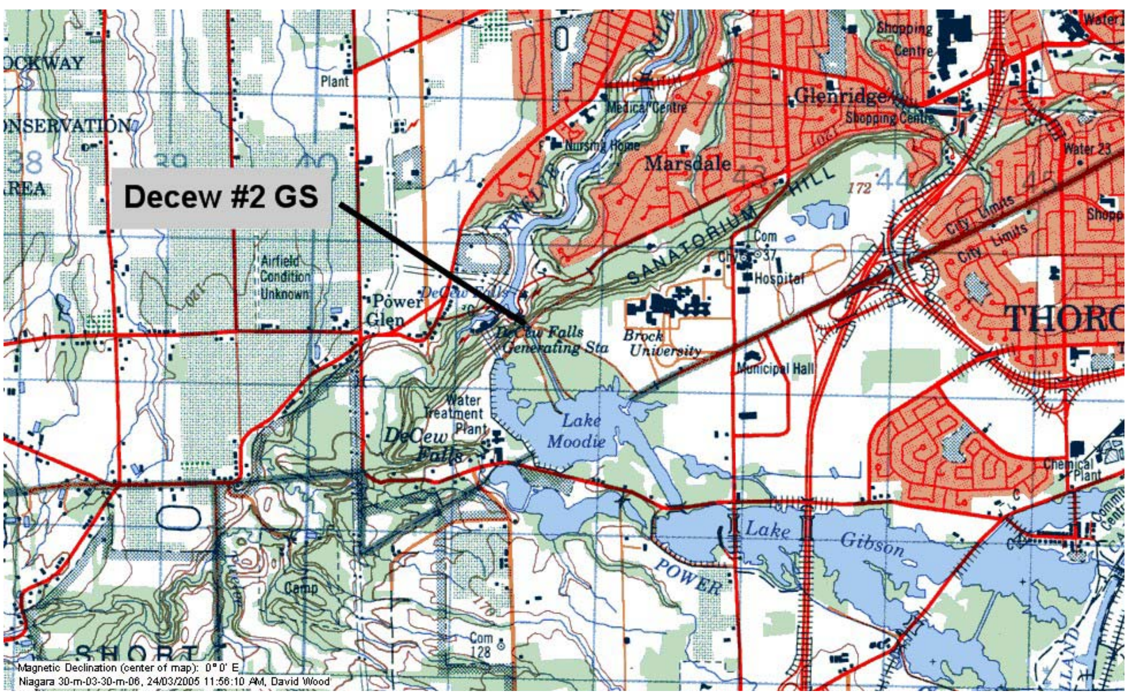

Figure 2. Location plan for Decew #2 Generating Station, St. Catharines, Ontario.

Figure 3. Aerial view of Decew #2 Generating Station showing major rock slope excavation from 1940s, courtesy OPG.

Figure 4. Photo-mosaic of Decew #2 GS rock face, courtesy Golder Associates.

Generating Station Name	Number of Generators	Output Capacity
USA		
Robert Moses GS	13 Generators	2,275,000 Kilowatts
Lewiston Pump GS - Reservoir	12 Reversible pump-generators	300,000 Kilowatts
Total Power Generation Capacity		**2,575,000 Kilowatts**
CANADA		
Sir Adam Beck #1 GS	10 Generators	470,000 Kilowatts
Sir Adam Beck #2 GS	16 Generators	1,290,000 Kilowatts
Sir Adam Beck Pump GS - Reservoir	6 Reversible Pump Generators	120,000 Kilowatts
DeCew #1 GS - St. Catharines	5 Generators	23,000 Kilowatts
DeCew #2 GS - St. Catharines	2 Generators	142,000 Kilowatts
Total Power Generation Capacity		**2,045,000 Kilowatts**

Table 1. Total power generation capacity for the Niagara Region schemes.

GEOLOGY & MORPHOLOGY

The site geology is only slightly different from the well-researched stratigraphy exposed 20 km to the east at Niagara Falls. The section exposed at Decew comprises approximately 80 metres of the 100 metres shown in Figure 5 from the upper part of the Upper Ordovician, Queenston Shale, through the Lower Silurian mixed sandstone and siltstones of the Whirlpool, Power Glen and Grimsby Formations, and up into the Middle Silurian with beds of massive dolostone set within finer grained sediments. The section at Decew reaches into the Decew Formation and the very lower parts of the Lockport Formation with dolostone cap rock along the crest of the cuesta.

The rock mass conditions are controlled to a large extent by the friable nature of the weaker sedimentary rocks, as well as limited surface weathering. Rock mass conditions could generally be described as: slightly weathered, finely bedded, dark reddish purple, fine grained, medium strong SILTSTONE, CLAYSTONE and SHALE with horizontal bedding and orthogonal jointing forming small blocks with fair surface condition, to fresh, blocky to massive, dark grey, medium grained, strong to very strong, SANDSTONE and DOLOSTONE with very large block sizes in good surface condition. Blocks of dolostone in the 20 cubic metre range are not uncommon. Some patterns of steeply dipping joints, perpendicular to the horizontal bedding can be seen locally, but there is little structural control to the overall failure mechanism that is dominated by ravelling of small, friable blocks and pieces of fine-grained sedimentary rocks.

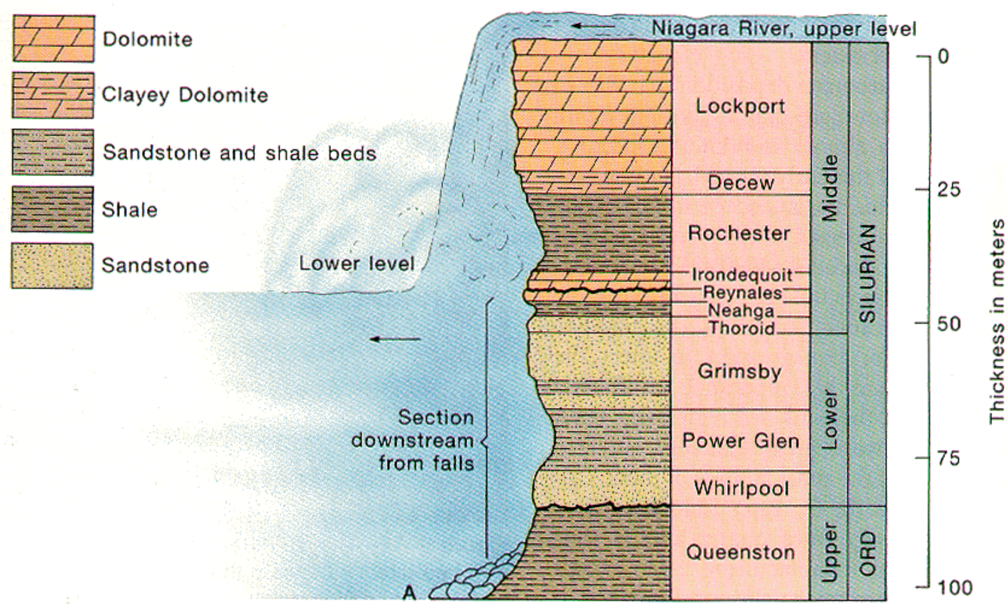

Figure 5. Generalized geology, type section from Niagara Falls, 20 km to the east.

Over the years since the rock mass was excavated for the penstocks and powerhouse of Decew #2, the weaker shales and siltstones have fretted away allowing superincumbent sandstones and dolostones to become detached in large blocks, which crash down the slope and sometimes strike the powerhouse building. Some damage has been inflicted on the building, and loose talus has also built up around the sides of the building. Safety has been a principal concern of the operators, both to workers and to the physical plant, so a program was developed to stabilize the rock slope.

The rock slope at Decew #2 Generating Station varies in shape across the exposed rock face. Close to the powerhouse, the rock mass has been excavated with almost vertical faces, while the upper slope and the outer slope away from the powerhouse has a face at about ½ to 1 (H:V) or 63^{o}. The full height of the works amounts to an elevation difference from 90.8 metres at the tailrace pond to 182.2 metres at the roof of the head-works above the penstocks. About 150 metres of slope length was involved in this project. A midslope bench had been created in the stronger sedimentary rocks at the base of the Middle Silurian, while the overall crest of the slope was in the Decew and lower Lockport Formations. The presence of the benches led to the proposed re-design of the stabilization treatment by Janod.

TECHNICAL SPECIFICATIONS

The RFP documentation[1] called for "rock slope stabilization and rockfall protection comprising rock excavation, rock scaling, rock bolt installation, wire mesh installation, rockfall fence

[1] "Request for Proposals for Decew GS Rock Face Stabilization", Ontario Power Generation, issue date April 16, 2004, RFP Number 6100000079.

construction at the Decew No. 2 Generating Station, located along the Niagara Escarpment in the City of St. Catharines, Ontario." Within the General Instructions section of the specifications, four separate submittals were required:

1. A method statement for carrying out the work safely that specifically addressed the issue of protecting workers from rockfalls during construction;
2. A detailed plan for accessing the various areas of the rock slope for the purpose of carrying out the work;
3. A schedule showing dates for delivery of materials and construction equipment as well as commencement and completion of the work; and
4. A proposal indicating how the work site would be isolated from hydro generating activities and other work areas.

The technical specifications continued with prescribed conditions for mobilization and demobilization, temporary facilities, rock scaling, rock reinforcement (including rock bolt testing), wire mesh and accessories, rock excavation and removal, rockfall fencing, environmental protection and safety. The accompanying drawings showed a general layout with a photo-mosaic of the slope, details of draped mesh, rock anchors, rock bolts, new rockfall fence locations and slope geometry. Details of the proposed works were presented on the general layout drawing, with additional details provided in the schedule of quantities, Table 2.

DESCRIPTION	UNIT	QUANTITY
Mobilization and Demobilization	Lump Sum	N/A
Rock Excavation (Trim)	m^3	100
Rock Scaling	Crew Hours	120
Rock Bolts (3 m length)	Each	75
Rock Bolt Performance Tests	Each	3
Rockfall Fences (including posts, cables anchors and accessories)		
2 m high, double-twist fence	m	120
5 m high, 1,500 kJ fence	m	65
5 m high, 2,000 kJ fence	m	50
6 m high, 2,000 kJ fence	m	75
Demolition and disposal	Lump Sum	N/A
Repair to new fences	Crew Hours	30
Talus removal from face	m^3	400
Draped Wire Mesh	m^2	3000

Table 2. Schedule of Quantities from RFP documents.

The notable feature of the proposed scheme involved installing four (4) rockfall fences. The first, referred to in Table 1 as double-twist fence and in the drawings as chain link fence with double-twist hexagonal mesh, was to be installed about 1-3 metres back from the crest of the midslope bench on both sides of the penstocks, some 50 metres on the east slope and 60 to 75 metres on the west. The second, 5 m high, lower capacity rockfall fence was planned for immediately upslope of the powerhouse and across the slope to the east of the powerhouse, some 60 to 65 metres in length. The 5 m high, higher capacity rockfall fence was to be located from the extension of the upslope powerhouse fence towards the west over some 45 metres, to protect the west side of the building from being struck by rocks falling from the intermediate bench. The 6 metre high, higher capacity fence was designed to extend laterally from both sides of the powerhouse close to road level (elevation 100 m) at to replace the existing 5 metre high fence and Jersey barrier structure protecting the roadway. Some 80 metres were identified on the drawings. Finally, draped mesh was planned from the crest of the slope to the top of the powerhouse, extending to the east and west of the roofline.

ALTERNATIVE BID

In their response to the RFP, Janod provided an alternative bid. The essential component of which was to replace all of the individual vertical rockfall catchment fences with draped ring nets and double-twist mesh. The basic logic behind the design was to remove the weakest component of the rockfall catchment system, which is the posts. By hanging the ring nets from the top of the slope to the base of the slope and then installing braking elements in the bottom 6 meters of the system the final product had no weak points. Since the specifications for the rock anchors were designed with rockfall catchment fences in mind and not draped ring nets, modification to the specified performance testing was required (see below). Six ring nets, banded and bundled, were shipped in individual containers; a total of 52 ring nets were supplied to the Decew site, see Figure 6.

Figure 6. Bundled ring nets at site, 14th July 2004

The common term “Ring Nets” refers to Anti-Attack Submarine Netting, constructed in the 1940s and 1950s to be used to protect harbours from submarine attack. Although production of these types of ring nets stopped after the war, production in Europe has been re-established due to a strong demand for these materials. Today they are used in various engineering applications, particularly in the fields of rockfall mitigation, debris flow and erosion control. The nets used on this project were constructed of 5 mm diameter steel strand woven into 304 mm diameter rings. One third of the net consisted of 12 mm diameter cable and two thirds of 9.5 mm diameter cable. The panels were approximately 22 metres by 11 metres and weigh roughly 1,814 kg. For corrosion protection, the wire is zinc coated and covered with “Cosmoline”, a rust preventative spray, giving a service life of decades.

Janod’s alternative bid required revised drawings to be prepared and provided to the Owner two weeks before planned mobilization in early July 2004. Existing fences were to be reinforced as the first order of business, followed by clearing the slope and crest of vegetation that might interfere with the placement of anchors and/or nets. In order to preserve as much of the existing vegetation as possible, Janod would consult with the Engineer prior to removing any large trees. Between mid-July and early August, it was planned to prepare an area to locate a crane for lifting components into place for the lower slope, and to drill off and install all crest and tie back anchors for the ring nets. Details of the alternative schedule of quantities are provided in Table3.

DESCRIPTION	UNIT	QUANTITY
Mobilization and Demobilization	Lump Sum	N/A
Rock Excavation (Trim)	m^3	100
Rock Scaling	Crew Hours	120
Rock Bolt Performance Tests	Each	3
Ring nets (including Anchors, Braking System and Accessories)	m^2	8008
Talus removal from face	m^3	400
Draped Wire Mesh and Accessories	m^2	8008
Freight	Lump Sum	N/A

Table 3. Schedule of Quantities from Janod’s Alternative bid.

CONSTRUCTION

Janod mobilized to site 5th and 6th of July 2004 and set up a site office for themselves and the Owner’s Engineer (Golder Associates, Mississauga, Ontario). A crew trailer was established at the crest of the slope. The boundaries of the work site were established and delineated, temporary storage areas identified, and other facilities introduced. A meeting was held with all employees to address site-specific safety hazards and emergency procedures.

The first engineering site visit was held on 14th and 15th July. The revised drawings were reviewed along with the original specifications to ensure that any discrepancies were identified. A field inspection of the midslope bench and the crest of the slope was made in order to initiate discussion about anchorage for the ring nets and appropriate testing to confirm design concepts.

It was immediately apparent that anchors might not be installed in vertical holes, but that their orientation should be based on loading requirements. This led to concerted discussion about the specified Rock Bolt Testing and modifications that might be made to better reflect the prototype usage of bolts – that is as ring net anchors rather than tensioned rock bolts to reinforce the rock mass.

Two large dolostone blocks had been identified, one immediately west of the penstocks and another within the wooded area to the west of the site just below the upper crest. These were inspected to determine what style of reinforcement or support would be needed to stabilize these blocks, see Figure 7.

Figure 7. Large, detached blocks of dolostone. Left hand block immediately west of penstocks (shown in foreground), right hand block at similar elevation in wooded slope to west.

A second field review was held on 22nd July, when mill certificates were checked, material strengths were confirmed and all parties agreed to a field version of the performance testing. In preparation for these tests, held on the midslope bench to the east of the penstocks, four (4) anchors had been installed exactly the same way as proposed for the works. The steel was #8 galvanized Threadbar, Grade 75 Dywidag Systems International (DSI). 3¼" diameter holes were drilled; two vertically, two inclined at 45^{o}. The holes were drilled, the anchors installed and grouted with SIKA 212 grout on Friday 16th July 2004, so the testing would be considered to represent six-day strengths. Figure 8 shows the layout of the anchor pull tests, while Figure 9 illustrates the loading frame.

The wire rope cable, identical to that planned for the ring net installation, was eye-spliced with a single cable clamp at the anchor end. The loading end was wrapped around a thimble through which a #11 bar was placed, and fed back towards the anchor. Four (4) cable clamps were used to secure the cable, as specified in the prototype. A one-tonne seating load was applied and a come-along used to take up the slack and pre-tension the system. Once the four cable clamps were set, the come-along was removed and the load dropped to ½ tonne. As the loading was increased on the hollow ram jack, deformation occurred rapidly although the load remained low. The grout strength was measured in the lab at 43 MPa at 5-days.

Figure 8. Four anchors installed six-days previously, reaction frame designed to be very stiff. Come-along used to take slack out of cables.

Figure 9. Loading arrangement to put cable into tension and transfer load to anchor bolt.

At about 4 tonnes load, the bar had bent approximately 45^o and the galvanizing was splitting, see Figure 10. The bolt failed moments later in bending with no discernable increase in load bearing capacity.

Figure 10. First pull test. Vertically installed bar had bent to 45^o under 4 Tonnes load.

It was observed that the “heel” of the eyebolt had penetrated the grout and acted as moment arm in the failure process, so for the second test a rock bolt faceplate was placed under the eyebolt to constrain its movement. This hole had 0.5 metres of free stressing length and 0.55 metres of grouted anchor. The test was run until the load reached approximately 11 tonnes without failure of the bar, see Figure 11. It was noted that this approach would work, but would use an additional 60-odd galvanized faceplates.

Although four anchors had been installed, three tests were required under the terms of the contract. The third test was undertaken on one of the inclined bolts. This hole was the same length as the other two but had only 0.28 metres of grout embedment, and almost 0.75 m of free stressing length. This anchor was successfully tested to 12 tonnes without failure. It was decided to stop the test at this load, since this was in excess of the design load by a considerable margin. Figure 12 illustrates how limited the deformation of the bar was at the end of the test. This is remarkable when compared to the other two tests. The outcome of the pull test program was a decision to install back anchors for ring nets in inclined holes where there was any concern about bond strength, bar deformation or load concentration. It was concluded that sharp angles should be avoided at all costs.

Figure 11. Test 2, vertical bar, restrained by using rock bolt faceplate. Load maintained to 11 tonnes without failure.

Figure 12. Test 3, inclined bar, also restrained. Load increased to 12 tonnes without failure, or notable deformation of bar.

While Janod was working on site, the topic of ice loading was brought up by one of the workers in the power station. Because of the nature of the rock mass, water seeps through the bedding planes and this causes ice to build up on the face over the winter. Although this topic had not been brought up in the original proposal or subsequent discussions, Golder and Wood ran calculations on the anchor and cable support system to confirm that it would be able to handle the additional weight of ice. Not only was the anchor and cable support system put forward by Janod able to handle the extra weight of the ice, there was a Factor of Safety of approximately 5.

Towards the end of August the crew was ready to start hanging the ring nets. The crane was not intended to be used for net hanging and a helicopter was chartered for this work. Wood made a third visit to the site on 2nd September when National Helicopters' Bell 211 Long ranger, C-GNHX, was used to sling the nets from a spreader bar having had the ring nets laid out on the road near the powerhouse. The bottom of the slung net was attached to the upper anchor cable with 5 or 6 shackles. Some side restraint was added for certain panels, then the helicopter was used to drape the ring nets over the face from the top down. Additional side cables were attached to some nets. A worker lower down the face would then detach the spreader and the ground crew hooked up the next ring net to be placed. Figure 13 shows the crew manhandling a ring net panel from the midslope bench.

Figure 13. Ring net panel hanging from helicopter while ground crew attaches end of net to anchor cable.

The crew worked diligently to place the ring nets, and then to sling the large rolls of double-twist hexagonal mesh to the crest of the slope. Over the next couple of weeks, an enlarged crew worked to secure the ring nets, added another horizontal cable for additional security, slung the double-twist mesh, hog ringed the mesh but not to the ring nets, scaled out loose rock from below the ring nets, and installed horizontal braking elements to allow the ring nets to deform if loaded with a significant quantity of broken rock.

Since the capacity of the individual rings was 10 tonnes, the braking elements were set to engage at 10 tonnes to ensure that there would be no system damage in the case of a large-scale rock mass failure. By draping the ring nets any material coming down the slope would be prevented from generating as much energy as would be the case with free-falling rock. The braking elements were installed horizontally across the whole draped system; one at the base, one at 3 meters from the base and one at 6 meters from the base. Each of the elements was set to engage at a load of 10 tonnes but it is assumed that the lowest braking element would not be engaged unless there was a massive failure in the slope. Another advantage of the system was maintenance; since it was evident that the slope would continue to weather and have small-scale ravelling failures it was important that the failed material could be safely and economically removed. In order to get rid of the material from the base of the slope, the braking elements can be easily loosened to allow the nets to be lifted up, so an excavator could safely remove the fallen material. Once all the material has been removed, the cables would be placed back into the braking elements, which would be re-torqued to the design load without having workers exposed to falling rock.

Re-vegetation of certain areas was carried out, sumac cuttings were transplanted, and the area rehabilitated to the Owner’s requirements. The final cleaning out of loose rock from behind the powerhouse and to the west of the powerhouse was undertaken, and the old fence/Jersey barrier protection at the roadway was rebuilt. Drainage was re-established at the base of the rock slope to the east of the powerhouse and small riprap was placed at the toe of the slope after drainage had been re-established.

A joint inspection of the site to review the project with the Engineer and Owner’s representative was carried out at the end of September when 95% of the works had been completed. The stabilization of the two large loose dolostone blocks was reviewed, the placement of shotcrete support to loose blocks near the head works was confirmed, and all other aspects of the stabilization program were evaluated. Minor deficiencies were identified and Janod corrected these within the next few days.

CONCLUSIONS

Janod demobilized from the site at the end of September 2004 having completed the stabilization of the rock face at the Decew #2 Generating Station on time and on budget. Design changes incorporated in Janod’s alternative bid meant that the original specifications required reinterpretation. Manufacturers’ recommendations were followed for all materials that differed from those originally specified and the Engineer and Wood worked effectively to revise the rock bolt pull test specification to reflect the loading of the prototype. The savings to the Owner though implementation of the alternative proposal amounted to some US $350,000 compared to the original proposal, which was valued at about US $1.2 Million.

As-built drawings were prepared over the next couple of months and a final site visit was made on 6th December 2004 to confirm a few outstanding issues. Final drawings were released to the Owner once all of the established protocols had been met. The system has gone through its first winter and OPG has contacted Janod to tell them that “the system is working beautifully”.

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The authors would like to recognize the assistance of OPG in managing this project; Tony Palma’s role was as Project Engineer, Michael Dudycz acted as Supply Chain Representative. The dedication of the Janod workers during the construction program is also gratefully acknowledged; Todd Reccord acted as office liaison, Pierre Gionet was the site coordinator. National Helicopters provided a skilled crew to work under quite difficult conditions flying in the ring nets and double-twist mesh rolls. The authors would like to thank John Pilipchuk and the Host Committee of the 56th Highway Geology Symposium for the opportunity to present this paper.

The Use of Graded Solid Rock for Rock Pad and Rock Embankment Construction along Highways in Karst Areas of East Tennessee

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Abstract

Highway embankments constructed across karst terrain in East Tennessee encounter numerous sinkholes and depression areas which require appropriate design and construction methods. Past experience in constructing highways in these karst areas shows that using graded rock pads and graded rock embankments in these sinkhole areas leads to a higher quality of long term stability. In addition, by using these graded rock embankments and rock pads, the quality of highway runoff drainage is greatly improved before entering the groundwater system through the affected sinkholes.

A recent TDOT roadway project along State Highway 66 in Hamblen County (East Tennessee) used graded rock pads and graded rock embankments to cross numerous sinkholes. Design and construction plans were developed that identified selected sinkholes and the appropriate remedial design to be used. Construction of the rock pads and rock embankments required using both rock excavated from the project and rock processed from nearby quarries.

The effect of using the graded rock pads and embankments on the roadway is to provide greater stability for the roadway and improved water quality for the runoff before entering the groundwater via the sinkholes.

Introduction

Landscapes of gently rolling hills and valleys textured with sinkholes and depressions, cave entrances, sinking streams and outcroppings of weather-beaten limestone picture our thoughts of areas typically known as East Tennessee karst. The recognition of areas of

The reactive approach to dealing with karst problems involves responding to local catastrophes in an emergency situation. Snap decisions are often made and remedial approaches taken that are usually conservative in nature, costly, and most often does not solve the overall problem.

Remedial action that is usually adopted in response to a karst type problem involves bridging, drainage, and relocation concepts. Bridging a collapse with a rock fill or a concrete structure may be considered. Trying to solve flooding problems may require the use of existing sinkholes for drainage outlets or even constructing special ditches to permit positive flow from a sinkhole area to a nearby stream. Sometimes moving a section of road or relocating a house is the course of action required.

If true consideration is given to the karst problem in advance of a construction project, then proactive measures can be taken. Simply avoiding a karst area in planning a highway or developing a commercial zone or residential subdivision can save future agony as well as dollars. If avoidance is not possible, then there may be certain measures taken during the design and construction of a project to lessen the impact of the activity on the karst regime.

This paper discusses the proactive approach to highway design and construction in a karst landscape of East Tennessee. Being situated in the Valley and Ridge Province of East Tennessee, the roadway project described in this discussion is located in the central part of Hamblen County near Morristown, between SR 160 and U.S. 11-E (Figures 1 and 2). The landscape consists of rolling hills with numerous closed sinkholes and internal drainage, typically karst terrain. Surface streams are absent.

The project site is underlain by dipping carbonate strata (mostly limestone) of the Conasauga and Knox groups. The limestone tends to be well jointed and typically exposed at the surface. The strata composition varies from argillaceous limestone to dense fine grained aphanitic limestone and dolostone, and in places contains high percentages of calcium carbonate.

No locally known caves are found along the project limits. However, a cave system of some degree must be present to have developed the surface karst features found along the project site. Surface runoff filters down through the sinkholes and into the groundwater system providing a recharge area for local springs and wells.

Design and Construction Plans

The roadway plans used on the subject project were developed by Campbell and Associates (Knoxville, Tennessee) as a consultant to TDOT. Geotechnical design work was performed by S&ME, Inc (Knoxville office). Both consultants were overseen by the appropriate TDOT personnel.

Karst related concepts involving graded rock pads and graded rock embankments were derived from the “Sinkhole Treatment Standards” sheet developed by the TDOT active karst subsidence and collapse is of considerable importance to those engaged in construction; especially the construction of infrastructure such as highways and bridges.

Some of the problems that have resulted from humans developing areas of karst terrain into subdivisions, highways, and commercial development include: subsidence beneath house foundations, the collapse of yards due to leaking swimming pools, or inappropriately located septic tanks, the collapse of highway surfaces, ditchlines and bridge foundations, and numerous instances of flooding. In most instances it is the impact of human activity that induces a collapse of a highway or a house or results in the flooding of a commercial strip mall.

Today with the numerous toxic and hazardous substances that are found throughout our society, damage to groundwater supplies in karst areas are also of concern, and until recently has not been adequately addressed. In addition, contamination from highway run-off falls in this category and is also an issue to be addressed. This is particularly so in karst areas such as in Middle and East Tennessee where toxic or hazardous spills along highways can directly flow from the highway into sinkholes and cave systems.

Karst problems along Tennessee highways have previously been described by Royster (1984), and Moore (1981, 1984, and 2003).Although not directly related to using graded rock embankments, Moore's study (1987) involved the analysis of 72 karst related subsidence and collapse problems experienced along highways in East Tennessee over a ten year period (1976-1986). The data collected in the study indicated that of the 72 sinkholes researched, 85% were "induced", while 15% were considered "natural". The most important result of the study was the revelation that 74% of the karst problems occurred in roadway ditchlines. The remaining 26% occurred in roadway subgrades and in areas unrelated to highway facilities (fields, yards, woods) (11% and 15% respectively).

The majority (93%) of the ditchline problems studied occurred along untreated roadway ditches. Untreated ditches are defined in this study as being standard roadway drainage ditches which are constructed without the benefit of pavement or other impervious materials.

In 2003 Moore updated the 1987 study by analyzing 163 cases of sinkhole collapse incidents in east Tennessee between 1969 and 2002. Of the 163 sinkhole incidents studied, 86.5% of the sinkhole occurrences were located in highway ditch lines (Moore, 2003). The 2003 study also supported the findings of the 1987 study by showing that of the ditch line collapse incidents analyzed, 93% also involved unlined ditches (the same amount disclosed in the 1987 study).

The bulk of the activity concerning these types of karst problems has been reactive in nature. This would include fixing a roadway after it has experienced a collapse that might have resulted in possible injury to motorists.

Geotechnical Engineering Section. In addition, past experience using these concepts were also used in the conceptual design including karst problems involving collapse and sinkhole flooding along Tennessee highways which have been previously described by Royster (1984), and Moore (1981, 1984). In addition, Sowers (1976), Newton (1976), Foose, et al (1979), and Amari and Moore (1985), and Moore (1987, and 2003)), have detailed possible geotechnical solutions to these karst problems in the Valley and Ridge Province from Alabama to Pennsylvania.

One of the more widely used concepts for constructing roadways across karst terrain is the use of graded rock pads and embankments. The graded nature of the shot rock material removes most of the fines and allows the larger rock pieces to have interlock with each other providing stability for the embankment. When used in thin soil areas where bedrock is exposed at the surface the rock pads and embankments can serve as a “bridging” element over the karst feature.

This discussion does not address the problem with soil voids developed in the residual soil over cavitose bedrock. These soil voids will typically collapse when the soil arch looses sufficient thickness to arch the open space in the soil. Drumm and Yang (2005) discusses the arching ability of soils and the residual soil stability in karst terrain.

In addition, the graded rock pads and embankments allow surface water to continue to flow into the existing sinkhole area thereby continuing to recharge the local groundwater regime. The graded rock also serves as a filter for larger debris such as trash, tree limbs and leaves.

The graded rock specification used in Tennessee is as follows:

Graded Solid Rock shall consist of sound, non-degradable rock with a maximum size of 1 meter (3.3 feet). At least 50 percent of the rock shall be uniformly distributed between 300 millimeters (1 foot) and 1 meter (3.3 feet) in diameter and no greater than 10 percent shall be less than 50 millimeters (2 inches) in diameter. The material shall be roughly equi-dimensional in shape. Thin “slabby” material will not be accepted.

The contractor shall be required to process the material with an acceptable mechanical screening process that produces the required gradation. When the material is subjected to five (5) alternations of the sodium sulfate soundness test (AASHTO T 104), the weighted percentage of loss shall be not more than 12. The material shall be approved by the Engineer be fore use.

In most instances the rock material is usually limestone or dolostone. If available near the project then sandstone or granite meeting the above specifications may be used. The purpose of the graded rock specification is to ensure that rock is resting against rock and not “floating” in a soil matrix.

A total of 16 sinkholes were to be treated on this project by using graded rock pads or graded rock embankments. These were identified on the construction plans both by station number and by mapping location. The typical drawing illustrating the desired design concept for treating the sinkholes was also included in the construction plans. Two design conceptual drawings were formulated to mitigate the sinkholes on the project: 1- a rock pad/embankment filling the selected sinkhole (Figure 3), and 2- an extension of the rock pad/ embankment outside of the normal roadway template for sinkholes that were receiving runoff from areas not filled in by the roadway embankment (Figure4).

Construction

The subject project which is SR 66 from SR 34(U.S. 11-E) to SR 160 in Hamblen County, was let to contract by the Tennessee Department of Transportation on December 5, 2003. The low bid was $4,242,724.44 submitted by Charles Blalock and Sons, Inc. of Sevierville, Tennessee. Construction work on the approximately 1.4 mile long project began on February 18, 2004.

After the necessary clearing and installation of required drainage structures, construction of the graded rock pads and embankments began. The rock material was obtained from both a nearby quarry operation and from the project. Rock excavation on the project required that the rock material be processed to meet the required specifications. The rock excavated from the project was limestone and dolostone with some of the material being very shaly limestone. The shaly limestone was not used for the graded rock embankment material due to not meeting the soundness specifications.

The sinkholes to be treated with the graded rock fill material were identified in the field prior to placement of the fill material (Figure 5). Debris as well as brush and trees were also cleared. Any openings were immediately protected from surface run-off by placing silt-fences around the open throats of the sinkholes.

First, the bottom of each sinkhole area was covered with a geofabric material to prevent fines (soil) from being eroded and washed into the throat of open sinkholes (Figure 6). Next the graded rock material was then placed into the sinkholes and constructed up to the require subgrade elevations (Figure 7). A minimum of 300 millimeters (1 foot) of No. 57 stone was then placed over the graded rock material to provide both a “choker” layer for any soil embankment material placed over the rock embankment and to also provide for additional filtration for surface runoff.

In most instances the graded rock material was placed directly on exposed bedrock which, combined with the interlock of the graded rock material, provides the necessary stability for the overlying embankment. Some of the rock pads and embankments were “topped-out” with common excavation consisting mostly of clay and weathered shaley limestone.

Some of the graded rock fills were constructed up to roadway subgrade where base stone will be placed on top of the rock fill.

Placing the geofabric material into and around each sinkhole can be problematic, especially during windy weather. This is usually a hand labor operation and can be neatly performed without excessive effort.

Placing the rock onto the geofabric requires careful attention and expert equipment operation. Often times placing the graded rock into the sinkhole can rip or move the geofabric requiring added labor in reorienting the fabric.

Once the graded rock fill is in place, then the rock embankment can begin to operate as designed. Normal construction activity can proceed with the required erosion control measures to guard against siltation.

Summary

A total of 16 sinkholes were treated with the graded rock embankment concept. This proactive approach has resulted in providing stability for the new roadway embankments and has also provided a primary level filtering mechanism to reduce contamination of the area groundwater by surface runoff from the roadway.

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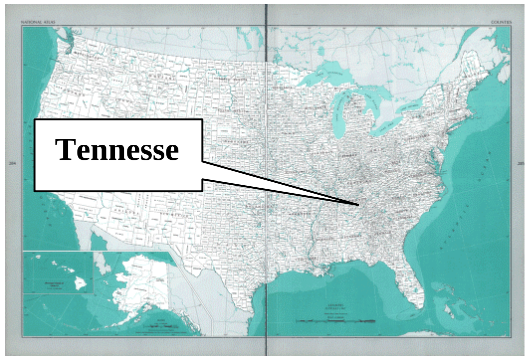

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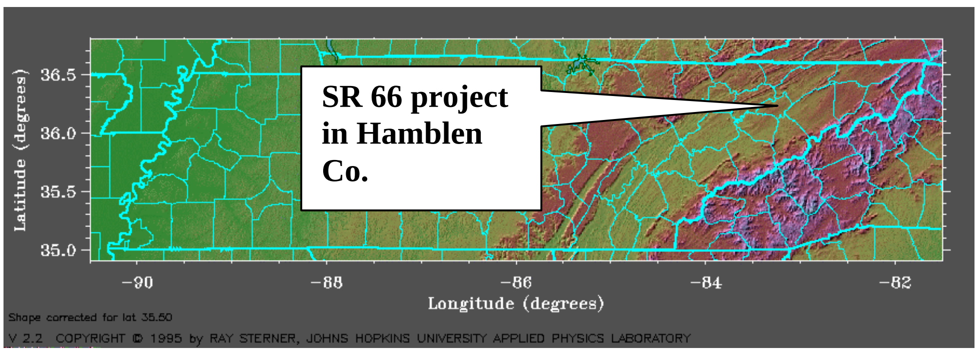

B

Figure 1. Location of SR 66 project in Hamblen County, Tennessee. (A-US map; B-Tennessee map)

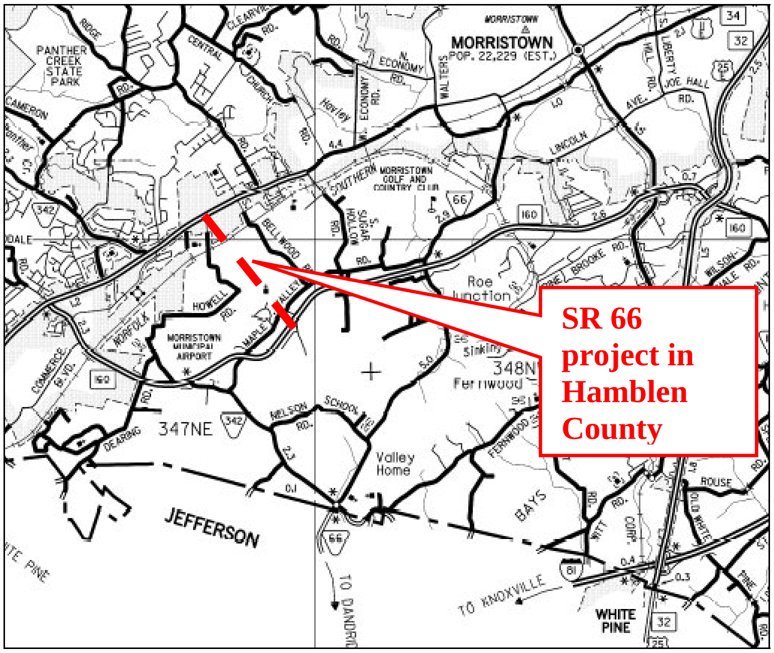

Figure 2 . Detail map of SR 66 project in Hamblen County, Tennessee (in the city of Morristown).

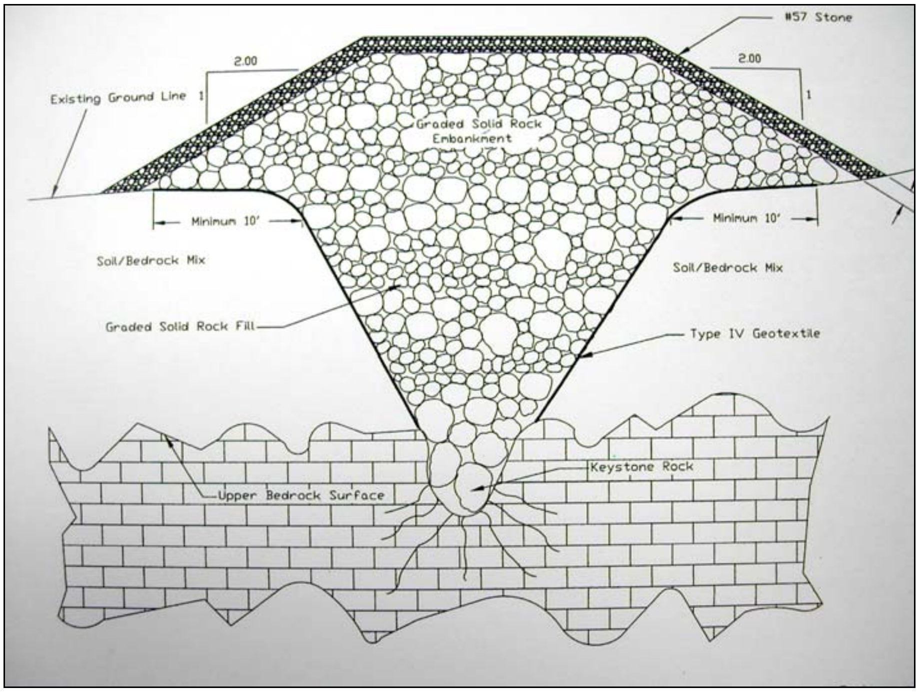

Figure 3. Treatment concept for filling entire sinkhole.

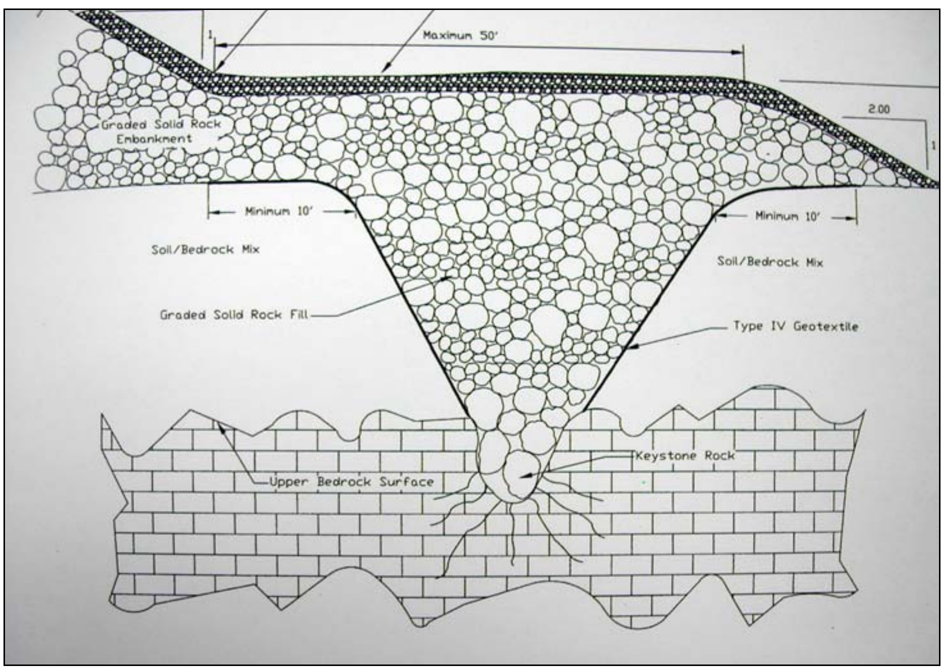

Figure 4. Treatment concept for filtering runoff into sinkhole adjacent to roadway embankment.

Figure 5. View of subject SR 66 project showing several sinkhole areas to be treated.

Figure 6. Placing the geofabric (geotextile) liner into the sinkhole (note man bending over in upper center of photo).

Figure 7. Typical sinkhole with graded rock placed onto the geofabric.

Figure 8. Typical sinkhole on SR 66 project, before placing graded rock embankment .

Figure 9. Same sinkhole with graded rock fill in lower lift of embankment.

Figure 10. Same sinkhole with choker stone lift on top of larger graded rock material and common fill material placed on top of choker stone material. The choker stone aids in filtering runoff as it enters the sinkhole basin.

Figure 11. Typical sinkhole on SR 66 project, before treatment.

Figure 12. Same sinkhole with geofabric and graded rock fill material.

Figure 13. Same sinkhole with full treatment of graded rock embankment. Truck is on subgrade of roadway.

A HYBRID ROCK FALL PROTECTION SYSTEM ALONG THE CANADIAN PACIFIC RAILWAY, NEAR FIELD B.C.

By
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Canadian Pacific Railway

ABSTRACT

Mile 134.0 Laggan Subdivision of the Canadian Pacific Railway, located beneath the North slope of Mt. Stephen (el.10,336 ft.) in the Rocky Mountains near Field, British Columbia, has been the site of frequent rock fall and snow slide activity since the Railway was constructed in the mid 1880’s. The majority of rock falls are stratigraphically controlled, originating from sub-horizontally bedded, Paleozoic age units of quartzite, shales and dolomite from over 1000 feet above the railway grade. Rock falls as well as snow slides generally concentrate in well-defined chutes down the glacially oversteepened face.

Rock fall protection at Mile 134.0, located at the toe of an active chute, has consisted of watchmen to observe and warn of rock falls or snow slide activity, slide detector or signal warning fences, timber sheds, catchment excavation and lock block barrier walls. Early rock fall/snow slide history is not well documented. However, in 1994, a rock slide of approximately 3200 cubic yards occurred which initiated a CPR study to determine if existing rock fall defenses were adequate and what might be required to improve them. After a review of several options was conducted, including a tunnel option, it was decided to construct a “hybrid” rock fall protection system ” consisting of double sided concrete lock block wall with reinforced compacted back fill and a Geobrugg rock fall catchment net installed on the upper surface. It was designed to withstand a rock slide volume up to 10,000 cubic yards and also be high enough so that if completely full of debris, “rollers” would roll over the track above the height of a train.

The final result was a double sided wall 25 feet high, 25 feet wide and 270 feet long made of 483 lock blocks (each block measured 2.5’x2.5’x5.0’) weighing in total over 1,050 Tons. The upper catchment net was a 500 Kj designed system with 10WFx60 steel posts 16.4 ft. high over a length of 200ft. In addition, the CPR signal warning system or slide detector fence is attached onto the upper wall facing the track as an extra safety precaution.

INTRODUCTION

Rock falls and avalanches have been a consistent hazard to trains along Canadian Pacific Railway's main line near Field, British Columbia since construction of the line in the mid 1880's. The existing section of track which runs along the North slope of Mount Stephen (elev. 10,495 ft.) still follows the original alignment above the Kicking Horse River Valley, with a grade of about 2.1 percent at an elevation of about 4,300 ft. Various attempts have been made over the years to protect the track from these hazards. These have included posting watchmen during critical times of the year in order to warn oncoming trains of any rocks or avalanches ahead. Timber avalanche sheds were constructed in key locations at the base of known active slide paths or chutes. In the 1960's slide warning fences were installed which activate the nearest signal to alert oncoming trains when any of the wires in the circuit are broken. These are still in use today, often installed in conjunction with a concrete lock-block barrier wall. In the 1990's a series of snow avalanches, debris slides and large rock falls over a length of approximately 2 miles, prompted CPR to re-evaluate its defenses against natural hazards through this area. This paper will describe a Hybrid rock fall protection system developed and constructed at one location, Mile 134.0 of the Laggan Subdivision.

Figure 1: Location map

GEOLOGY

Mount Stephen, situated in the Western Rocky Mountains, is characterized by Paleozoic and older age, flat lying to gently folded sediments deposited more than 500 million years ago. The basal rock unit, the Gog Group, is predominantly light brown quartzose sandstones, with some red, green and grey shales, and rare thin conglomerates (1). These are capped by the Mt. Whyte Formation which consists of three members. The basal member comprises flaggy carbonates with lenticular beds of pebbly water carbonates and deeper water shales. The middle member is made up of green shales with thin sandstones and shallow water conglomerates. The upper member is interbedded carbonates and shales (2) . Thick carbonates of the Cathedral Formation are the highest rock unit in the section. Above Mile 134.0, the unit is nearly 2000 ft. thick and made up of massive, rusty weathering, light grey to white dolomite and dolomite breccia with conspicuous amounts of graphite, and local lead and zinc mineralization (3). It was within this unit that lead and zinc ore bodies were discovered at a height of 1000 to1200 ft. above the Kicking Horse River valley floor. Two mines, the Monarch and the Kicking Horse, located on opposite sides of the valley, operated from about 1884 until 1950. Mount Stephen is on a gentle dome structure with at least one northwest-trending anticline (4). Regional bedding dips on the northeast face are 10 to 15 degrees to the northeast. North trending regional faults define the approximate east and west sides of Mt. Stephen. Zones of closely spaced nearly vertical parallel joints and fractures are observed, particularly in the Cathedral formation, striking north to north 10 degrees west. The lower slopes of Mount Stephen are oversteepened by extensive glaciation through the Kicking Horse River Valley. Glaciers still exist on the northeast flank of Mount Stephen at elevations over 7,200 ft. Talus of varying size extends from below the lower exposed rock faces to the Kicking Horse River floodplain at elevation 4,100 ft.

Figure 2: Aerial view of northwest face of Mount Stephen with mileages indicated.

ROCK FALL HISTORY

For many years, the timber sheds protected the track from smaller rock falls and slides and avalanches. As a result, recorded rock falls through this area are very scarce prior to the 1990's. However, large rock fall and debris flow events have occurred based on photographic evidence that has been retained. In 1945, an estimated 6500 cubic yards of debris and blocks up to 12 ft. diameter fell at Mile 134 destroying a timber shed. Up until the late 1980's a watchman patrolled the tunnels and sheds before each passenger train. The watchman's hut was at approximately Mile 134.5.

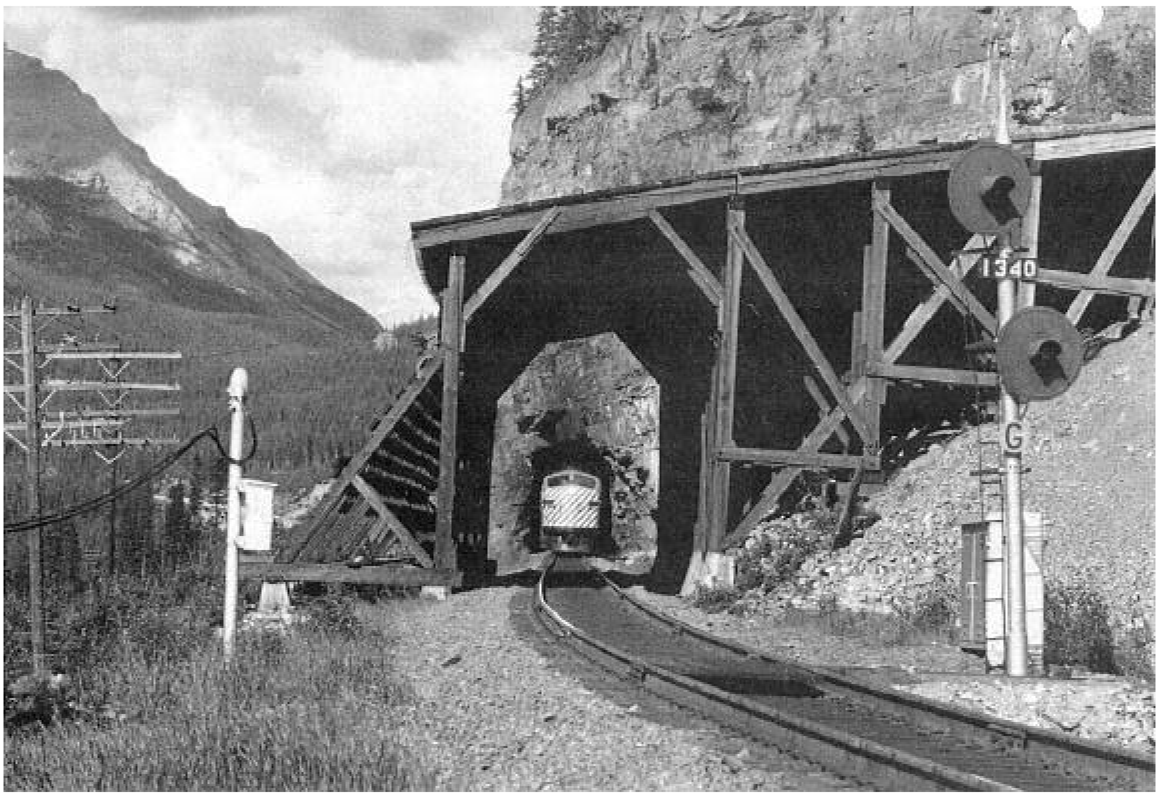

Figure 3: Looking west towards Mile 134 timber shed, circa. early 1970's

Table 1: All recorded rock falls Mile 133.9 to 134.1 Laggan Subdivision

Mile	Fall Date	Description	Fall Size (cu.yd.)	Delay (hrs)
133.90	26-May-94	3,200 cu.yd. rock slide overtopped lock-block wall and damaged 80 feet of track and 8 lock-blocks. Slide fence was triggered. Track was cleared in 26 hours using a D7 bulldozer and front end loader.	3,200	18
134.00	9-Apr-14	Train struck about 1.5 tons of rock. 9 cars were damaged		
134.00	4-Nov-67	Train moving at 20 mph contacted a rock slide.		0.58
134.00	24-May-95	2 Rocks removed by CPR ballast regulator.	0.06	1
134.05	21-Aug-95	Single rock. No track damage. Two trains delayed 45 minutes. Maintenance of way employees cleared the fall in 25 minutes.	0.75	1.15
134.05	17-Nov-95	3,200 cu.yd. slide of rock dirt & snow came down during construction of catchment. Slide fence was triggered. Only three 4 cu.yd. blocks came over wall and broke rail. 99 percent of slide retained behind wall	3,200	8
134.05	11-Sep-03	Rockfall landed between the rails and created a delay for 3 trains (2 hrs).	0.8	2

In 1994 and 1995, two large rock fall events occurred as shown on Table 1. These events prompted Canadian Pacific Railway to evaluate the effectiveness of the current rock fall protection and to evaluate various options for improvement. In addition, a rock fall hazard study was conducted by CPR in 1998 in order to evaluate potential stratigraphic source zones.

The May, 1994 rock fall originated from a height of 1000 ft. above the track level from the altered dolomite breccia zone of the Cathedral formation. This failure was associated with steep vertical joint intersections with bedding and with the weathering of the carbonaceous unit below which causes undermining of the blocks above. The estimated volume of 3,200 cubic yards was 95 percent retained by a 10 ft. high lock block wall.

The November, 1995 rock fall originated from a height of about 900 feet above track level, from the lower limestone and dolomite of the Cathedral formation. Failure of columnar blocks along nearly vertical joint intersections with bedding is one identified failure mechanism. One such column dubbed “the pinnacle” measures nearly 250 ft. high x 50 ft. wide x 20 ft. thick and has detached almost 20 ft. from the face at its upper end as shown in Figure 9. This column is situated directly above the Mile 134 catchment area.

Based on mapping and observations of numerous potentially unstable large columns and blocks, it was felt that future rock falls of magnitudes comparable to the 1994 and 1995 events were likely. Since rock scaling was not feasible due to the inaccessibility and large number of these unstable areas, it was decided that rock fall hazards could be most effectively mitigated at track level. Consequently, it was decided to evaluate several alternatives for rock fall protection at this location.

Figure 4: Rock fall May 26, 1994 – An estimated 3,200 cubic yards fell. Most was retained behind the existing 10 foot high lock-block wall. Cleanup took 26 hours.

Figure 5: Rock fall November 17, 1995 – An estimated 3,200 to 4,000 cubic yards fell behind the lock-block wall under construction.

ROCK FALL PROTECTION ALTERNATIVES

The following alternatives for rock fall protection were considered:

- Maintain the existing catchment areas with limited lock-block walls and slide detector fences.
- Enlarge catchment capacity by excavation and barrier wall construction.
- Construct an approximately 2,200 ft. long tunnel through the base of Mt. Stephen

The existing 10 to 12.5 ft. catchment areas, although adequate for smaller rock falls, would not be capable of retaining large rock fall events greater than 3,500 cubic yards. During the winter, avalanches often slide into these catchment zones, futher reducing the capacity available should a large rock fall or additional avalanches occur.
The tunnel option, although the most effective in rock fall and avalanche protection, would cost an estimated $15 Million to construct, which the Company could not commit to as part of its short term plan.
As a result, efforts were focused on designs for catchment enlargement and barrier walls, which would protect the track from both rock falls and avalanches. The costs of construction of which could be spread over several years as part of a multi-year plan.

CATCHMENT ENLARGEMENT

In September of 1994, after the rock fall event in May, surveying was carried out of the catchment area at Mile 134 where the event occurred and a cut template designed.
In October, 1994, excavation of approximately 6000 cubic yards of rock was carried out by drilling and blasting of the lower portion of the rock face to increase the available catchment capacity to nearly 8000 cubic yards. The existing 10 foot high lock block wall was left in place with the slide detector fence attached to the upper row of blocks for the next 5 years. No large rock fall event occurred during that period.

WALL DESIGN

In 2000, based on the1998 hazard assessment conducted by CPR, a conceptual design for a double sided lock block fortress wall, with compacted backfill and Geogrid with a 400 Kj Net system on the upper surface was considered as a viable protection system for Mile 134.
The key considerations for the design were as follows:

- The height of the wall should be sufficient to protect the full height of a locomotive or train car
- The net system should be capable of protecting the train from any flying rock should the entire catchment area become filled
- The wall should be designed with a safety factor to withstand sliding with the catchment filled with rock.
- The capacity of the catchment should be in the order of 10,000 cu. yd.

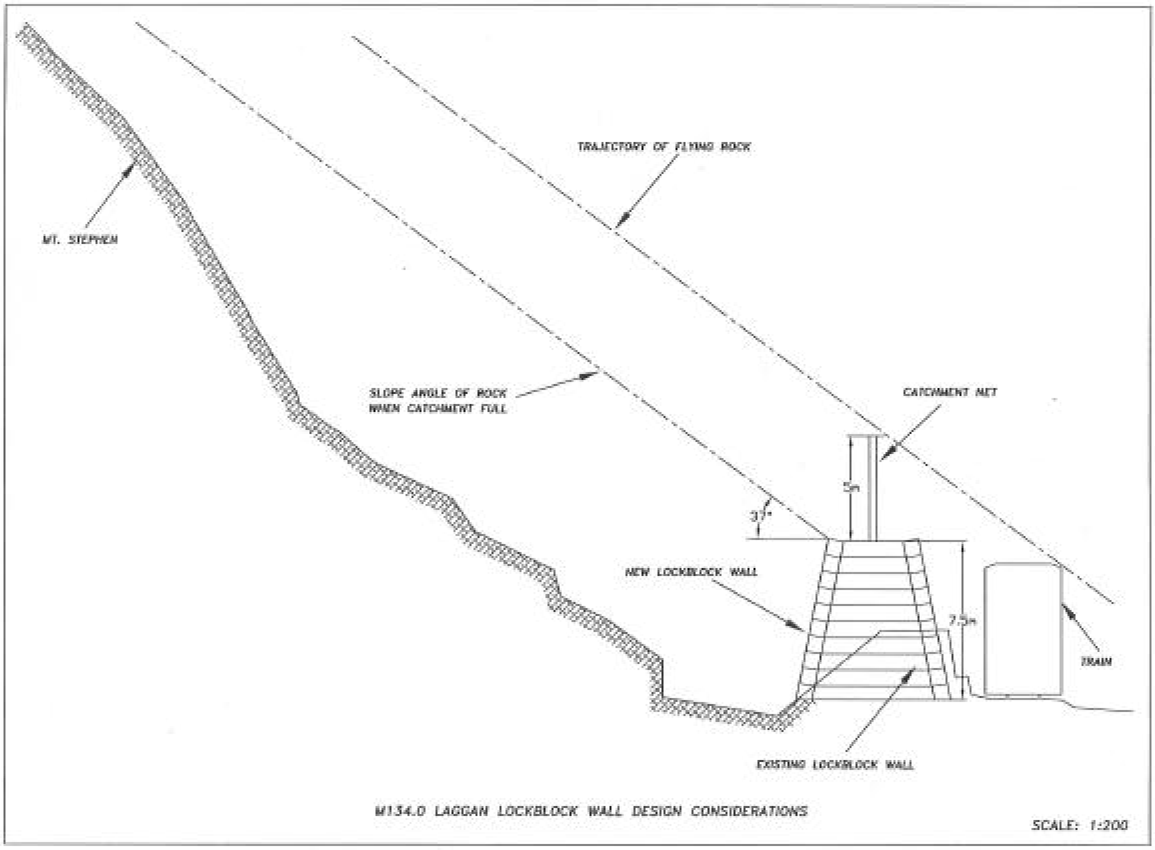

Figure 6 : Rock fall protection system design considerations

The resulting wall design was supplied by GeoPacific Consultants, Vancouver B.C. Its dimensions were 25 feet high by 25 feet wide at the base and 14 feet wide at the top over a length of 270 feet, with a 16.4 feet high net system on the upper surface over a length of 200 feet. The wall would be double sided and constructed of concrete lock-blocks, each measuring 5 feet by 2.5 feet by 2.5 feet. The wall would be required to retain rock fall debris at an angle of 37 degrees above the back of the wall. The front and back lock-block walls would be connected by geogrid (Paragrid 150/15) of design strength 59KN/m, with the strong axis extending from blocks of the south to the north wall on each row. Each course of lock-blocks would be backfilled with graded 6 inches and less native talus material to a height of 1.25 feet and compacted. Each wall is battered at 1H:10V. The factor of safety for sliding for this design is 1.86. The final net design was a 500 Kj net, 15 ft. high, supplied by Geobrugg in 25 foot long panels. Posts were galvanized steel WF10x60, 17 feet long.
The ends of the wall were sloped at 1.3V: 1V and stepped down in order to contain the back fill. The sloped ends also serve as ramps for access to the upper wall surface.

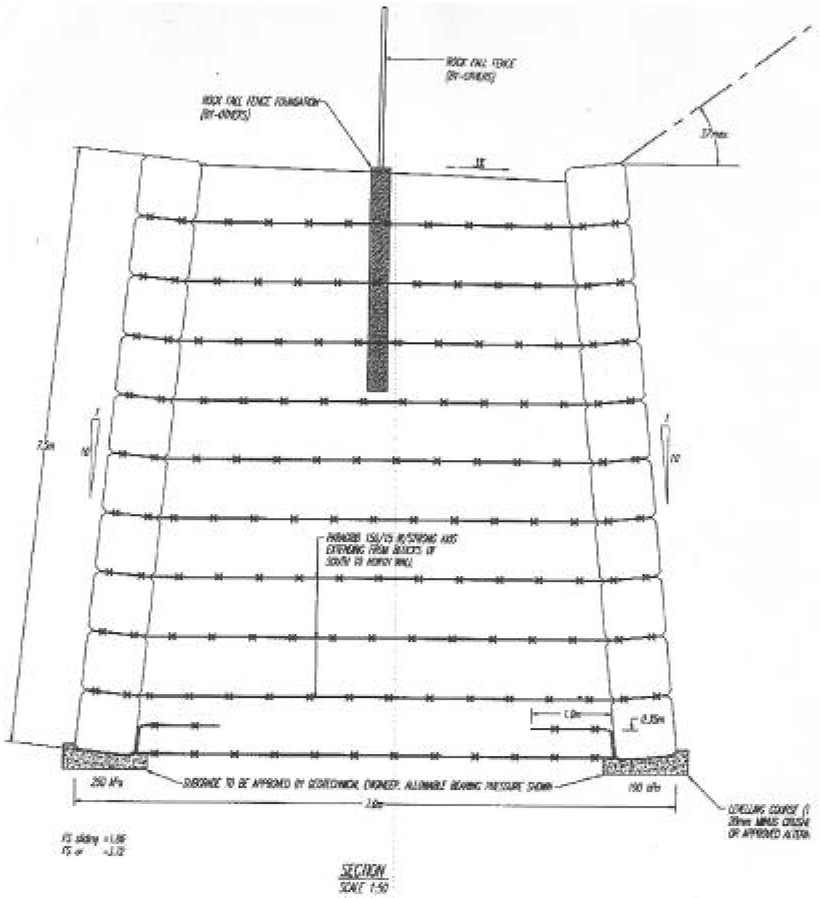

Figure 7: Wall Design – GeoPacific Consultants

WALL CONSTRUCTION

The lock-block wall in service today was constructed over a three year period, commencing in 2000. Work was carried out by CPR's own Bridge and Building crews in conjunction with local contractor Emil Anderson Construction of Hope. B.C. from September 20 to October 18, 2000. Initially, the existing lock-block wall and slide detector fence had to be removed. A temporary slide fence was constructed and re-attached each night during construction. It was noted that the existing wall had been constructed on the concrete foundation of an earlier timber shed. Due to the limitations of working with pre-cast lock blocks, this foundation could not be incorporated into the new wall construction. The base course for the track side wall was started immediately behind this foundation. Backfill material was obtained from the natural talus on site and graded using a "Grizzly" to remove oversize fragments. With the onset of snow and rain in the late fall, there was an increased hazard of rock falls. Consequently, due to the concern for safety of workers as well as budgetary restraints, construction of the wall was terminated with the wall 12.5 ft. high over a length of 235 feet at the base.

In 2001, work commenced on August 21. In order to install the fence posts required for the rock fall net, PVC pipe sleeves were placed in the fill a portion at a time to the height of each row of lock-blocks. Backfill material was compacted around them and holes cut in the Geogrid accordingly. By October 30, the wall was completed to its full height of 10 blocks or 25 feet and a length of 150 feet on the upper surface. The 17 ft. high WF10x60 posts used for this system were bolted to similarly sized WF bases, 10 feet long, which where concreted into the PVC pipes. The 500 Kj Geobrugg Net system was installed along the centre of the upper surface over the full 150 feet. The slide detector fence was strung across 10 ft. long, 4"x4" wooden posts, which were attached to the upper two rows of lock blocks facing the track.
In 2002, it was decided that the length of the wall needed to be increased to the west in order to fully utilize the available catchment area and maximize the rock fall protection zone. Consequently, the wall was lengthened 35 feet at the base in order to accommodate an additional 50 feet of Geobrugg catchment net at the top.

Construction costs for the total project are shown on Table 2.

Figure 8: October 21, 2002 – Western extension of the wall nearly completed

TABLE 2: TOTAL LOCK BLOCK WALL CONSTRUCTION COSTS(CDN$)

Labour	2,400 Manhours	$ 150,000
Equipment	1,400 Hours	$ 220,000
Geobrugg Net System	5,000 Square Feet	$ 43,000
Steel Posts	217 Linear Feet	$ 18,000
Concrete Lock Blocks	483	$ 49,650
Geogrid	20,000 Square Feet	$ 41,500
	TOTAL	$ 522,150

CONCLUSIONS

The term "hybrid", defined in the Oxford Dictionary as "a thing composed of incongruous elements", could be applied to the rock fall protection system described above. The "elements" are the double walled lock-block structure, the Geobrugg rock fall catchment net and CPR's slide detector fence system. All three of these elements will hopefully work congruently towards protecting the railway from rock fall hazards of magnitudes up to 10,000 cu. yd. As yet, the system has only been tested by snow avalanches and small rock falls that have previously bounced over the track (or sheds). However, the hazards still remain, as evidenced by " the Pinnacle" which , if it does fall, will prove to be the ultimate test.

Figure 9 : Vertical fractures in the Cathedral limestone have resulted in the development of features like “the Pinnacle” with estimated volume of 7,000 cu.yd.

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3D Interpretations of Subsurface Soil and Rock Conditions: Spaulding Turnpike, Rochester, New Hampshire

By: Marc Fish[1]

Abstract

The New Hampshire Department of Transportation initiated a large scale subsurface investigation covering several miles of ground surface for the Spaulding Turnpike expansion project in Rochester, New Hampshire. The investigation included several hundred test borings and test pits. These explorations helped to determine the depth to bedrock and the different soil layers along the new highway alignment. The test borings and test pits identified multiple soil and bedrock layers of varying depths and thickness within the limits of the project. Engineering Geologist's need to identify how these layers interact with one another and how they relate back to the new roadway alignment. Data from a test boring database has been imported into a software program, which allows for three dimensional subsurface interpretations to be made over the entire length of the project or over certain localized areas. Through kriging statistics, the test boring and test pit data are displayed as complete three dimensional block diagrams. Slices are taken through any part of the three dimensional diagram and separate soil or bedrock layers are extruded from one another allowing for subsurface interpretations to be made. Digital orthoquads (DOQ) and new roadway alignments are draped over the three dimensional block diagrams and bedrock contour lines are exported into a CAD/D program where bedrock lines can be drawn onto the project cross sections.

Introduction

New structures have been proposed and new roadway alignments have been drawn for the Spaulding Turnpike expansion project. To develop appropriate road and bridge foundations, subsurface information is required as part of the design process. The following approach was used to pull together and visualize all the spatially located subsurface information that was collected as part of this project. The data is composed of test boring and test pit information that is pulled together and exported from a test boring database in the form of a specialized text file. Through a unique software extension to a geographical information system (GIS), three dimensional models of the subsurface were developed depicting the soil and bedrock depths throughout the project area. Once the models were refined, bedrock surfaces were exported as DXF files into a CAD/D system where the bedrock lines were drawn on the project cross sections at specified intervals along the roadway alignment.

Procedure for 3D Model Development

The procedure for developing the three dimensional models was designed to use spatially referenced geotechnical data that had already been collected and entered into a computer database by the Earth Scientist staff of the Bureau of Materials and Research. A specialized text file was exported from the computer database and slightly modified on

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an “as needed basis” and then entered into a GIS running an “off the shelf” three dimensional modeling extension. Because inconsistent geologic layer names were occasionally encountered when multiple Earth Scientists worked on this project, data modifications were limited to the renaming of geologic layers.

The data was obtained through the drilling of test borings and the digging of test pits and the information that was collected was stored in a computer database on the Bureau’s server. The data included geographical coordinates, elevations, depths to the different soil layers and bedrock, and blow counts. An “off the shelf” three dimensional software extension running within a GIS framework conducted a statistical analysis on the data and visually displayed the data as three dimensional fence and block diagrams. The program was capable of visually displaying the diagrams confidence by adjusting the parameters of the statistical analysis.

The type of statistical analysis conducted by the program is a kriging analysis. This type of analysis is a weighted moving average interpolation that minimizes the estimated variance of a predicted point with the weighted average of its neighbors. The weighting factors and the variance are calculated using a semivariogram model that describes the differences versus the distance for pairs of samples in the input dataset (2). The three dimensional software visually displays where the highest and least confident interpretations are located. It does this by taking the log10 of the confidence bound value and then compares it to the log depth values and a corresponding standard deviation calculated for every node in the domain. The confidence bound value can be changed to any value so the confidence displayed by the software will be within a factor of the log10 of the confidence bound value of the actual depth (2).

Data is extracted from the test boring database using minimal structured query language (SQL) statements. A text file is developed that contains the same number of entries for each boring location, so every geologic layer in the model is represented in each boring. For borings where geologic layers are absent or where borings have not extended deep enough to encounter layers that are known to be present, flags are used to allow the automated processing of the data. To help determine the locations where the model has its greatest and least confidence additional kriging parameters will be utilized when the software conducts its statistical analyses.

Digital orthoquads and the CAD/D drawings of the new roadway alignment are incorporated into the three dimensional view to help visualize the subsurface conditions directly beneath the existing ground surface and the proposed roadway alignment. Project cross sections, drawn at certain intervals along the proposed roadway centerline are developed through a CAD/D system. These cross sections contain the bedrock lines developed from the bedrock surface elevations by the three dimensional software extension.

3D Modeling Results

To develop the three dimensional model, data was extracted from several test boring databases using SQL statements and was placed within a specially formatted text file (figure 1). The data was brought into a GIS and displayed two-dimensionally with the digital orthoquads and “Routes” layer (figure 2). The three dimensional modeling extension was initiated through a drop down menu and the text file that was extracted from the test boring database was loaded into the GIS.

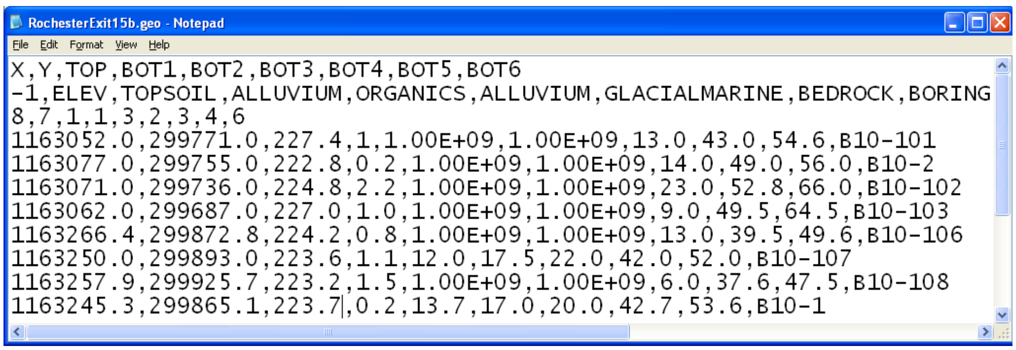

Figure 1: Geology file format, exported directly from the test boring database. 1.00E+09 is a flag value that tells the program there is a missing layer in the boring.

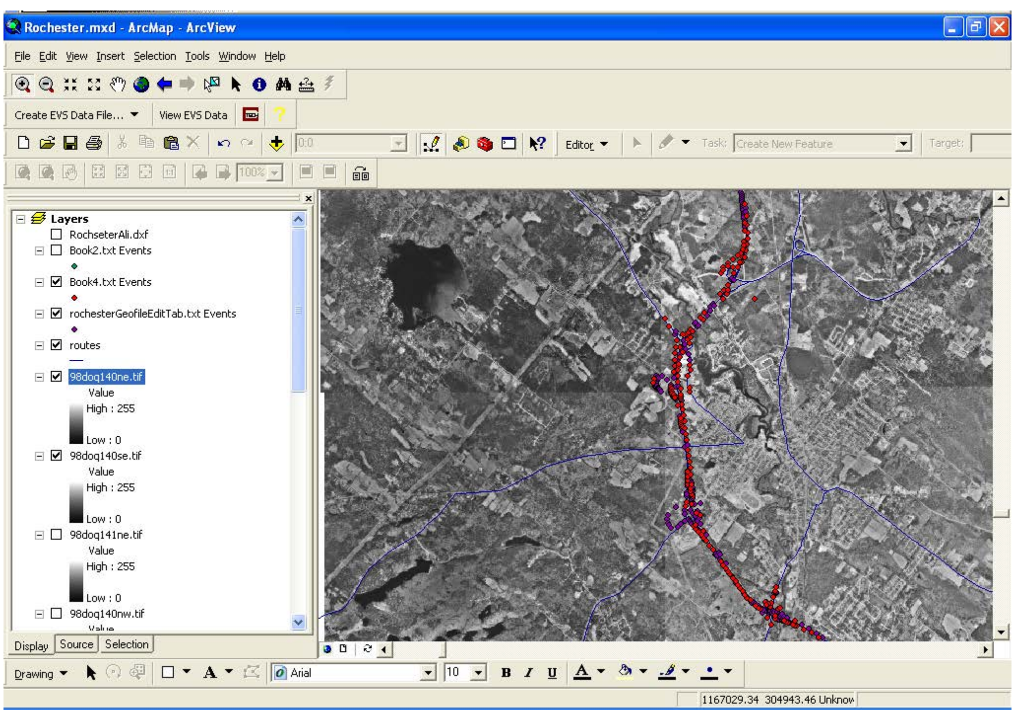

Figure 2: GIS containing the “Routes” layer, the test borings & pits, and digital orthoquads.

By using different combinations of modules within the three dimensional software extension, three dimensional diagrams were made that display the test borings, the subsurface soil and bedrock diagrams, the project plans and digital orthoquads. Figure 3 is an example of a three dimensional diagram showing the test borings, the soil and bedrock layers, the project plans and a digital orthoquad for the entire Spaulding Turnpike project. The scale in figure 3 makes it difficult to observe the subsurface conditions directly beneath the roadway centerline at any specific location within the project. To look at one specific area within the project the model is "zoomed-in" using a simple "mouse" control. Figure 4 shows a three dimensional block diagram zoomed to a portion of the project in the vicinity of the exit 12 interchange. Just south of exit 12, a new larger bridge is being proposed to replace the existing bridge over the Cocheco River. To recommend a foundation for the new bridge, three phases of drilling were conducted to determine the specific subsurface conditions around this area. The view displayed in figure 4 is derived from a text file that was limited to the exploration locations only in the vicinity of the existing bridge over the Cocheco River. Some of the software's modules were changed so the road and bridge alignments and the digital orthoquads would not display. Figures 4 & 5 show the subsurface conditions after the first phase of drilling. The view in figure 4 looks directly east and is perpendicular to the bridge alignment and parallel to the Cocheco River. A steeply dipping bedrock surface covered by a thin layer of glacial till and a thick layer of glacial marine silts and clays can be observed. The top surface of the block diagram displays recent alluvial deposits, a man-made fill, and topsoil. The view in figure 5 is from the same direction as in figure 4, but the bedrock and glacial marine deposits are the only layers displayed and the glacial marine deposit is slightly transparent.

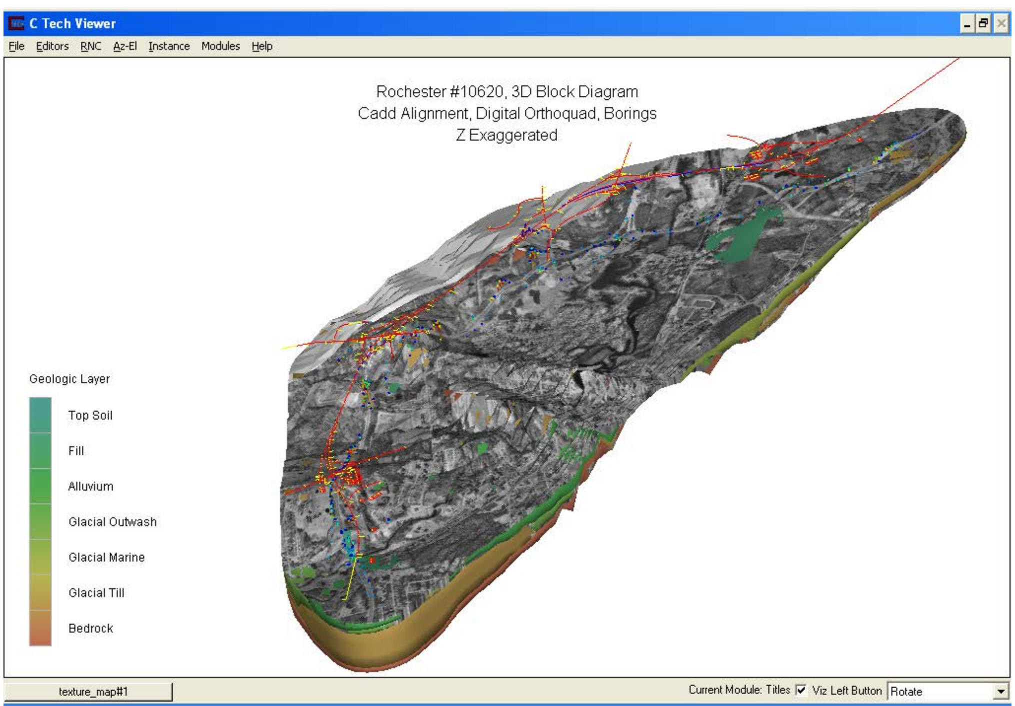

Figure 3: 3D block diagram of the subsurface conditions with 3D borings, digital orthoquad and CAD/D drawing draped onto the surface.

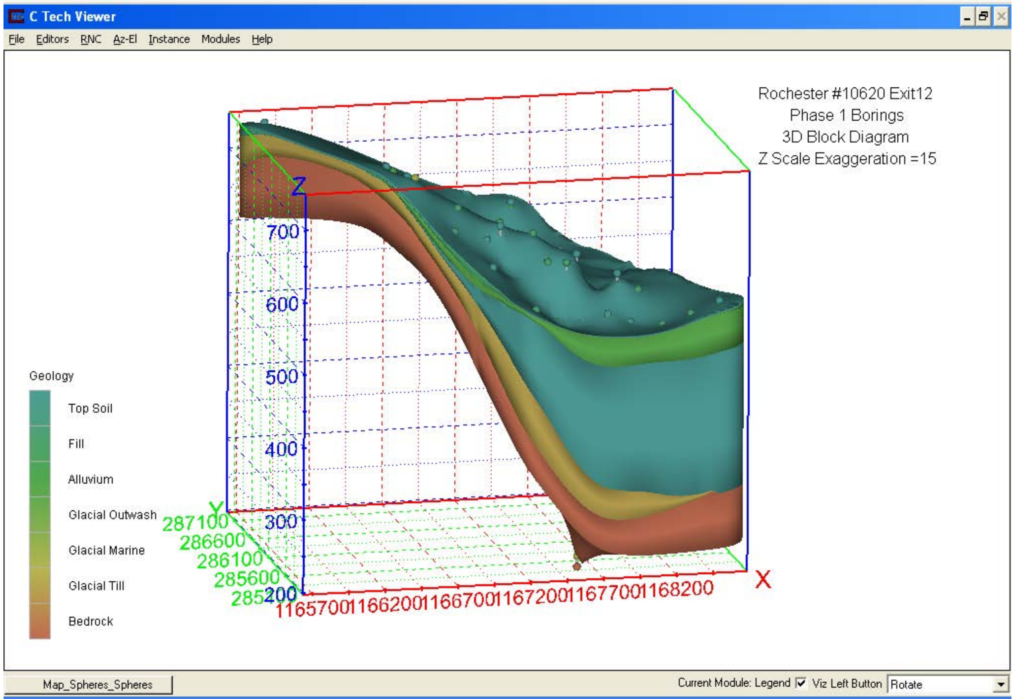

Figure 4: 3D block diagram of subsurface conditions at the Cocheco River Bridge after the first phase of drilling, looking east parallel to the river. Z scale exaggeration = 15.

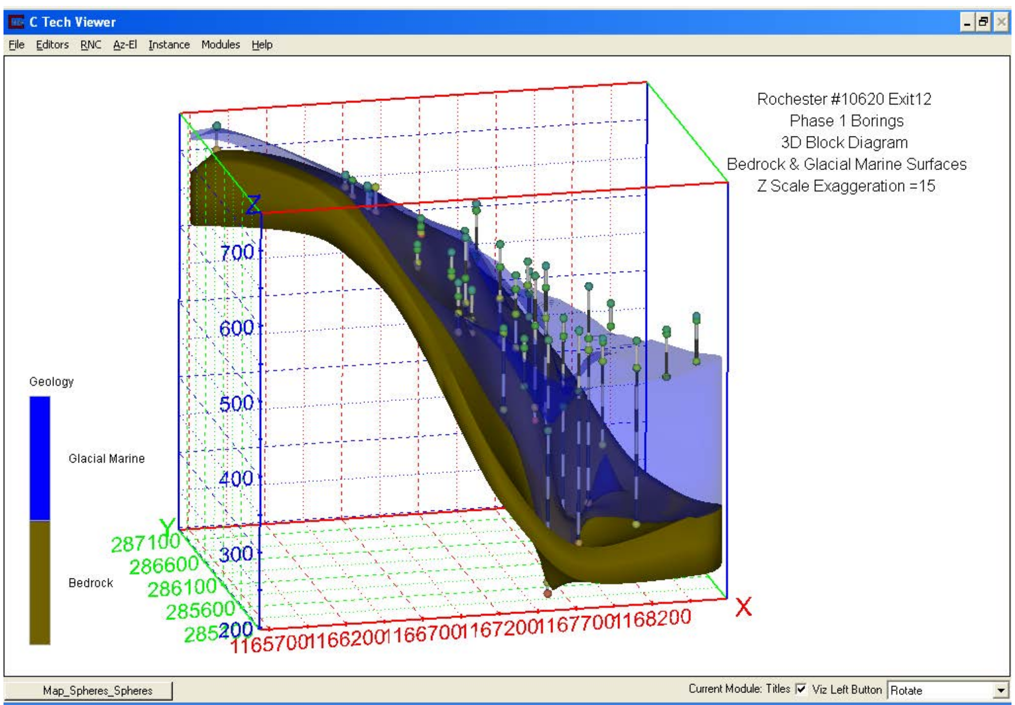

Figure 5: 3D block diagram of subsurface conditions at the Cocheco River Bridge after the first phase of drilling, looking east parallel to the river. This view shows only the test borings, the bedrock surface and the glacial marine deposit. Z scale exaggeration = 15.

Marc Fish, NHDOT

To obtain additional soils information and to better delineate the bedrock surface two additional phases of drilling were completed near the existing bridge over the Cocheco River. Figures 6 & 7 are three dimensional block diagrams that were created after the results from the second and third phases of drilling were added into the text file. The three dimensional model becomes more refined as additional information is added into the program. To observe any side of the block diagram a simple "mouse" driven procedure is used to rotate or zoom into or out of the view. Layers can be extruded from one another or turned on or off to help reveal where the soil layers are thickening or where they are pinching out.

It is also possible to construct a three dimensional block diagram based upon the soil densities that are derived from the number of blow counts it takes to drive a two foot, split spoon soil sampler, twelve inches with a one-hundred and forty pound weight. This is accomplished by extracting the sample depths and blow counts, in addition to the other data, from the test boring database. Through a slightly different text file format, a three dimensional block diagram is constructed to display where all the soft and hard soils are located in the vicinity of the existing bridge over the Cocheco River (figures 8 & 9). This three dimensional block diagram can be used to help determine the type and depth of a new bridge foundation based upon the soil densities collected during the subsurface investigation.

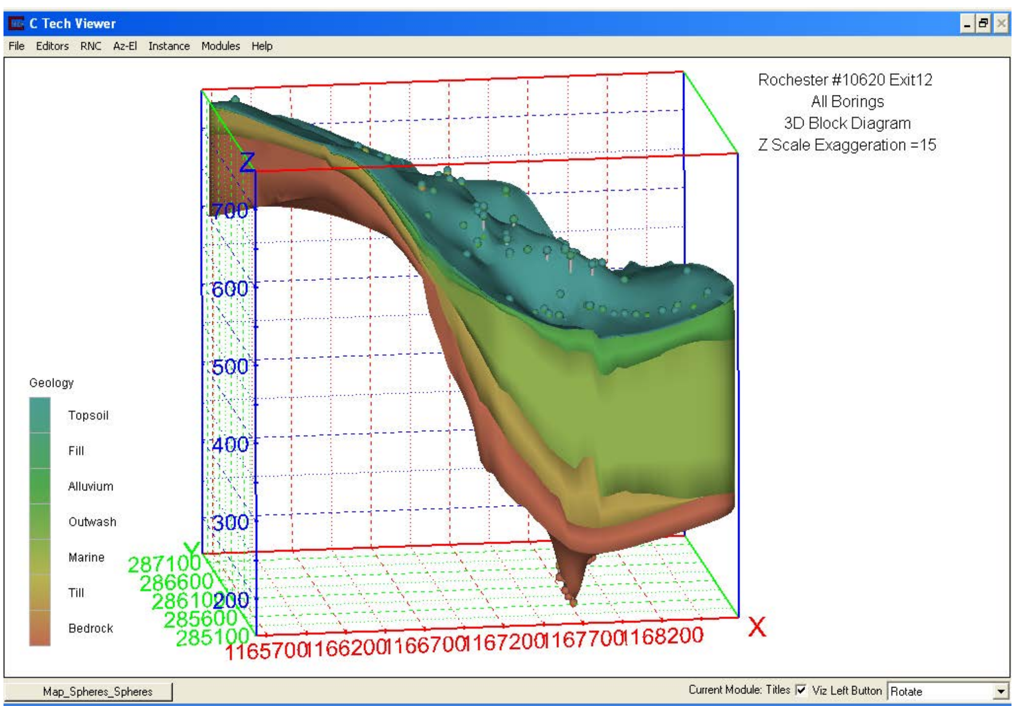

Figure 6: 3D block diagram of the subsurface conditions looking east over the Cocheco River after all phases of drilling. Z scale exaggeration = 15.

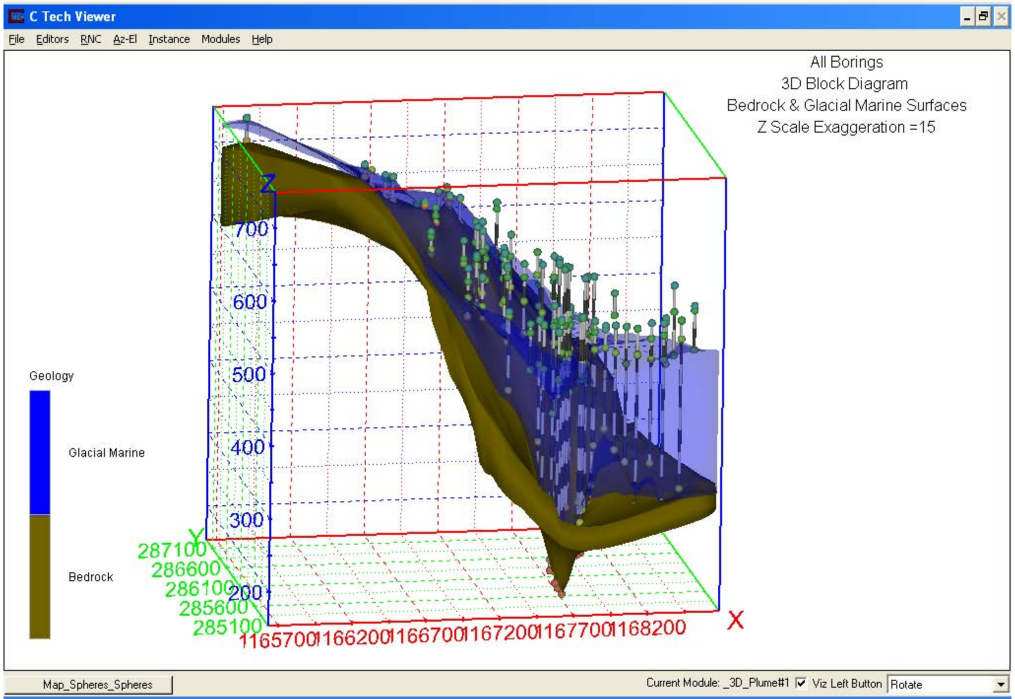

Figure 7: 3D block diagram of subsurface conditions at the Cocheco River Bridge after all phases of drilling, looking east parallel to the river. This view shows only the test borings, the bedrock surface and the glacial marine deposit. Z scale exaggeration = 15.

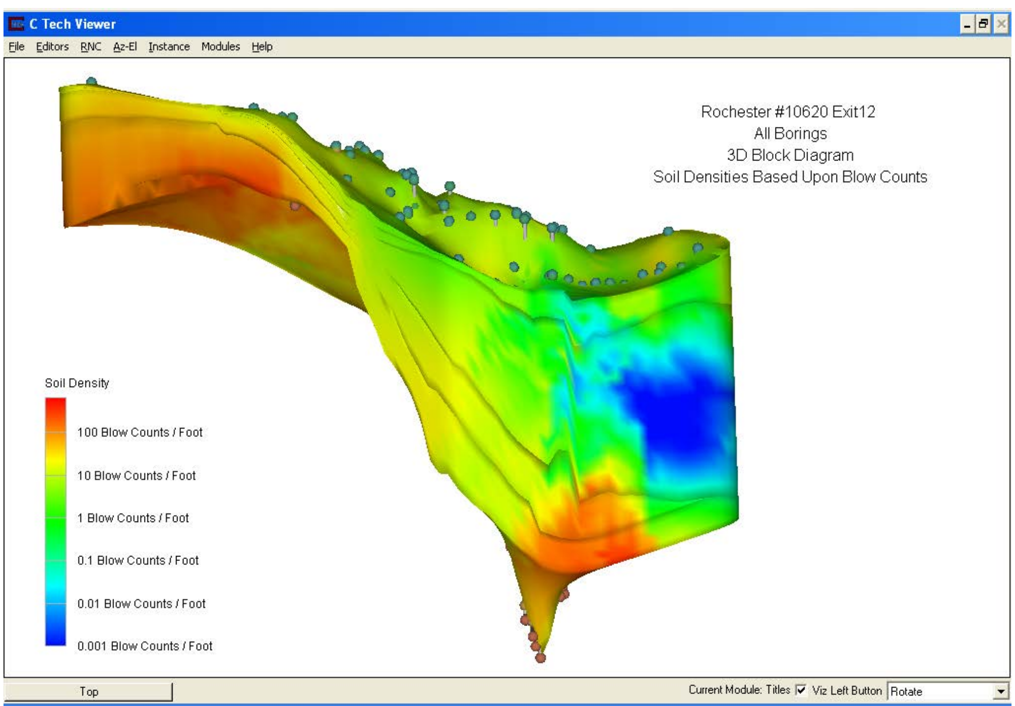

Figure 8: 3D block diagram looking east over the Cocheco River showing the locations of the soft and hard soils. Soil densities are based upon sample blow counts.

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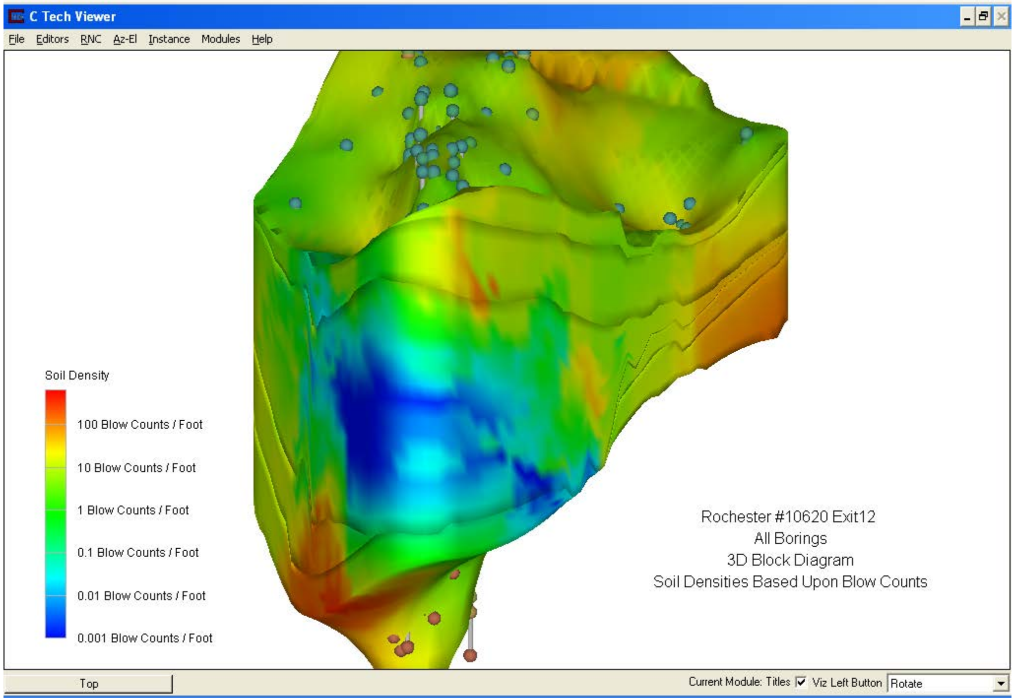

Figure 9: 3D block diagram looking north over the Cocheco River showing the locations of the soft and hard soils. Soil densities are based upon sample blow counts.

Through the addition of another module and by adjusting the kriging parameters a confidence interpretation of any surface within the model can be made. Figure 10 shows the confidence interpretation of the bedrock surface after all three phases of drilling. Figure 11 shows the confidence interpretation of the glacial marine deposit after all three phases of drilling. The confidence bound value was set to 25 for both of these diagrams. This value enables the program to display a decent representation of where the most confident data is located. As it would be expected, the most confident data surrounds areas where explorations were conducted. Figures 12 & 13 show the confidence interpretations displayed below a couple of fence diagrams. At this scale, pinch outs and depths to different geologic layers at specific locations along the alignment can be observed and compared to the displayed confidence interpretation below.

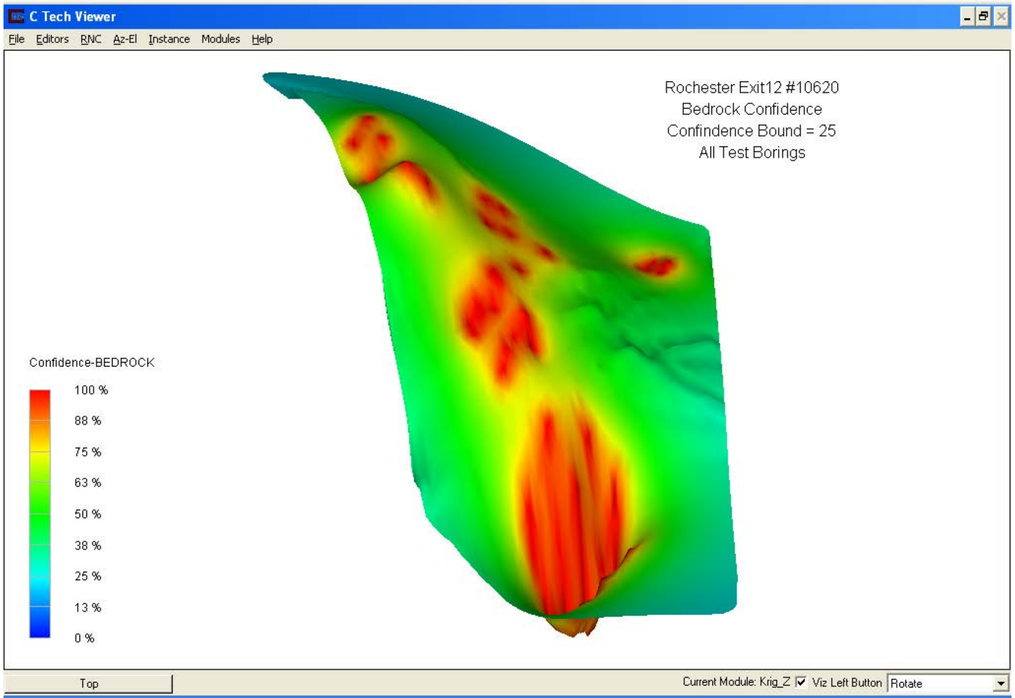

Figure 10: Confidence interpretation of the bedrock surface after all phases of drilling at the Cocheco River Bridge.

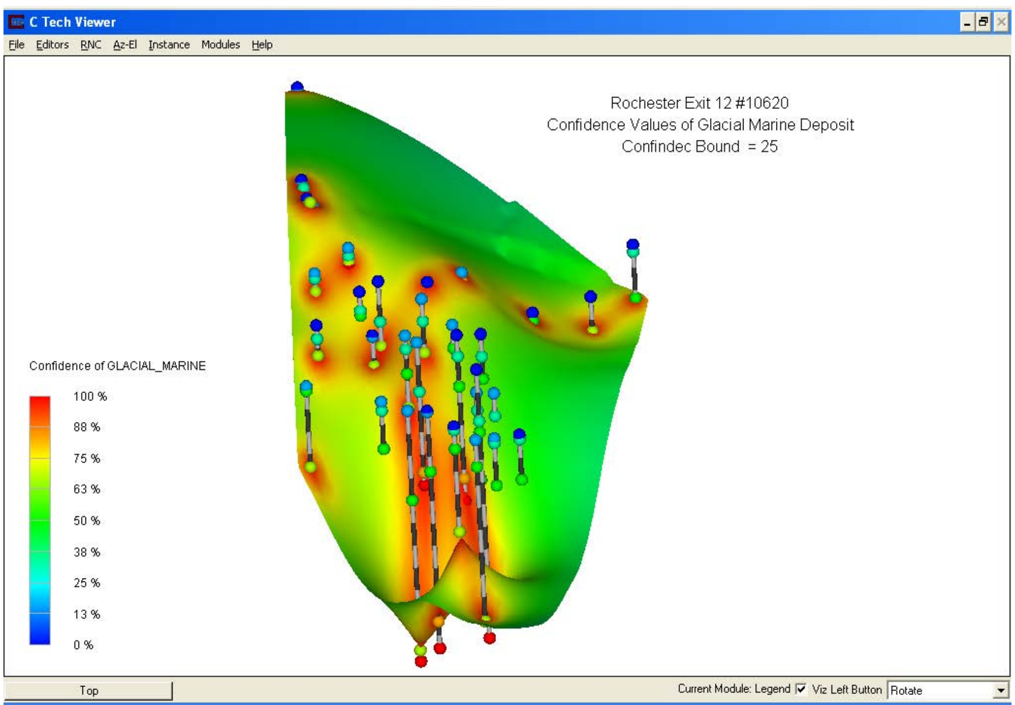

Figure 11: Confidence interpretation of the glacial marine deposit after all phases of drilling at the Cocheco River Bridge.

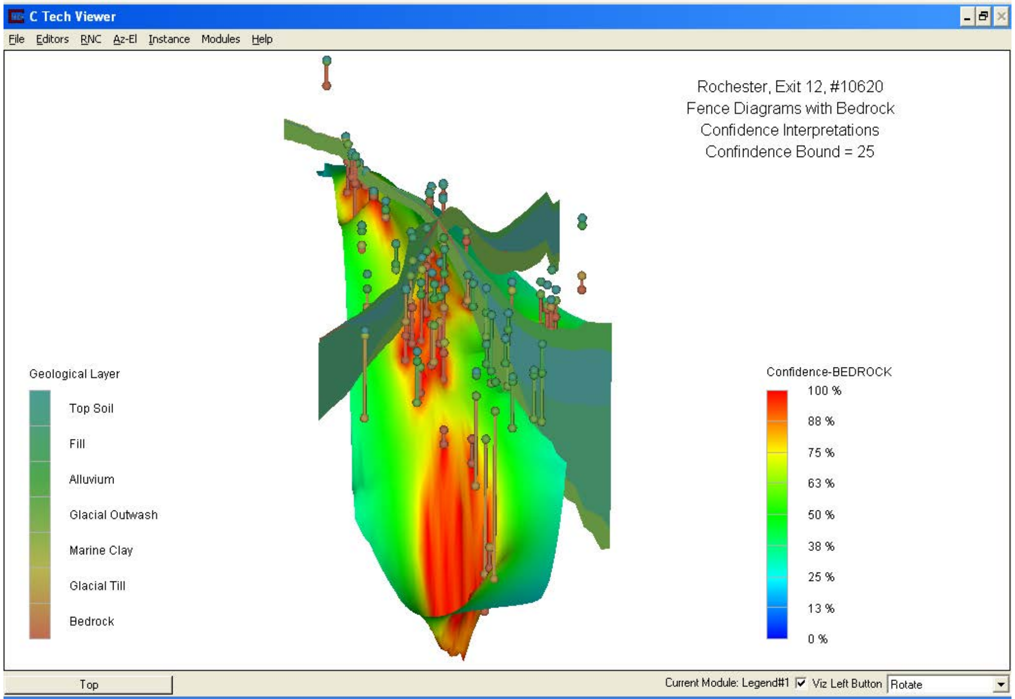

Figure 12: Confidence interpretation of the bedrock surface beneath the fence diagrams after all phases of drilling at the Cocheco River Bridge.

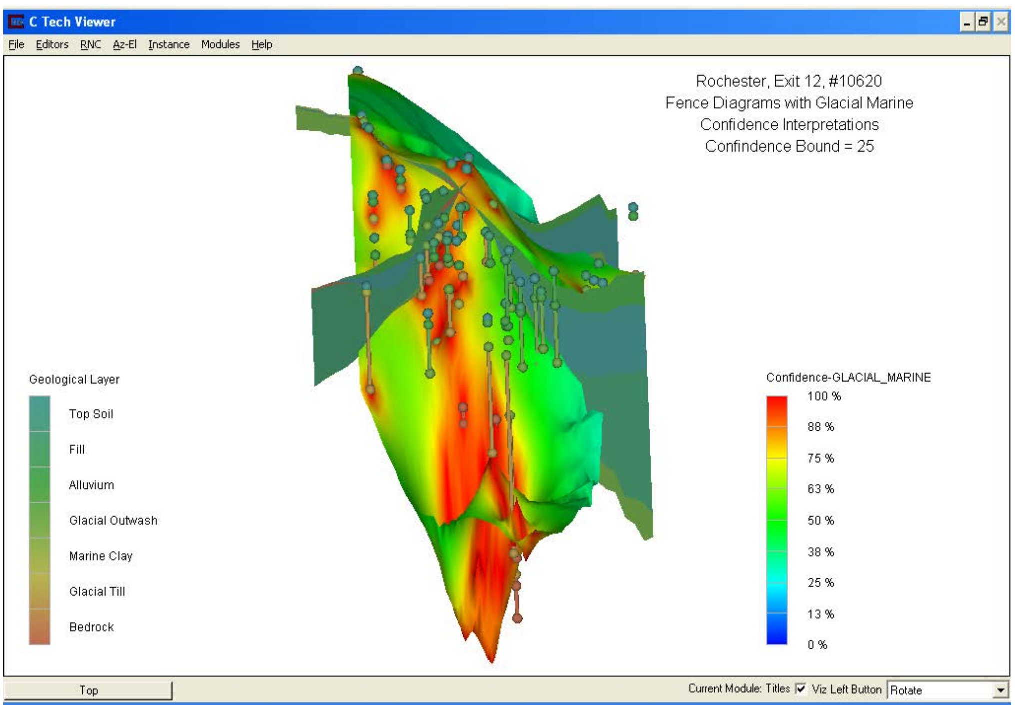

Figure 13: *Confidence interpretation of the glacial marine deposit beneath the fence diagrams after all phases of drilling at the Cocheco River Bridge.*

To better isolate the bedrock surface surrounding the area of the Cocheco River Bridge, a three dimensional block diagram is used with the above lying soil layers turned off and contour lines draped onto the surface (figure 14). The three dimensional software extension writes a CAD/D file containing the bedrock surface contour lines using the "Write DXF"(2) module. The DXF file is loaded into a CAD/D program and project cross-sections containing the bedrock lines are developed along the roadway centerline at specified locations and intervals (figure 15). In the same fashion, contour lines can be developed for any layer within the project and written as a DXF file and loaded into a CAD/D program for the development of project cross-sections. These contours can cover any specific location or the entire area of the project. Figure 16 is a diagram displaying the bedrock contour lines covering the entire area of the project. It should be noted, that contour lines covering the entire area of the project are only as accurate as the confidence interpretations indicate. Specific locations within the limits of the project that contain minimal subsurface information will have less confidant interpretations.

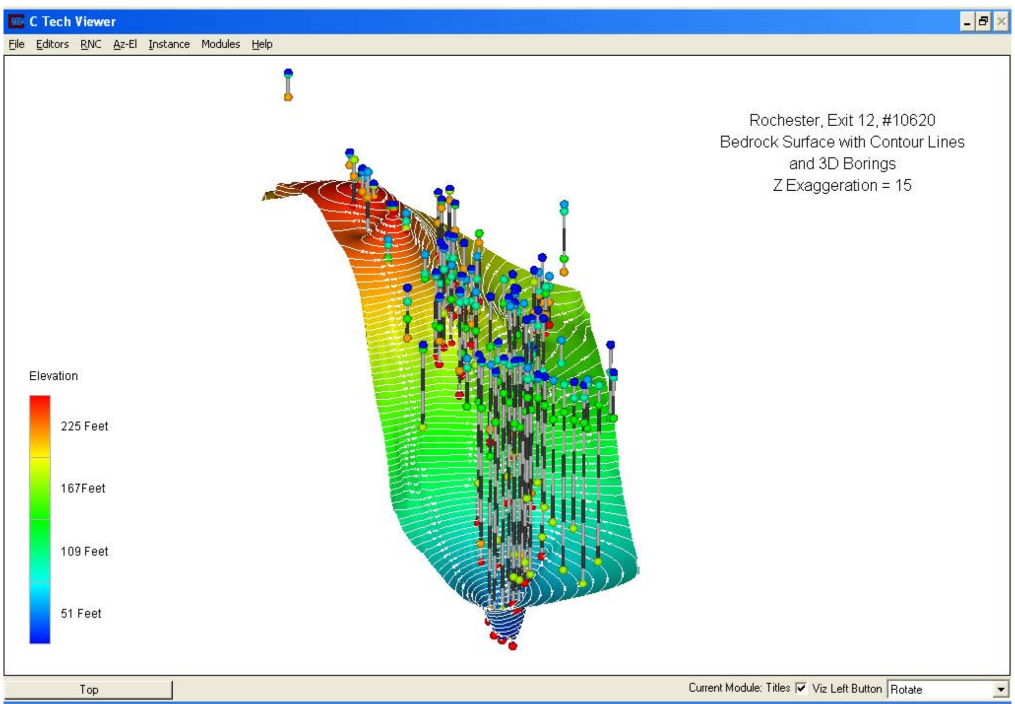

Figure 14: Bedrock surface with contour lines and 3D borings after all phases of drilling at the Cocheco River Bridge.

Marc Fish, NHDOT

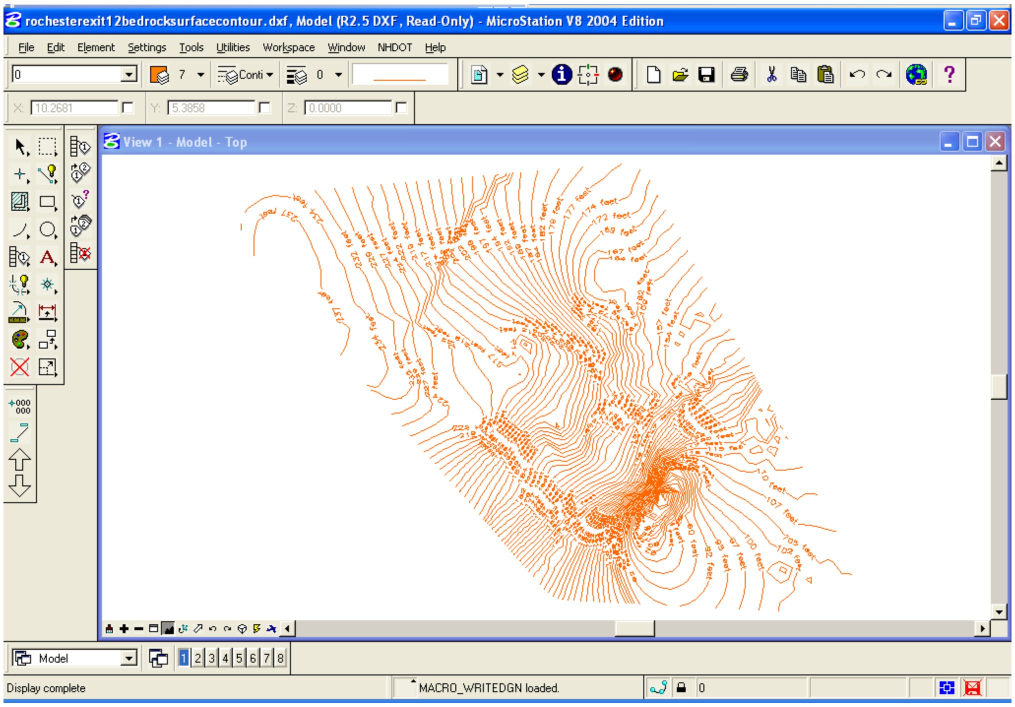

Figure 15: Bedrock contour lines, displayed two dimensionally, within a CAD/D program of the Cocheco River Bridge location.

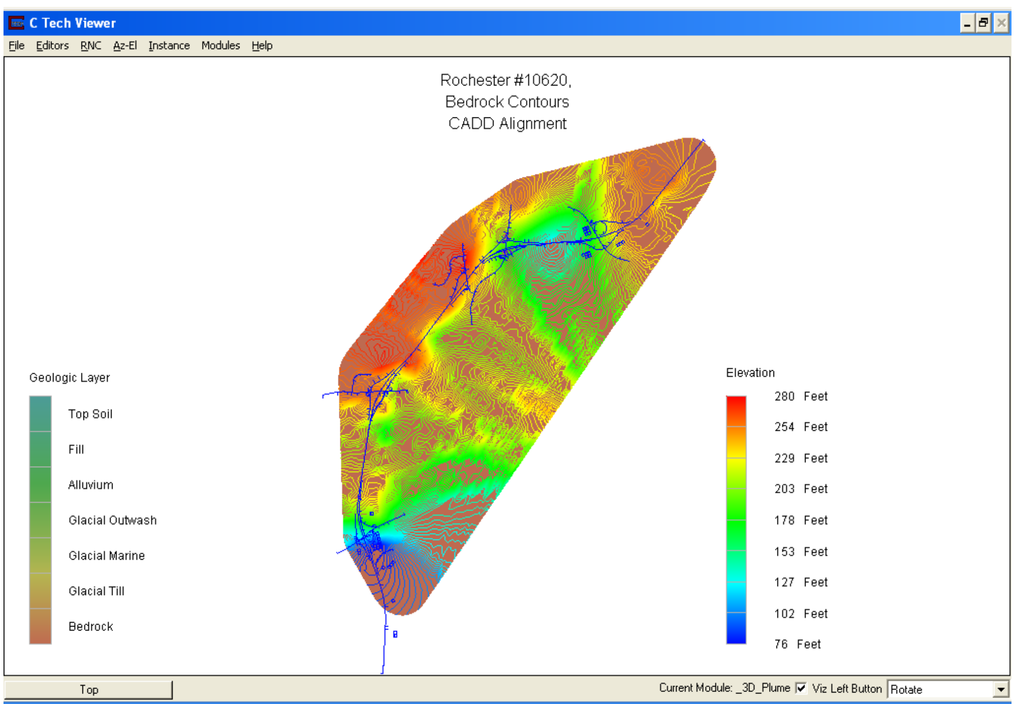

Figure 16: Color coded bedrock contour lines for the entire Spaulding Turnpike expansion project. The contours are draped over the bedrock surface.

Cocheco River Bridge Foundation Recommendations

A deep foundation that extends through the soft soils and derives its support from the deeper more competent soils is considered necessary for the new Cocheco River Bridge foundation. As observed in the three dimensional models, there is a steeply dipping bedrock surface that dips downward in a southerly direction at an angle of at least 55 degrees. A pile foundation driven to bedrock is not considered a viable option because the piles could slide along the steep surface making the foundation unstable $_{(7)}$. The existing bridge abutment and wingwall foundation rest on closely spaced timber piles and overlap the new bridge abutment locations. Drilling through the existing bridge foundation and wood piles would be technically difficult and could disturb the foundation's integrity and put the existing bridge structure at risk $_{(7)}$.

The recommendations for the new bridge foundation are to slightly shift the bridge alignments and construct a pressure injected footing foundation to a depth located within the glacial marine deposit. The depth of the foundation should extend to an intermediate depth below scour depth and the unsuitable upper soils. This type of foundation is considered necessary because the soft soils encountered during the subsurface investigation would not support a spread footing foundation and piles driven to bedrock could slide along the steeply dipping bedrock surface $_{(7)}$.

3D Modeling Discussion

Because explorations cannot be conducted everywhere within the limits of a project, subsurface interpretations must be made using limited information. To develop a reasonably accurate three dimensional model, an exploration plan should include a minimum number of explorations. As the number of explorations are increased the confidence in the three dimensional model is also increased. If the precise depth to a subsurface layer is needed at a specific location then this location should be explored and nearby explorations can be added until the model is confidently predicting the depths to this layer. When it is possible, nearby exploration locations should be placed close to where the specific information is needed. In other words, explorations placed where bridge foundations will not be located only help to define the geologic layer over the entire area of the project and not over the specific area of the bridge foundation. The level of confidence established for a three dimensional model should by based upon specific project information and the project engineer's level of experience.

A geological hierarchy for the project must be developed and the test boring data collected in the field must conform to this hierarchy. Accurate interpretations must be made about which geological unit a soil sample belongs to. Poor geological interpretations lead to inaccurate or unrealistic three dimensional models. If an extremely precise three dimensional model is desired then data that is in close proximity to one another must be used. Simply speaking, the greater the quality and quantity of the data that the model uses, the better the results will be. To locate additional exploration locations the module that develops confidence interpretations can be used to identify new drilling locations, which will increase the model's confidence. Additional exploration locations can also be used to demonstrate how well the software is interpreting the subsurface conditions by determining the actual depth to the location of an interpreted layer. As a final note, the development of a GIS and three dimensional models using this software extension requires time and significant computing power.

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DETERMINING SOIL AND ROCK STIFFNESS WITH MASW
Investigations of the 2004 I-40 Landslide and other Projects

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ABSTRACT

The Multi-channel Analysis of Surface Waves (MASW) method is gaining popularity as a value-added, non-intrusive geophysical tool to aid in subsurface investigations. Using readily available seismic acquisition equipment and an appropriate energy source, surface wave data can be collected in areas with high traffic counts, in locations with overhead and buried utilities, and on a variety of surfaces, including reinforced concrete and asphalt. The data can be processed to obtain 2D cross-sections of subsurface shear wave velocity – a direct indication of the stiffness of the soil and rock. Typical applications including mapping the depth to rock, locating weak zones in soil and rock, and providing average shear wave velocities for IBC site class designation. Several example MASW investigations are presented, including studies performed on the section of I-40 in Haywood County, NC that failed from erosion by floodwaters in September, 2004.

INTRODUCTION

The selection of appropriate geophysical methods to use for a particular subsurface investigation depends on a variety of factors, including subsurface soil and rock conditions, surface conditions, background noise, sources and levels, survey objectives, cost, and schedule. For example, there is no single method appropriate in all situations for determining approximate depth to rock; conditions at one site may require the use of seismic refraction while 2D resistivity may be more applicable at another location. Each geophysical method has limitations and pitfalls that need to be considered when planning a geophysical program. Of course, there are always some situations where geophysics is not cost-effective or will not provide the required resolution.

The relatively recent development of Multi-channel Analysis of Surface Waves (MASW) provides a robust method that fills a gap in geophysical methodology. MASW can be used to develop 2D images of subsurface shear wave velocity – a direct indication of the relative stiffness of subsurface materials. MASW can be performed in urban areas where traffic noise would prohibit the use of conventional seismic refraction and where buried utilities would interfere with resistivity data collection. This makes MASW very useful for surveys along roadways.

As with any geophysical technique, there are limitations to the MASW method. For example, we have found MASW to work best in areas with a smooth, compacted ground surface and with relatively straight survey lines. Areas with rough topography make it difficult to pull the geophone array along the ground surface while loose soil conditions can attenuate the surface wave energy and reduce the quality of the results. Another important factor is selection of an appropriate energy source.

BACKGROUND

Surface Waves and Shear Wave Velocity

Surface waves are ground-coupled seismic energy, often referred to as ground roll, and are the most damaging seismic energy in earthquakes. The predominant surface wave component is the Raleigh wave, which travels in a retrograde elliptical ground motion, with a velocity approximately 92 percent of the shear wave velocity. The other property that makes surface waves useful is their dispersive nature (Figure 1). Surface waves are dispersive in that the phase velocity of surface waves varies with frequency (wavelength). Higher frequency (shorter wavelength) components travel through the near surface at a velocity close to the shear wave velocity of that layer. Lower frequency (longer wavelength) components travel through a thicker section of the surface and are more affected by the deeper shear wave velocity. This dispersive nature allows variations in the subsurface shear wave velocity to be recognized and modeled. Surface waves are also easy to generate, making up about 70 percent of seismic energy generated by an impact source.

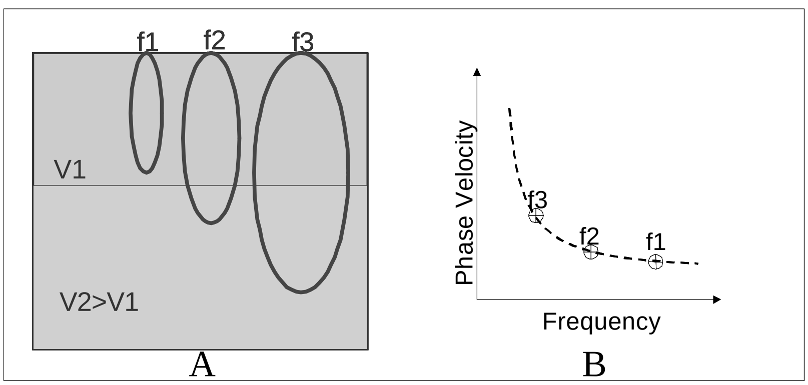

Figure 1 – Variation the wavelength of the Raleigh wave with frequency (A) leading to dispersion, or the change in velocity with frequency (B).

Spectral Analysis of Surface Waves (SASW)

Surface wave data have been used for over 20 years to determine subsurface shear wave velocities for various applications. The Spectral Analysis of Surface Waves (SASW) method was developed in the early 1980’s and was initially used for pavement evaluations (Nazarian, Stokoe, and Hudson, 1983, e.g.). Since then, the SASW has been adapted for widespread use in geotechnical investigations, including depth to rock, condition of concrete structures, and characterization of waste disposal sites (Haegeman and Van Impe, 1999, e.g.). The SASW method utilizes two receivers (accelerometers or geophones) spaced evenly about or on either side of an impact source. While only one receiver spacing may be needed for very shallow investigations, the receivers typically have to be moved further apart a number of times to develop a shear wave velocity profile with depth.

Multi-channel Analysis of Surface Waves (MASW)

In the past decade, the MASW method has been developed to take advantage of the multi-channel seismic approach. While surface wave energy has long been considered a nuisance in seismic surveys, the advent of MASW has turned this “noise” into data. The MASW method has largely been developed through research and development by the Kansas Geological Survey (Park, Miller, and Xia, 1999, e.g.). The results of that work include powerful algorithms to recognize, display, and invert surface wave energy. In addition to the active MASW method that is the subject of this paper, work has been done by others to develop the passive MASW method, where ambient noise (ground roll from vehicles, etc.) is collected and processed (Pullammanappallil, Honjas, and Louie, 2003).

MASW data are typically collected using a linear array of at least 24 vertical geophones connected to a standard engineering seismograph. For shallow surveys of less than 50 feet, geophone frequencies of 8 or 10 Hz can be used. However, for deeper investigations, lower frequency geophones such as 4.5 Hz should be used so that the geophone roll-off frequency does not limit the wavelengths that are recorded. Depth of investigation is also a function of array length and source-receiver offsets. One rule of thumb is that the geophone array length should be twice the investigation depth. The energy source used is selected based on the desired frequency/wavelength range. Our experience has shown that while sledgehammers can be used for investigations less than 50 feet deep, heavier weight drop sources should be used to generate the lower frequencies needed for deeper investigations.

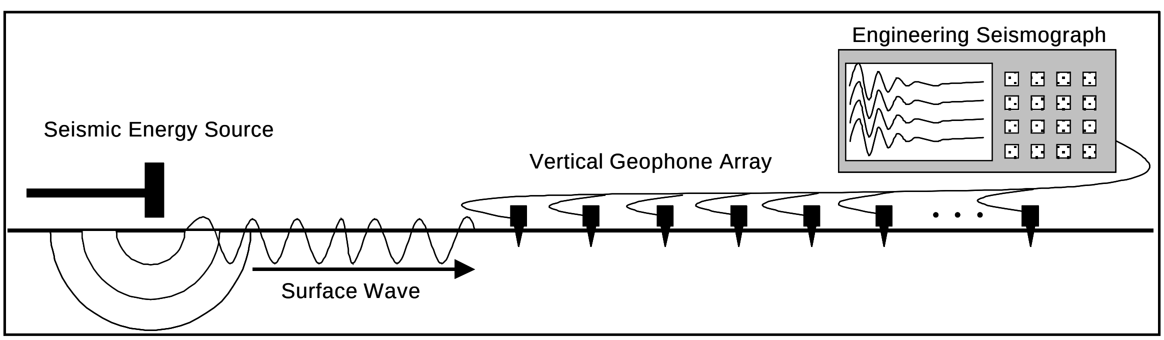

Figure 2 – MASW data acquisition. Critical factors include size of energy source, source-receiver offsets, geophone frequency, number of geophones, geophone spacing, and total array length.

The processing steps for MASW data include recognition of the surface wave energy, transformation into the frequency-domain, selection of points of maximum surface wave energy to form the dispersion curve, and inversion to create an earth model of subsurface shear wave velocity (Figure 3). Although the Raleigh phase velocity is a function of five factors - frequency, compressional (P-wave) velocity, shear (S-wave) velocity, density, and thickness of layers, research has shown that shear-wave velocity is the dominant influence on a dispersion curve, so usually only the S-wave velocities are varied during the inversion process (Xia, Miller, and Park, 2002). We utilize the Surfseis software developed by the KGS for our MASW analysis. For multi-array surveys, the individual shear wave models are combined into a 2D data set and then contoured in Surfer to produce a 2D cross-section model.

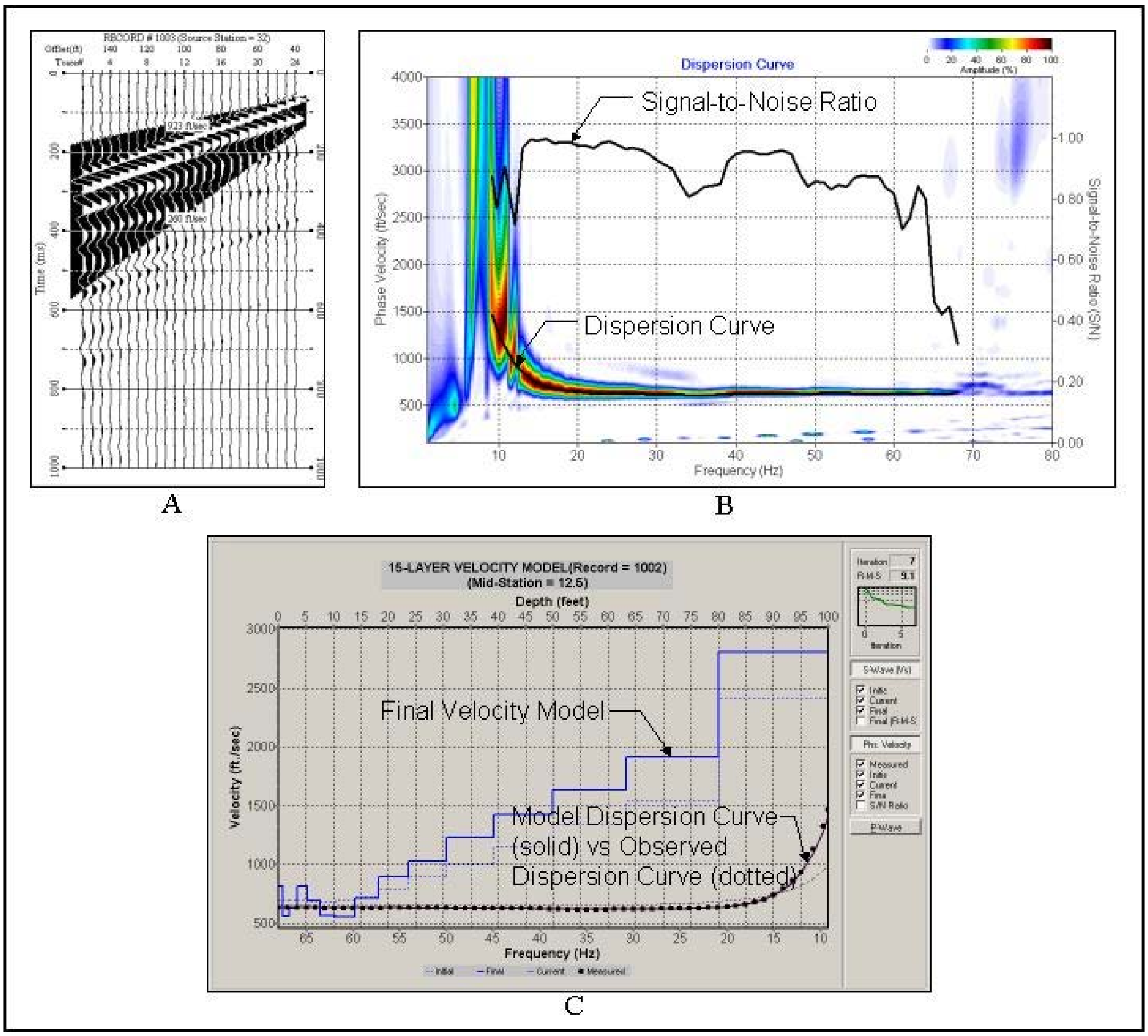

Figure 3 – MASW data processing using Surfseis. A) Seismic Time Series Data with Selected Surface Wave Energy (shaded), B) Surface Wave Energy in Frequency Domain with Observed Dispersion Curve, C) Shear Wave Velocity Inversion Model

EXAMPLE MASW APPLICATIONS

1D MASW for IBC Site Class Designation

The adoption of the International Building Code by various states beginning in 2000 led to the need for lower cost methods to determine site shear wave velocities. Available methods included downhole shear wave surveys, including seismic cone penetrometer testing (CPT), crosshole shear wave surveys, and seismic refraction. Crosshole and downhole surveys required drilling and casing holes to 100 feet depth or at least into competent rock, so were considered too costly for most IBC site class designations. Seismic refraction was not considered accurate enough for this purpose. The passive MASW method (ReMi) developed by Optim Software is used by some for this purpose.

In 2003, Schnabel Engineering developed a 235-pound portable weight drop source that could be quickly erected at a site and used to generate surface wave energy. Data are typically recorded using a single 24-channel array with a 5-foot geophone spacing and a 50-foot source offset. The weight drop source is activated up to about 10 times to stack sufficient surface wave energy. The data analysis are processed and modeled using Surfseis, as shown in the example in Figure 3.

The benefit of the active MASW method over other techniques for obtaining shear wave data for IBC site class designation is that it is rapid, is lower cost than borehole methods, can be performed over almost any surface conditions, and is dependable, since an active source is used and data collection is not dependent on background noise.

2D MASW for Abandoned Mine Detection

In 2004, Schnabel Engineering conducted MASW surveys to investigate an abandoned underground iron mine in the upper Midwest. Sinkholes had developed on the site and one building was experiencing cracking thought to be caused by subsidence. The goal of the geophysical investigation was to determine the approximate extent of the abandoned mine and the presence of incipient sinkholes.

We used a 48-channel land streamer array composed of 4.5 Hz geophones spaced at 5-foot intervals. Data were recorded using two RAS-24 seismographs, controlled by a laptop computer. The energy source was an 80-pound accelerated weight drop (AWD) source, activated by a large rubber band and striking an aluminum plate. The source interval was 10 feet, resulting in individual shear wave profiles every 10 feet along the ground surface. Surface conditions at the site included asphalt, concrete, mowed grass, and loose mine tailings. Data were also collected on the carpeted concrete slab of the bottom floor of the building most affected by the apparent subsidence.

Data were collected along three separate lines ranging from 350 to 465 feet in length. Surface wave data collected over the paved and grassed surface were high quality. However, the data collected over the loose mine tailings suffered from a lack of coherent surface wave energy, especially in the higher frequencies, most likely due to poorer geophone coupling and attenuation through the loose surficial materials.

In addition to processing the 48-channel data to provide deeper sections, the nearer 24-channels were processed separately to yield higher resolution, longer 2D sections. The MASW data analysis provided 2D sections to over 150 feet depth using the full 48-channel data, while the 24-channel data provided sections to about 70 feet depth. The deeper sections on the two lines over pavement and grass showed low velocity anomalies corresponding to the historic mine levels (Figure 4) while the line over the loose tailings provided a section with a similar velocity structure but no apparent lower mine anomaly.

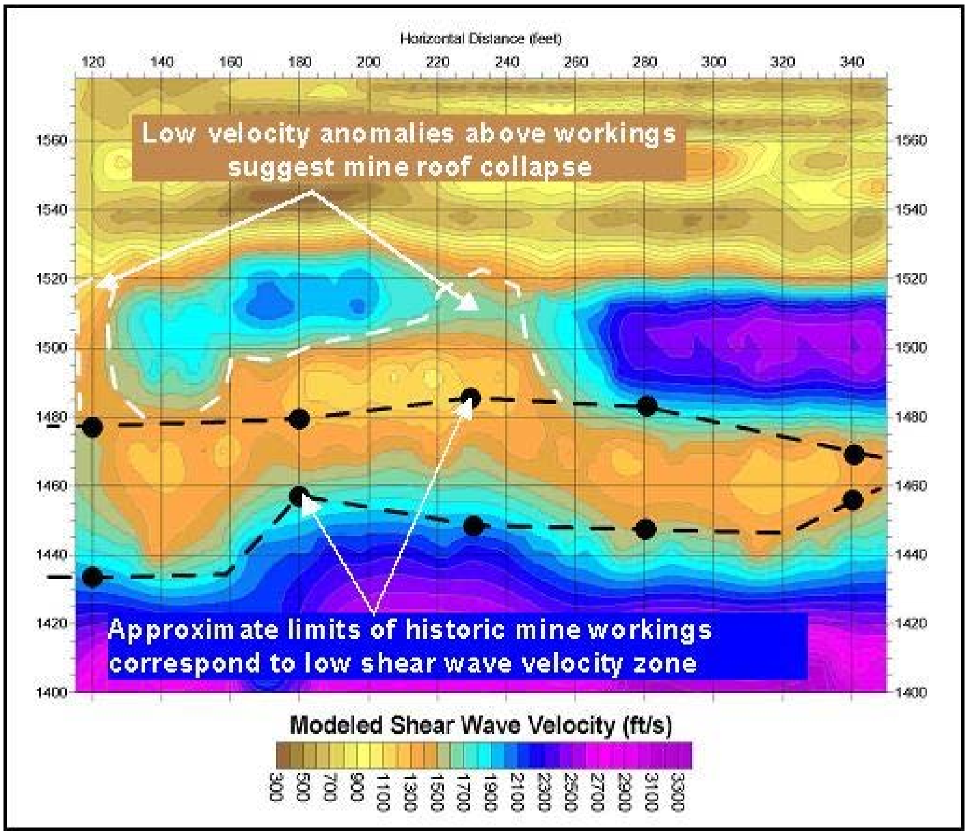

Figure 4– Example 48-channel MASW shear wave velocity model showing low velocity anomalies corresponding to historic mine workings level and to possible roof collapse zones above historic mine workings.

2D MASW for I-40 Landslide Investigation

In September 2004, floodwaters in the Pigeon River from Hurricane Ivan caused embankment slides that closed the two east-bound lanes of I-40 in North Carolina near the Tennessee border (Figure 5). Prior to obtaining design-build proposals for repair of the roadway embankment, the NCDOT Geotechnical Unit conducted a subsurface investigation to determine the depth to rock along several alignments. Centerline stationing was established by the NCDOT along the left lane marker of the left-hand east-bound lane as location control for the project.

Figure 5– Section of I-40 collapsing in September 2004 due to erosion of embankment by floodwaters in the Pigeon River from Hurricane Ivan.

The boring data showed the typical geologic profile to consist of a loose to very dense silty sand and stiff to hard sandy silt containing rock fragments (fill), overlying a layer of boulders (blast rock), followed by hard to very hard competent rock (metagraywacke and quartzite). All layers were of variable thickness with the top of rock varying from 11 to 32 feet below roadway surface along the alignment located 3 feet right of the centerline. The drilling investigation also indicates the rock line dips steeply towards the river.

Immediately following the boring investigation, the NCDOT requested that Schnabel Engineering provide geophysical surveys to tie the boring data together and provide a basis for a continuous rock profile. After examining several options, we decided to conduct MASW surveys to obtain 2D cross-sections of subsurface shear wave velocity. Unlike seismic refraction, for example, the MASW method was expected to yield good data in spite of noise from traffic in the adjacent lanes. It was also hoped that the shear wave velocity of the boulder layer would be significantly different from the underlying rock to produce a contrast on the MASW section.

MASW Data Collection and Analysis

MASW data were collected along the left-hand lane of the east-bound lanes at the main slide that closed the roadway and along the shoulder of the right-hand east-bound lane adjacent to a smaller slide about 1000 further east (Figure 6). The data were collected using a 24-channel seismic system consisting of a RAS-24 digital seismograph, 4.5 Hz geophones, cables, and a laptop computer to control the seismograph and record the data. A "Digipulse" accelerated weight drop (AWD) source striking an aluminum plate on the ground surface was used to generate the surface wave energy for the survey. Four to eight blows of the AWD were used at each shot location to generate sufficient surface wave energy. The source was offset from the nearest geophone by 30 feet. A 5-foot geophone spacing was used to provide an array length of 115 feet. A source spacing of 10 feet was used; after each shot, the array and source were moved forward 10 feet.

Figure 6 – MASW data acquisition along main slide on I-40 using 24-channel land streamer and AWD energy source. Photo looking east.

The data were analyzed using the Surfseis software, version 1.5, written by the Kansas Geological Survey. Analysis steps included parameter setup, filtering (as needed), recognition of surface wave energy, conversion to frequency domain, selection of a dispersion curve, and iterative modeling to produce an subsurface shear wave velocity model to match the selected dispersion curve. A single profile of shear wave velocity versus depth was produced by the modeling for each source and array location. The individual profiles were combined in Surfer to form a contoured cross-section of shear wave velocity versus depth. The MASW results were combined with the drilling data and plan data and used by the NCDOT to develop a top of rock map for use by contractors proposing on the remedial work.

MASW Line 1 – Main Slide

The 2D MASW model for Line 1 shows a velocity range of about 500 to 5000 feet/second (ft/s) with a maximum modeled depth of about 90 feet below ground surface (Figure 7). In general, the velocities increase with depth although some velocity inversions are present in the model. The projected location of the coincident and nearby borings are shown for correlation. Comparison with the results of Borings FB-10, FB-6, FB-5, and FB-4 show that the top of rock corresponds to a shear wave velocity of about 1500 to 2000 ft/s. This velocity probably represents the top of more highly weathered rock. The depth model indicates higher velocities more typical of competent rock (about 3000 ft/s) are reached at a depth of about 40 feet below ground surface (bgs) on the western half of the profile and about 30 feet bgs on the eastern half of the profile. The higher velocity material overlying the rock on the eastern half of the profile from about 10 to 30 feet bgs appears to correspond to a layer of boulders. The velocity of the rock below 40 feet bgs on the eastern half averages higher than that on the western half, suggesting the bedrock is less fractured/weathered on the eastern half. There is also a low velocity zone between Stations 18+00 and 18+60 that may represent a fault or fracture zone.

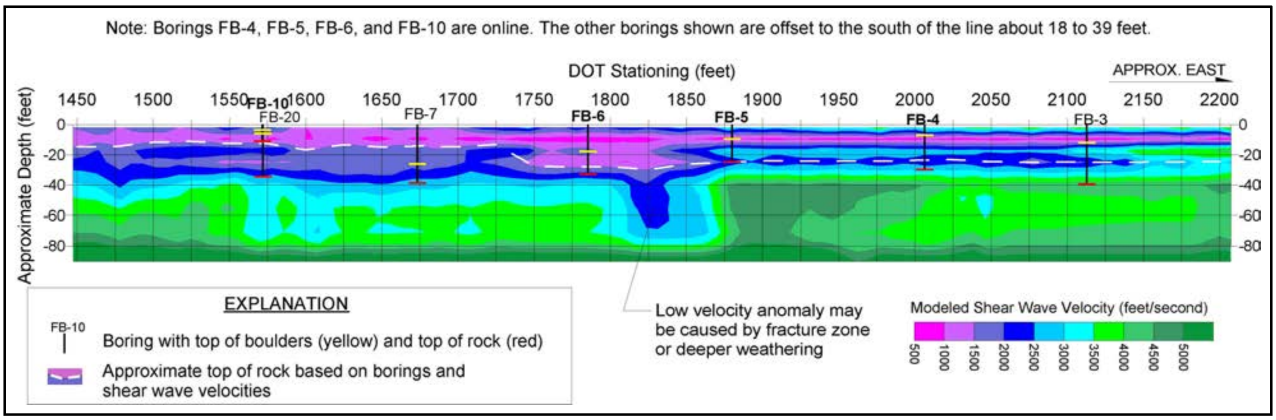

Figure 7 – MASW model for Line 1 on main slide. East is to the right.

MASW Line 2 – Smaller Slide

The model for Line 2 shows a velocity range of about 500 to 4000 ft/s and a maximum depth of about 60 feet bgs (Figure 8). Correlation with coincident borings indicate that the top of weathered rock corresponds to a shear wave velocity of about 1500 to 1750 ft/s. Depth to rock is approximately 12 feet below ground surface at the eastern end of the line, nearest the tunnel. This relatively shallow depth is confirmed by the results of Boring SB-9. The model indicates that the depth to top of rock increases from 12 feet at Station 35+50 (Boring SB-9) to about 35 feet bgs at Station 35+00. The top of rock on the western half of the model corresponds to a velocity inversion at about 35 feet bgs; the relatively high velocity zone overlying the rock from about 20 to 35 feet bgs appears to correspond to more competent, massive boulders.

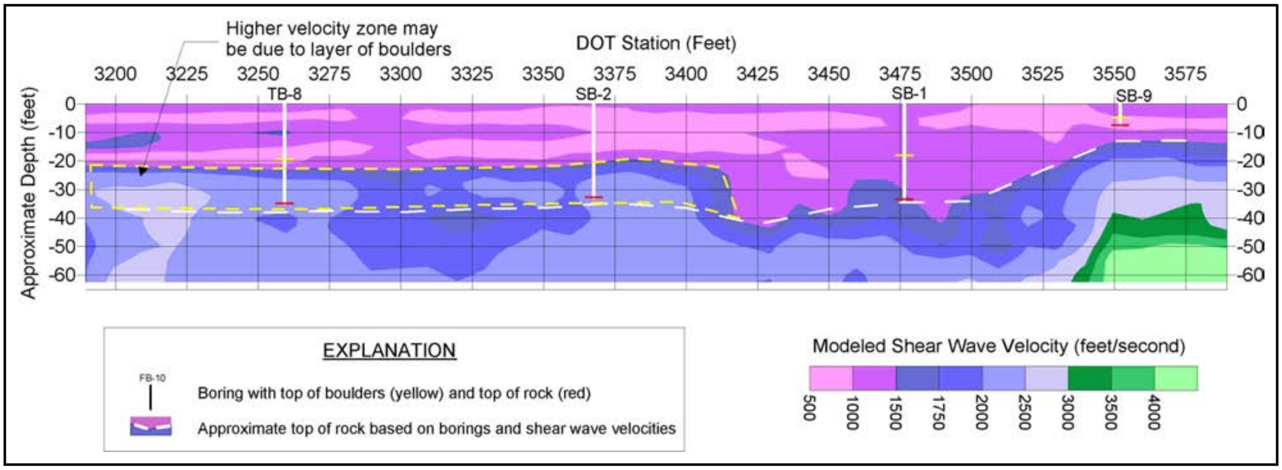

Figure 8 – MASW model for Line 2 on smaller slide. East is to the right.

Comparison to As-Built Data

Top of rock data from tiebacks and vertical piles installed during remedial construction provide as-built data to examine the accuracy of the MASW results. These data were superimposed on the MASW top of rock line and the top of rock from the 2004 borings (Figure 9). Variations are expected due to some averaging over the length of the MASW array and due to out-of-plane variations in the depth to rock. While the MASW results on Line 1 match the boring data fairly well, the difference between the MASW and the tieback data ranges from about 5 to 15 feet. However, the majority of the tiebacks encountered rock about 13 to 19 feet right (south) of the station centerline, downslope of the area imaged by the MASW method. The low bedrock anomaly on the tieback curve from Station 1800 to 1825 does correspond to a low velocity anomaly on Line 1, suspected to be a fracture zone or area of deeper weathering. The correlation between the as-built and the MASW for Line 2 is very good and has a typical variation of about 5 feet.

CONCLUSIONS

In just a few years, the MASW method has proven to be a powerful geophysical technique for subsurface investigations. The ability of MASW to image subsurface stiffness in areas of background seismic noise and buried utilities fills a gap in the geophysical methodology. While limitations in resolution and surface exist, the towed land-streamer method makes MASW a valuable technique for roadway investigations for abandoned mines, karst features, and depth to rock.

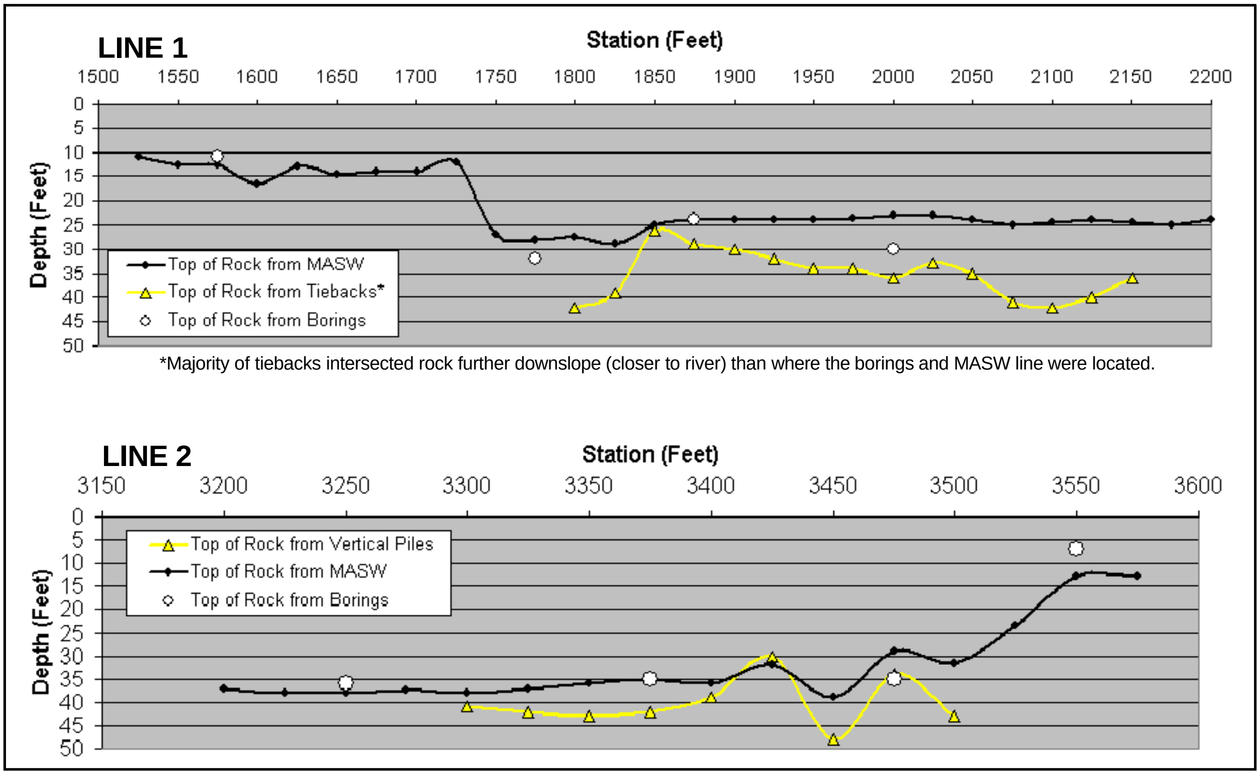

Figure 9 – Comparison of top of rock from MASW models, boring data, tiebacks and sheet piles for Line 1 (main slide) and Line 2 (smaller slide). East is to the right. Apparent misfit between the tieback data and the MASW and boring data on Line 1 is likely due to steeply sloping and rapidly varying top of rock interface. The tiebacks intersected rock closer to the river than where the MASW and boring data were located. Top of rock is probably less variable in the vicinity of Line 2, as indicated by the good fit between the piling depths and the MASW and boring data.

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EXHUMING ROCK REINFORCEMENT
Barron Mountain, Woodstock, New Hampshire

By: Richard Lane[1], Ken Fishman[2] and Andrew Salmaso[3]

ABSTRACT

In 1972, during the construction of Interstate 93 in Woodstock, NH, a rockslide occurred at the base of Barron Mountain. The slide, consisting of approximately 17,000 cubic yards of rock, buried the I-93 northbound barrel. A redesign of the roadway was immediately undertaken to include stabilization of the rock slope by installing extensive rock reinforcement and instrumentation. Continuous plots of the instrumentation readings were maintained until the mid 1980's, when the last of the active instruments stopped working. Visual inspections of the rock slope and the reinforcement have been conducted periodically since construction.

Longevity of the reinforcement at the Barron Mountain rock slope is a concern of the New Hampshire Department of Transportation, since more than half of the generally accepted 50-year service life has passed. The NHDOT contracted with McMahon & Mann Consulting Engineers, P.C. to perform a two-phased study on evaluation of the rock reinforcement at Barron Mountain. Phase I included non-destructive testing, condition assessment and service life estimates for the rock reinforcement. Phase II involved invasive testing of selected rock reinforcement to verify the results of the Phase I study. This paper describes the fieldwork conducted as part of Phase II, and the challenges encountered in exhuming existing rock reinforcement.

The Phase II fieldwork consisted of integrity testing of selected rock bolts, scaling of loose rock, removal of an unstable block, installation of replacement rock reinforcement, proof testing of replacement rock bolts, lift-off testing of several existing rock bolts, the installation of strain gages along two 60-foot long tendons, over-coring existing rock reinforcement, and exhuming portions of four resin grouted, pre-stressed rock bolts and one cement grouted, passive tendon.

Exhuming rock reinforcement is a unique and difficult task. The over-coring was accomplished with two types of drill rigs both using water to flush drill cuttings. In most cases, recovery was accomplished by over-coring along a segment of the reinforcement at an angle slightly different from the drill hole, until the diamond drill bit encountered and cut through the steel. Difficulties included no grout within the free stressing zone of the rock bolts, deviation of the drill holes for the existing rock reinforcement, steel couplings, maintaining constant down pressure, anchoring the drill rig, alignment of the drill and

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core barrel with respect to the rock bolt, jamming of the core barrel, cutting the reinforcement and access for the drill rig.

Although sometimes a slow and tedious process, exhuming rock reinforcement is feasible. Detailed information regarding the existing rock reinforcement is critical to a successful outcome. Information should include the type of reinforcement, type and extent (full or partial) of grout, diameter of the drill hole and of the steel reinforcement, depth and orientation of the reinforcement, existence and location of couplings, location of seams and joints, and a detailed sketch map of the slope.

INTRODUCTION

On November 7, 1972 a rockslide occurred at the base of Barron Mountain during the construction of Interstate 93 in Woodstock, New Hampshire (Figure 1). The slide, consisting of approximately 17,000 cubic yards of rock, completely buried the I-93 northbound barrel (Fowler, 1976a and 1976b). This resulted in a significant delay to the roadway construction, while the project was redesigned. The redesign involved changes to the interstate alignment, redesign of the rock slope, construction of a concrete retaining wall, relocation of a segment of NH State Route 175, construction of three new bridge structures for the relocated route, horizontal drains to reduce water pressure in the slope, installation of rock reinforcement to stabilize the rock cut and instrumentation to monitor for further movement (Haley & Aldrich, 1973a). The instrumentation included extensometers, strain gages and load cells (Haley & Aldrich, 1973b). Instrument readings along with continuous plots were maintained until 1985, when the last of the active instruments stopped working. Although inspections of the rock slope and the reinforcement are conducted annually, there has been no method for determining the actual condition of the existing rock reinforcement.

Figure 1. *Barron Mountain Rock Cut*

A limited performance history and the difficulty in accurately determining the condition of buried rock reinforcement elements make the longevity of these types of systems a critical issue on civil engineering projects. Portland cement-grouted rock reinforcement was initially used in tunneling and underground construction in the mid-1950s. Polyester resin grouted reinforcement was utilized in the United States in the mining industry in the late 1960s and in tunneling in the early 1970s (Kendorski, 2000). The Barron Mountain rock cut was one of the first sites in the United States to use polyester resin grouted rock bolts to stabilize a highway rock slope.

A study, conducted in the Yxhult Mineral AB's Centralgruvan Mine in Sweden, over-cored different types of rock reinforcement installations in a corrosive underground environment. The study compared the degree of corrosion relative to the age of the different rock reinforcement systems (Helfrich, 1990). The Curtin University of Technology, Western Australia School of Mines and Corrosion Research Centre have been conducting research in underground mining to determine corrosion mechanisms affecting rock reinforcement and to assess the effectiveness of corrosion classifications. As part of the field-testing portion of the research, bolt over-coring was undertaken to study the condition of the rockmass, the grout quality, integrity of grout encapsulation and the degree of corrosion within the reinforcing steel (Hassell, 2004). Other studies conducted at coal mines, tunnels, underground construction sites, hydroelectric projects and a rock anchor tie-back retaining wall have looked at corrosion and longevity of rock reinforcement. Some studies have indicated potential longevity problems with resin grouted rock reinforcement in permanent installations. It has been suggested that most of these problems were due to difficulties in the installation and the failure to follow the manufacturer's recommendations (Kendorski, 2000). The study at Barron Mountain is the first known attempt to recover and analysis the condition of rock reinforcement at a highway rock slope.

LOCATION AND SITE CONDITIONS

The rock cut is located at the base of Barron Mountain in Woodstock, New Hampshire, approximately 60 miles north of Concord, NH. The site, which overlooks the Pemigwasset River, is situated on the east side of Interstate 93 between exits 30 and 31 (Figure 2). The rock slope reaches a maximum height of 130 feet, is 600± feet in length and has a 30-foot wide rock bench at approximately 90 feet above ditch elevation along the southern portion of the cut. The southern half of the rock cut is composed of quartz-mica gneiss, which grades into foliated, quartz-mica schist in the northern section. A large andesite dyke, exposed the entire height of the cut face, intrudes into the country rock along the contact between the two rock formations. Smaller basalt dykes are visible on the rock face. The rockslide occurred along a highly fractured, mylonite zone, which dips toward the road at approximately 38 degrees (Fowler, 1976a and 1976b). The remaining scar from the 1972 slide is located at the north end of the rock cut and is visible in he lower left corner of Figure 1.

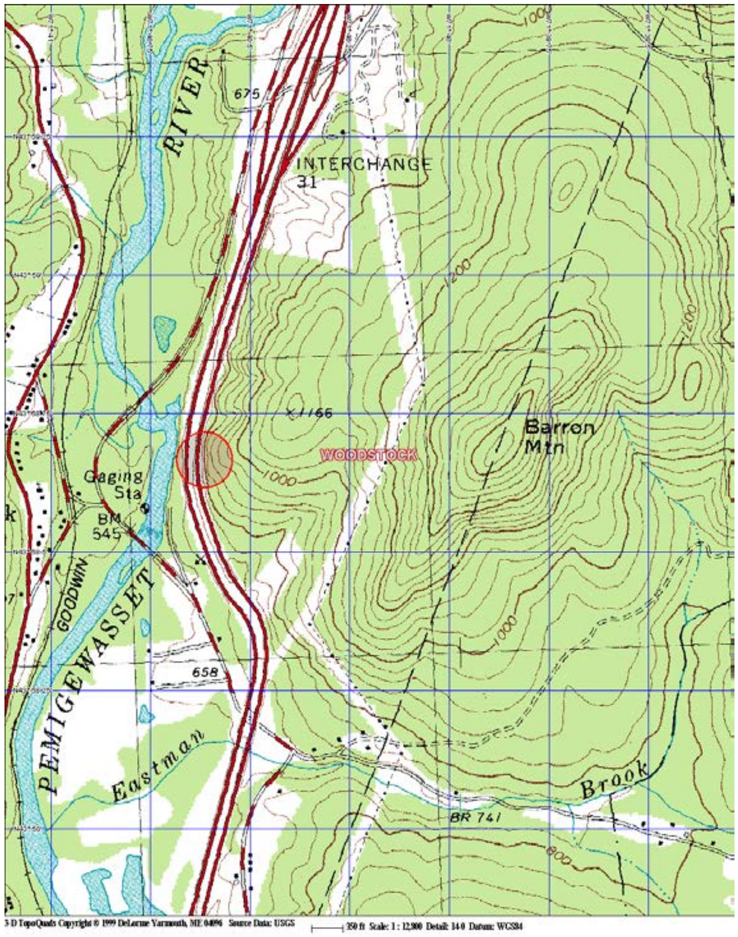

Figure 2. *Location Map*

Both active and passive reinforcements were installed at the Barron Mountain site. The passive reinforcement consists primarily of three rows of 60 foot long tendons with no anchorage assembly, installed in a 10’ X 10’ grid pattern along the toe of the rock slope. Additional tendons were installed in the upper portion of the rock slope above the slide area. The tendons are 1.25 inches in diameter, Dywidag, Grade 150, continuously threaded, solid steel bars, which are encapsulated in cement grout along their entire length. In general, the tendons were installed at an upward angle of 25 to 30 degrees from horizontal (Haley & Aldrich, 1974). The primary purpose of the tendons is to prevent large-scale failures in the rock slope. The active reinforcement consists of polyester resin grouted, pre-stressed rock bolts to secure existing blocks and to tie together the rock mass. The rock bolts are 1 inch in diameter, Dywidag, Grade 150, continuously threaded, solid steel bars which are grouted along the anchor zone with polyester resin grout. A small number of the rock bolts are Bethlehem Steel, Grade 80, continuously threaded, solid steel bars (Haley & Aldrich, 1974). The pre-stressed rock bolts are end point anchorages secured with a bearing plate and nut at the rock face. The unbonded, free-stressing portion of the rock bolts is not grouted and is unprotected. The rock bolts ranged in length from 10 to 30 feet and were initially pre-stressed to 20 or 40 kips, depending on the grade of steel. In most cases, the rock bolts were oriented approximately perpendicular to the rock surface (Figure 3). The existing drill holes were 2 ½ to 3 inches in diameter. The holes for the resin grouted rock bolts were drilled at a smaller diameter (1 3/8 inches) in the anchor zone. Approximately 100 tendons and more than 150 rock bolts were installed at the Barron Mountain site.

Figure 3. *Pre-stressed, resin grouted rock bolts, center section*

Longevity of the reinforcement at the Barron Mountain rock slope is a concern of the New Hampshire Department of Transportation, since more than half of the generally accepted 50-year service life has passed. A two-phased research study was undertaken to evaluate and to assess the condition of the rock reinforcement at the Barron Mountain site. Phase I included non-destructive testing, condition assessment and service life estimates for the rock reinforcement (Fishman, 2004). Phase II involved invasive testing of selected rock reinforcement to verify the results of the Phase I study. The Phase II fieldwork consisted of integrity testing of selected rock bolts, scaling of loose rock, removal of an unstable block, installation of replacement rock reinforcement, proof testing of replacement rock bolts, lift-off testing of several existing rock bolts, the installation of strain gages along two 60-foot long tendons, over-coring existing rock reinforcement, and exhuming portions of four resin grouted, pre-stressed rock bolts and one cement grouted, passive tendon.

INTEGRITY TEST

The first task performed during the Phase II fieldwork was the GRANIT Integrity test, which was conducted on selected rock bolts at the site (Figure 4). This is a type of non-destructive impact test, which is utilized in the mining industry to evaluate the condition of rock bolts. The test can be used to determine the pre-stress load on the rock bolt and to assess the condition of the grout near the proximal end of the reinforcement. AMEC Group Ltd., a company from the United Kingdom, conducted their patented test on a total of 56 rock bolts. The intent was to compare the GRANIT test results with information gathered from other non-destructive test techniques utilized at the site and with results from invasive testing conducted during Phase 2. The other non-destructive test procedures utilized at Barron Mountain included half-cell potential, polarization current, impact and ultrasonic. The invasive testing consisted of lift-off tests, and testing of steel and grout samples retrieved from exhumed rock reinforcement.

Figure 4. *Non-destructive GRANIT Integrity test*

SCALING, REMOVAL OF UNSTABLE BLOCK AND REPLACEMENT OF ROCK REINFORCEMENT

Replacement rock bolts and tendons were installed prior to over-coring and exhuming any of the rock reinforcement (Figure 5). Initially, the plan was to exhume six rock bolts and two tendons. It soon became evident that this was an optimistic goal, due to difficulties in the drilling process and unknown conditions relating to the installation of the existing rock reinforcement. Before starting the replacement work, the rock slope was hand scaled to remove any loose rock that could pose a potential threat to the workers. Rock remediation technicians of JANOD Contractors worked off of ropes, scaling the slope from the top to bottom (Figure 6). A large block, located in the vicinity of the andesite dyke, was determined to be unstable. The block had become detached from the surrounding rock and was precariously hanging from a single rock bolt. The Boulder Buster™ rock breaking equipment, a trigger device with a small charge encased in a shotgun size shell, was used to split the block. A rubber mat was draped over the block and the rock-breaking device was inserted into a small hole that had been drilled with a jackhammer (Figure 7). Detonation of the device split the block into two pieces and severed the steel rock bolt (Figure 8). The rock fragments were separated from the slope and landed in the ditch without damaging the surrounding rock.

Figure 5. *Drilling replacement holes for rock reinforcement*

Figure 6. *Hand scaling*

Figure 7. *Rubber mat draped over block*

Figure 8. *Splitting the detached block with the Boulder Buster™ device.*

Strain gages were attached at intervals along the two replacement 60-foot long tendons (Figure 9). Five vibrating wire strain gages were installed on each tendon at distances of 5, 15, 25, 35, and 45 feet from the proximal end. The strain gages were mounted on the steel bars with epoxy and protected with metal covers. Electrical cables from each strain gage ran along the bar, passed through the plastic centralizers attached to the steel tendons, exited at the proximal end of the bar, ran along the rock face inside protective PVC tubing and were connected to an instrument readout box. The strain gages will be read periodically to monitor the rock mass performance and to detect changes in strain on the tendons. Care was taken during the installation of the tendons to avoid damaging the gages and the electrical cables.

Figure 9. *Preparing 60-foot long tendons*

The next task in the Phase II preparation work was to install the replacement rock reinforcement to include six cement grouted, pre-stressed rock bolts and two cement grouted, passive tendons. The replacement rock bolts ranged from 15 to 30 feet in length and the two passive tendons were each 60 feet long. The replacement holes were drilled with a small air track, wagon drill mounted on a steel-framed trailer with rubber-tired wheels and a winch (Figure 10). The drill rig weighs approximately 1000 lbs and is completely powered by compressed air. These rigs are compact and very maneuverable. They can change angles, drill on a vertical face or even drill at an inverted angle. The replacement bolts and tendons were fully grouted with a 300 PT Sika Grout. The compressive strength of grout samples after 72 hours was measured at 8300 psi. All the pre-stressed rock bolts were proof tested for compliance in accordance with procedures recommended by the Post-Tensioning Institute (PTI, 1996).

Figure 10. *Air track drill with winch mounted on wagon*

The replacement rock bolts and tendons were installed with no couplings and grouted along their entire length. The 1.25 inch diameter steel bars for the passive tendons arrived at the rock cut in 60-foot long stock length. This is the longest continuous length of steel bar that the manufacture can deliver to a project site. The 60-foot long steel tendons were installed with a crane utilizing a rope sling configured to approximate the upward installation angle of the tendons (Figure 11). The installation was completed with minimal disruption to the interstate traffic.

Figure 11. *Installing 60-foot long steel tendon*

Lift-off tests were performed on seven existing pre-stressed rock bolts to determine the magnitude of their actual loads. A hydraulic, center hole jack was used to apply a load to the bolt and to lift the bearing plate from the rock surface (Figure 12). The measured loads for the tested rock bolts ranged from 7.2 to 38.3 kips. Only two of the seven bolts tested had loads that were close to the original design load of 40 kips. Five of fifty-six rock bolts had loose or slack plates. The loss of pre-stress could be the result of several factors to include uneven grout coverage, inadequate bond length,

Figure 12. *Lift-off test for pre-stressed rock bolts*

deteriorating grout, loss of grout in voids or fractures, water bearing discontinuities in the anchor zone, movement in the rock slope, redistribution of the load or poor installation procedures.

OVER-CORING AND EXHUMING ROCK REINFORCEMENT

Over-coring and exhuming existing rock reinforcement is a unique operation with difficult challenges and unknowns. The process requires patience and a willingness to be innovative. Site conditions and the initial installation procedures for the existing rock reinforcement have a significant impact on the method of operation and the potential for a successful outcome.

The equipment used to over-core two rock bolts and one tendon was a Boart Longyear, MetreEater pneumatic diamond core drill with rotary head mounted on the contractor's rubber tired, steel framed wagon (Figure 13). This drill is a screw-feed machine capable of advancing AQ rod horizontally up to 200 meters in depth. The machine is light-weight (500 lbs, drill only) and suitable for both underground and surface rock drilling. The pneumatic core drill utilized drill casing in two-foot long sections and a 4-inch inside diameter, diamond impregnated bit (Figure 14). A third rock bolt, located at the toe of the rock slope approximately 6 feet above the ditch level, was over-cored by the NHDOT utilizing a CME-45C drill mounted on a CME tracked Carrier. The CME drill is powered by an air-cooled 3-cylinder diesel engine. Drill casing utilized with the CME rig was 5 feet in length. The drill bit used by the NHDOT was a PW heavy-duty casing shoe with a 4.6+ inch inside diameter. This bit is impregnated with a high concentration of diamonds to give it maximum performance under severe conditions. Both drill rigs use water to flush drill cuttings from the hole and both are capable of angle drilling. A segment (4 feet long) of a fourth rock bolt was recovered from splitting a block with the Boulder Buster™.

Figure 13. *Pneumatic core drill*

Figure 14. *Two foot long casing and diamond drill bit*

The recovery of the rock reinforcement was accomplished by over-coring along a segment of the steel bars at an angle slightly different from the existing drill holes, until the diamond drill bit encountered and cut through the steel. All the existing drill holes showed deviation along their length, making it impossible to predict where the drill bit would encounter the rock reinforcement. In several cases, the existing holes began to deviate within 3 feet of the proximal end. Access for the over-coring equipment can be an issue depending on the type and size of the drill rig. The contractor's wagon drill could maneuver to any location on the slope, while the larger NHDOT track mounted drill would require a platform and crane to reach locations higher on the rock slope. Down pressure and speed of advancement depend on the ability to secure the equipment to the rock slope, and the type and size of the drill. The contractor's drill was secured to the rock face with cables and straps (Figure 15). The rate of advancement was limited by a screw fed mechanism and by the overall light-weight of the contractor's drill equipment. The NHDOT drill, which weighs over 11,400 lbs. and has a hydraulic feed system, could exert greater down pressure and advance at a faster rate (Figure 16).

Figure 15. *Over-coring (Janod Contractors)*

Figure 16. *Over-coring (NHDOT)*

Since there was no grout along the unbonded free stressing length of the rock bolts, the unprotected portion of the steel bars would move and flex when encountered. The drill bit would sometimes travel for a distance along the bar without cutting into the steel. Although the field notes from the installation of the existing rock bolts were detailed and comprehensive, the extent of the grout cover and the coupling locations were not identified. The four rock bolts that were recovered during the fieldwork were all partially grouted. This information supports the assumption that all the pre-stressed rock bolts were grouted only in the anchor zone with no protective cover along the free stressing length. The passive steel tendons were completely encapsulated in cement grout and held tightly in the hole, so that movement of the bar could not occur. During over-coring of the tendon, the diamond bit encountered and cut along the entire length of a coupling, which was located within 5 feet of the proximal end. A total of 25.75 feet of one-inch diameter, rock bolts and 13 feet of 1.25 inch diameter, steel tendons were recovered for testing (Figures 17–22). In addition, samples of resin grout and cement grout were recovered for analysis.

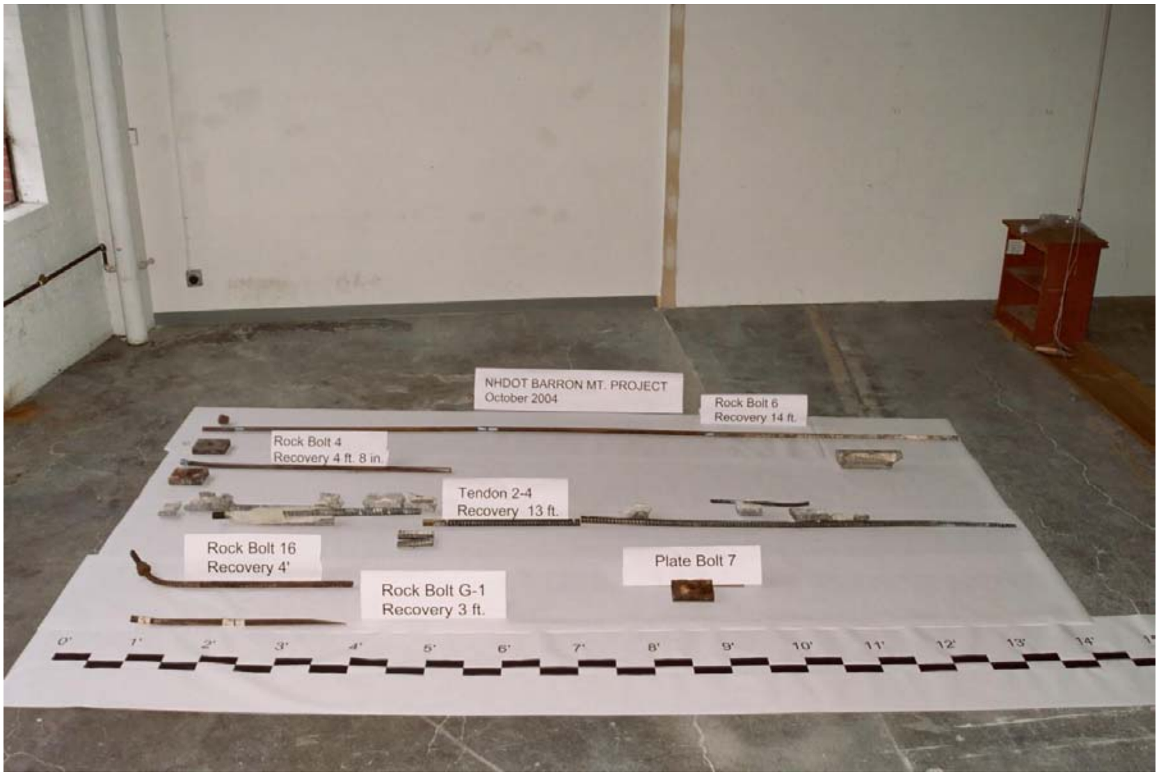

Figure 17. *Recovered rock reinforcement*

Figure 18. *Rock bolt with resin grout*

Figure 19. *Pitting and cratering along rock bolt*

Figure 20. *Loss of cross section*

Figure 21. *Corroded rock bolt*

Figure 22. *Recovered 1.25 inch diameter, steel tendon and cement grout*

Grout cover, location of couplings and alignment of the drill hole are all characteristics of the existing rock reinforcement that will impact the over-coring process. Although the site conditions and the characteristics of the rock reinforcement are fixed, the process for exhuming them can be modified. The method utilized should be tailored to the existing conditions. The first and most critical step in the exhumation process is the alignment of the drill and core barrel with respect to the orientation of the existing rock reinforcement. This is challenging because the orientation of the existing drill hole may not be consistent along its entire length. After drilling has started, the orientation of the core barrel cannot be changed without pulling out of the hole, reorienting the core barrel and then re-drilling the hole. This operation was repeated numerous times at the Barron Mountain site in an attempt to follow wandering drill holes and to increase the recovery length of the rock reinforcement. The inside diameter of the drill bit utilized in the over-coring process is important, particularly if the rock reinforcement has couplings and the bar is not fully grouted. If the diameter of the bit is too small it may get hung up on a coupling or it may cut across the rock reinforcement too soon. If the rock reinforcement is not fully grouted, it could increase the potential for jamming of the core barrel with rock fragments falling in from the wall of the drill hole. If the diameter of the bit is too large, the recovered core may be difficult to remove from the hole and the rate of advancement may decrease. The maximum size of the drill bit will also be dependent on the capability of the drill rig. At the Barron Mountain site some improvement in the exhuming process for the rock bolts may have been realized by pre-grouting the free stressing zone (unbonded length) and by utilizing a diamond bit with a larger inside diameter. When drilling at a shallow angle, holes should be drilled at a minimum of 3 to 5% upward from horizontal to facilitate removal of water and drill cuttings. The quality of the rock, rate of advancement, location of weathered/fractured zones and seams should be noted when drilling the replacement holes for the rock reinforcement. Under some circumstances it may be advisable to drill and recover NX-size rock cores adjacent to the actual over-coring location to determine the condition of the rock.

INFORMATION RECOMMENDED BEFORE EXHUMING ROCK REINFORCEMENT

Over-coring and recovery of existing rock reinforcement can be challenging, time consuming and expensive. Key information is needed before selecting the equipment and the method of operation. This information falls into three categories to include the site conditions, characteristics of the existing rock reinforcement and the original installation procedures. It is recommended that the following information be gathered prior to starting the exhuming process:

<u>Site Conditions</u>

- Date and method of rock slope excavation.
- A detailed sketch map of the rock slope showing the location of the rock reinforcement; height of rock reinforcement above ground level.
- Potential access for the drilling equipment and available space at the toe of the slope.
- Overhead utilities, site distance along the roadway, traffic control issues.
- Depth, extent and orientation of seams, fractured or weathered zones, major discontinuities (joints, shear planes, etc.).
- Discontinuities - spacing, persistence, aperture and infilling material.
- Rock type and overall condition of rock (hardness, degree of weathering, fracturing, etc.).
- Hydrology (presences of water in discontinuities, degree of flow, staining, precipitates)
- Photo documentation (before, during and after initial installation)

Characteristics of Rock Reinforcement

- Reinforcement material and anchor device
- The type(s) of reinforcement to include diameter, length and location of couplings.
- Bearing plates (dimensions), nuts, washers, other accessories.
- Diameter and orientation of the existing drill holes.
- The type of grout, location of the grout (full or partial coverage) and thickness of grout cover.
- Corrosion protection and coatings.
- Type and location of centralizers.

Installation Procedures

- Date of installation.
- Size and number of grout cartridges used (resin grout).
- Amount of cement grout used in each drill hole.
- Drilling equipment utilized.
- Does the drill hole deviate along its length? If so, where, how much and in what direction?
- Was a grout tube used? Was the tube removed or left in place?
- Does the diameter of the drill hole change along its length?
- Project specifications for installing the original rock reinforcement.
- Type and location of instrumentation (extensometers, strain gages, load cells, etc.).
- Pres-stressed loads on rock reinforcement during initial installation.

The procedures utilized for installing the existing rock reinforcement can have a significant impact on the exhumation process. Adherence to the manufacture's recommendations and/or the project specifications is not only critical for quality control, but important in the ability to the development a successful plan for exhuming rock reinforcement. Information on the rock reinforcement and the installation procedures are often not available or lacking in details.

CONCLUSIONS

The primary purpose for exhuming selected rock reinforcement was to verify the results from the non-destructive testing and the service-life estimates performed for the Barron Mountain site. Although the exhumation of existing rock reinforcement can be a challenging, a time consuming and an expensive process, it is the most direct method for determining their condition and for estimating their remaining longevity. Detailed knowledge of the site conditions, characteristics of the rock reinforcement and the installation procedures are important in developing a successful plan for recovery. A thorough investigation is needed to determine the most cost effective method(s) to assess the condition of the rock reinforcement at a site. Both non-destructive and invasive test methods should be considered. The analysis of the data and samples collected during the Phase II study, and direct comparison of results from the invasive testing and NDT are described in a companion paper, "Condition Assessment Of Thirty-Year Old Rock Reinforcement", submitted to the 2005 Highway Geology Symposium.

ACKNOWLEDGMENTS

McMahon and Mann Consulting Engineer's, P.C. conducted both phases of the research under contract with the NHDOT; AMEC Group Ltd. conducted the GRANIT integrity testing; JANOD Contractors was the specialty contractor who performed lift-off testing, installed replacement reinforcement and over-cored the existing reinforcement during the Phase II study. Phase II was organized as a pooled-fund study with contributions from the state DOTs of New Hampshire, New York and Connecticut, and the Federal Highway Administration (TPF-5(096)).

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CONDITION ASSESSMENT OF THIRTY-YEAR OLD ROCK REINFORCMENTS

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ABSTRACT

Thirty-year old rock reinforcements at the Barron Mountain rock cut along I-93 near Woodstock, NH are the subject of condition assessment and estimation of remaining service-life. Two types of rock reinforcements are installed at Barron Mountain including: (1) partially bonded, resin grouted, prestressed rock bolts, and (2) fully bonded, Portland cement grouted, passive tendons. The two-year project includes nondestructive testing (NDT) of selected elements (Phase I), and invasive testing (Phase II) to verify results from Phase I. In another paper submitted to this symposium, the second author describes fieldwork conducted as part of Phase II. This paper describes analysis of data and samples collected during Phase II, and direct comparison of results from invasive testing and NDT.

INTRODUCTION

Background

In 1972, during the construction of Interstate 93 in Woodstock, NH, a rockslide occurred at the base of the Barron Mountain rock cut. Details of the slide and subsequent slope remediation and redesign of the highway are described by Fowler (1976(a)). A redesign of the roadway was immediately undertaken to include stabilization of the rock slope by installing extensive rock reinforcement (Figure 1) and instrumentation. Fifty to sixty feet long rock tendons were installed to counteract sliding along the anticipated sliding failure plane. Shorter, 10 to 30 feet long, rock bolts were installed to keep the rock mass intact; to preserve the full gravity effect of the rock bench used to maintain global stability, and to prevent minor rock falls onto the highway.

The estimated design life of unprotected rock reinforcement systems is approximately 50 years (Kendorski, 2003). The New Hampshire Department of Transportation (NHDOT) is concerned with the longevity of the system given half the anticipated design life has passed. To address this concern the NHDOT undertook condition assessment and evaluation of the thirty-year old rock reinforcements at Barron Mountain. The condition assessment followed the recommended practice from NCHRP Project 24-13 (NCHRP, 2002) and was performed in two phases implemented in the summer and fall of 2003 and 2004.

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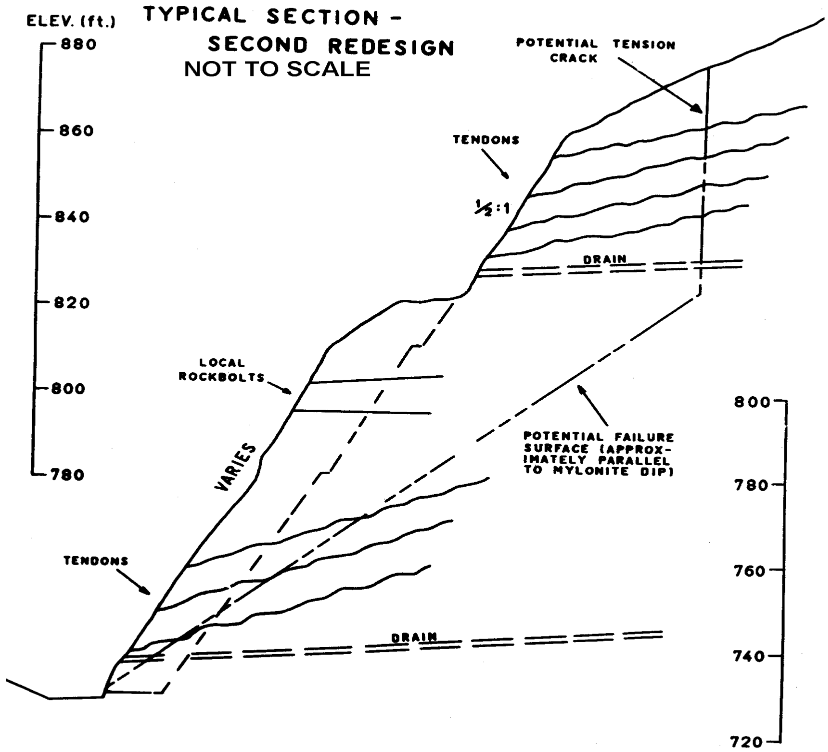

Figure 1. Typical Cross Section of Rock Cut Showing Rock Reinforcement (Fowler, 1976(b))

Phase I & Phase II Evaluation

Phase I of the condition assessment included an evaluation of site conditions, a review of installation details, estimation of remaining service life and condition assessment using nondestructive testing. An interim report *"Phase I: Condition Assessment and Evaluation of Rock Reinforcement Along I-93, Barron Mountain Rock Cut, Woodstock, New Hampshire,"* describes details from the Phase I condition assessment (Fishman, 2004).

The second phase of the project (Phase II) consists of invasive testing of selected rock bolts and tendons to verify results from Phase I. Invasive testing includes lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Corrosion of reinforcements is observed in terms of surface distress and metal loss. Data from Phase II are compared to results and interpretations from NDT. The comparison is in terms of qualitative and quantitative condition assessment relative to the reinforcement population at the site, as well as features and attributes observed for specific reinforcements.

SITE CONDITIONS

Lane et al. (2005) describe details of the geometry the rock cut (see Figure 1, Lane et al., 2005) and rock conditions. Additional details are required to assess the corrosiveness of the rock mass and the vulnerability of the reinforcements to metal loss. Generally, moisture content, chloride and sulfate ion concentration, resistivity and pH are identified as the factors that most affect corrosion potential of metals underground. Quantitative guidelines are available for assessing the potential aggression posed by an underground environment relative to corrosion (FHWA, 1993).

Samples of the weathered rock and groundwater were collected to evaluate the corrosiveness of the rockmass. The measured pH (4.2 to 5.1), resistivity (4000 Ω-cm), and moisture conditions within the weathered rock correspond to a corrosive environment. Measured sulfate and chloride ion concentrations (650 ppm and 720 ppm, respectively) are also at levels high enough to be conducive to a corrosive environment. The corrosiveness classification at the site is between II and III, on a scale where “I” is considered highly corrosive and “IV” is slightly corrosive (FHWA, 1993). This rating is used to estimate the rate of metal loss anticipated over the service life of the reinforcements.

Details of Rock Reinforcements

Figures 2 and 3 portray the rock bolt and tendon installations, respectively. Rock bolts and rock tendons include 1 inch or 1.25 inches diameter steel threadbars. Most of the reinforcements are Dywidag, Grade 150, high-strength prestressing steel threadbars. Some rock bolts are Grade 80 threaded steel rods supplied by Bethlehem Steel. Prestressed rock bolts are essentially end point anchorages, grouted at the distal end with polyester resin grout, and supported by an anchorage assembly consisting of a nut and a bearing plate at the rock face (proximal end). Rock bolts were initially prestressed to 20 or 40 kips depending on the steel grade. Tendon elements are fully grouted with Portland cement grout, and the proximal ends are recessed into the rock mass. The tendons are passive elements, i.e. they were not prestressed, and there is no anchorage assembly.

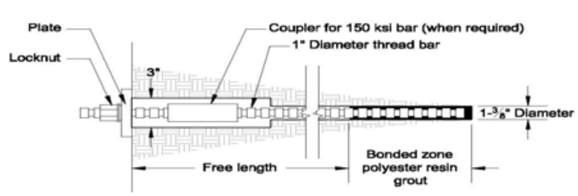

Figure 2. Rock Bolt Details

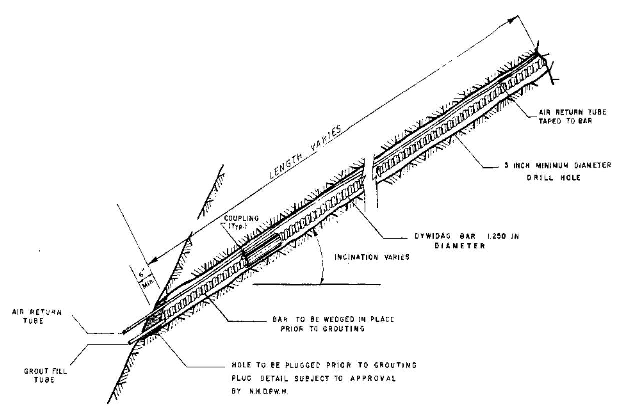

Figure 3. Detail of Rock Tendon (Haley and Aldrich, 1973)

Due to the different installation details including grout type, method of grouting, anchor head details, drillhole diameter, and the lengths of the reinforcements, we considered rock bolt and tendon reinforcements separately for the purpose of condition assessment. Grout type is an especially important detail. Portland cement based grout is alkaline and protects the steel reinforcement by passivating the steel as well as providing a barrier to moisture and oxygen. However, passivation of the steel may be compromised by the presence of chlorides or acidic conditions. Polyester resin grouts are neutral and do not passivate the steel. They protect the steel by creating a barrier. However, the rock bolts include an unprotected free-length and the amount of cover associated with the resin grout within the bonded zone is uncertain. Also, prestressing tends to cause resin grout to crack. One of the goals of the condition assessment is to study the integrity of the grouts with respect to providing a barrier surrounding the reinforcements, and the degree to which Portland cement grout is passivating the steel.

PHASE I CONDITION ASSESSMENT & NDT

NDT

Nondestructive test techniques are used to probe the reinforcements, and the results are analyzed for condition assessment. Four NDT's are employed including measurement of half-cell potential, polarization current, impact and ultrasonic testing. Details of NDT including test procedures are described by NCHRP (2002).

Half-cell potential and polarization measurements are electrochemical tests and the impact, and ultrasonic techniques are mechanical tests involving observations of wave-propagation. In general, these NDT's are useful indicators of the following:

- Half-cell potential tests serve as an indicator of corrosion activity.

- Results from the polarization test are indicative of grout quality and degree of corrosion protection.

- Impact test results are useful to diagnose loss of prestress, assess grout quality and may indicate if the cross section is compromised from corrosion, or from a bend or kink in the bolt.

- Ultrasonic test results are useful for obtaining more detailed information about the condition of reinforcements within the first few feet from the proximal end of the reinforcement.

Results from NDT

Detailed description of the results from the NDT conducted during Phase I can be found in the interim report for the project (Fishman, 2004). Results from Phase I can be generally summarized as follows:

1. Site conditions are moderately corrosive, corresponding to an estimated remaining

service life of approximately fifteen to twenty years due to metal loss from corrosion of the rock reinforcements,
2. Fully grouted rock tendons are apparently in better condition than resin grouted rock bolts,
3. Corrosion is occurring or has occurred along many of the rock bolts,
4. At least 30 percent the rock bolts have suffered loss of prestress,
5. The grouted length of the rock bolts is variable and grout quality is questionable along many of the rock bolts,
6. Some elements may have suffered loss of section of at least 20 percent due to metal loss, which is equivalent to a loss of approximately 0.1 inches in diameter,
7. More problems with loss of section and/or prestress were observed for rock bolts located within an identifiable, lower quality section of the rockmass (Lane at al., 2005) located in the vicinity Station 1775+25, near the andesite dyke.

PHASE II INVASIVE TESTING

Description of Invasive Testing

Phase II includes some reinforcements with questionable condition, and some reinforcements considered to be in good condition, based on the results from NDT. Table 1 is a summary of the reinforcements included in the Phase II test program. Seven rock bolts were selected for lift-off tests and three rock bolts and one tendon element were over cored and sampled. In addition, one rock bolt sample was exhumed as a loose block was fractured with a “boulder buster” and removed from the face. Rock bolt and tendon locations included in the Phase II test program are located near Station 1775+00 and are identified in Figure 4.

Lift-off Testing

Lift-off tests provide a direct measure of the prestress sustained by the anchorages. In this study, they are useful to check the veracity of NDT results, which are an indirect measure of prestress. Lane et al. (2005) describe details of the lift-off tests, which were performed in general accordance with equipment and procedures recommended by PTI (1996).

We also observed loose or slack bearing plates at the anchorage for five rock bolts numbered G-40, G46, G-47, G-52 (#17), and G-54 and shown in Figure 4. Because a slack bearing plate indicates that the anchorage cannot sustain prestress, these observations contribute to five additional direct observations. Thus, Phase II includes twelve direct observations of prestress; seven lift off tests and five slack plates.

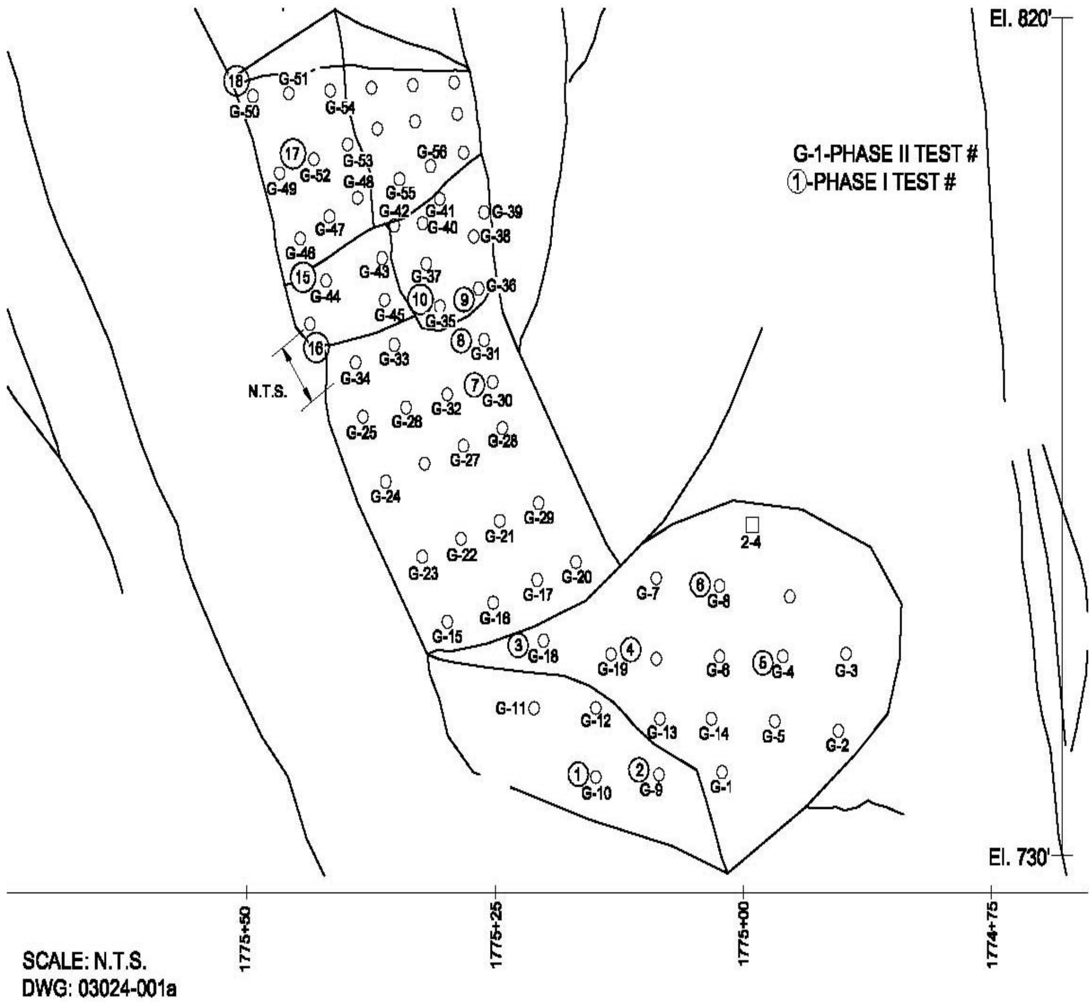

Figure 4. Rock Face Elevation View with Stationing, Reinforcement Locations, and Test Numbers for Phase I & II

<u>Over Coring</u>

Lane et al. (2005) describe details, difficulties, and limitations of over coring and a layout showing the samples retrieved from the site (see Fig.17 in Lane et al., 2005). Thirty-eight feet of rock reinforcements were exhumed from the site including samples from four rock bolts and one tendon element. Approximately, three feet long samples of resin grout and Portland cement grout were also obtained.

Table 1. Reinforcements for Phase II Invasive Testing

Phase I NDT #	Phase II #	Lift-off	Exhumed	Condition Assessment (NDT)	Comments
NA	G1	Y	Y[1]	Questionable	Apparent loss of prestress; relatively poor gout quality; likely corroded
NA	G3	Y	N	NA	NA
3	G18	Y	N	Good	No apparent loss of prestress; relatively good quality grout; likely corroded
4	G19	Y	Y	Good	No apparent loss of prestress; relatively good quality grout; not likely corroded
6	G8	N	Y	Questionable	Apparent loss of prestress; relatively poor quality grout; possible loss of cross section or kink in bolt; very likely corroded
7	G30	Y	N	Questionable	No apparent loss of prestress; relatively poor quality grout; not likely corroded
8	G31	Y	N	Questionable	No apparent loss of prestress; relatively poor grout quality; likely corroded
9	G36	Y	N	Questionable	Apparent loss of prestress; relatively poor quality grout; very likely corroded
16	NA	N	Y[2]	Questionable	No apparent loss of prestress; relatively poor grout quality; very likely corroded.
2-4[3]	NA	NA	Y	Questionable	Relatively good grout condition; likely corroded

[1] Exhumed by NHDOT
[2] Sheared-off as loose block was removed
[3] 2-4 is a tendon element, all others are rock bolts

Observations from Phase II

Observations from Phase II include rock conditions observed during drilling for replacement reinforcements and the conditions of samples retrieved from over coring and "boulder busting". Samples were studied by visual observations, measurement of geometry and laboratory testing as described in the following sections.

Driller's Logs

Replacement bolts were installed prior to invasive testing of the reinforcements. The drilling logs document locations for seams, cracks and weathered rock encountered as drill holes for the replacement bolts were advanced. The propagation of compression waves, observed from impact test results, is affected by changes in condition along the length of the element. Reflections observed in these waveforms may be correlated with data included in the driller's logs and can serves as a basis for explaining interpretation of results from impact testing.

These seams won't be apparent in the results from NDT where they intersect the "free length" of the rock bolts. But seams that intercept the grouted zone can be identified in the results from impact testing.

Condition of Reinforcements

Bolt # 6 (G-8) was retrieved in its entirely for a sample length of approximately fourteen feet. The bolt appears to be in relatively good condition although some pitting corrosion is evident. Grout was observed at intermittent locations beginning four feet from the distal end of the bolt. The resin grout appeared to provide poor coverage to the bar, and for most of the area that had traces of grout, the thickness was not sufficient to cover the bar deformations. The best coverage was observed in an area about 4.25 inches in length, covering one side within the last foot of the bar. This poor coverage probably accounted for the bond breaking and the bolt spinning as the contractor removed the nut and bearing plate prior to overcoring. Bolt 6 was installed at an upwards angle and slid out of the hole after over coring to a depth of approximately eight feet. Bolt #7 (G30) was also loosened as the nut was turned, but this bolt could not be pulled from the hole with 70 kips, and the bolt was not over cored.

An approximately four feet long sample of Bolt #4 (G-19) was retrieved. The sample was terminated as a coupling was encountered within four feet from the rock face. This sample exhibited more corrosion compared to Bolt #6 and loss of cross section was observed at a location near the backside of the bearing plate. A three feet long section of Bolt G-1 was also retrieved and similar loss of cross section was observed near the bearing plate.

Bolt 16 was not over cored, but was recovered as the loose block of rock it supported was fractured and removed with a "boulder buster". A "boulder buster" is a small charge that usually fractures the rock surrounding a rock bolt when detonated, causing the loosened rock to slide toward the base of the rock cut. In this case the rock bolt was severed and removed with the block. The fracture surface at the end of the approximately four feet long sample of bolt 16 appeared to include striations indicative of a shear failure, and the surface did not appear to exhibit a luster that could be attributed to a freshly fractured surface. Therefore, this bolt may have been partially fractured prior to being disturbed by the "boulder buster." Loss of section was also observed near the backside of the anchor plate.

Tendon 2-4 was over cored to a depth of approximately twelve feet. The proximal end of the sample included an approximately two feet long annulus of grout adhered to the reinforcement and surrounding rock core. The steel reinforcement appeared to be in excellent condition and the surface did not appear to have been subject to corrosion. A coupling was encountered at a depth of approximately five feet from the rock face.

<u>Consistency and Physical Properties of Grout Mix</u>

Grout condition is evaluated in terms of the observed coverage of the reinforcement (discussed in the preceding section), and consistency, and physical properties of the grout mix. Consistency is observed via hardness measurements distributed along the Portland cement and resin grout samples. The distribution of results from consistency measurements is considered to reflect the relative quality of the grout mixtures. Physical properties include bulk specific gravity and absorption, which relate to the effectiveness of the grout to act as a moisture barrier and mitigate the intrusion of harmful elements such as chlorides. Bulk specific gravity and absorption were only obtained for the Portland cement grout sample.

Hardness Measurements

Hardness measurements were obtained using a Type D durometer (Shore D scale) in general accordance with the procedure described in ASTM D 2240. The Shore D scale ranges from 0 to 100, and is considered a useful indicator of material type and consistency. A template was used to scribe a 0.5 square inch area at each measurement location. Five measurements were obtained at each measurement location and averaged to yield one data point. One hundred and sixty measurements were obtained along the Portland cement grout sample exhumed with Tendon 2-4. About 100 measurements were obtained from 20 locations, where the coverage was sufficient, along the resin grout sample exhumed with Rock Bolt #6.

Figure 5 compares histograms for the Portland cement and resin grout hardness measurements. Hardness measurements for the Portland cement grout ranged between 84 and 96, with an average of 93 and standard deviation 2.1. Hardness measurements for the polyester resin grout ranged between 83 and 90, with an average of 85 and standard deviation 1.8. The comparison shown in Figure 5 indicates that hardness testing may be a useful technique to identify grout type. Grout hardness measurements are very consistent along both samples. Based on this data, differences between Portland cement or resin grout condition appear to be more in terms of the amount of coverage of the reinforcement elements, rather than with respect to the consistency of the different grout mixtures.

Bulk Specific Gravity and Absorption

The Portland cement grout was removed from the bar following completion of hardness testing. Five samples were selected for bulk specific gravity and absorption testing. Bulk specific gravity was determined by measuring the mass of the specimen in air and also submerged in water. Absorption was determined by comparing dry mass of the specimen to the saturated surface dry condition reached after the specimen was soaked in water for fifteen hours. Measured bulk specific gravity averaged 1.58 (99 pcf) with a range from 1.57 to 1.59 (98 pcf to 99 pcf). The grout mix was proportioned using a water/cement ratio of 0.4 by weight (Haley and Aldrich, 1973). Assuming no air voids in the mix, this renders a theoretical maximum specific gravity of 1.91 (119.6 pcf). The difference between the maximum theoretical specific gravity and the bulk specific gravity may be attributed to the presence of pore spaces in the grout as depicted in

Figure 6. The presence of these pore spaces contributes to a high absorption for the grout mixture. The measured absorption ranged between 36.3% and 33.7 % corresponding to an average of 35.2%.

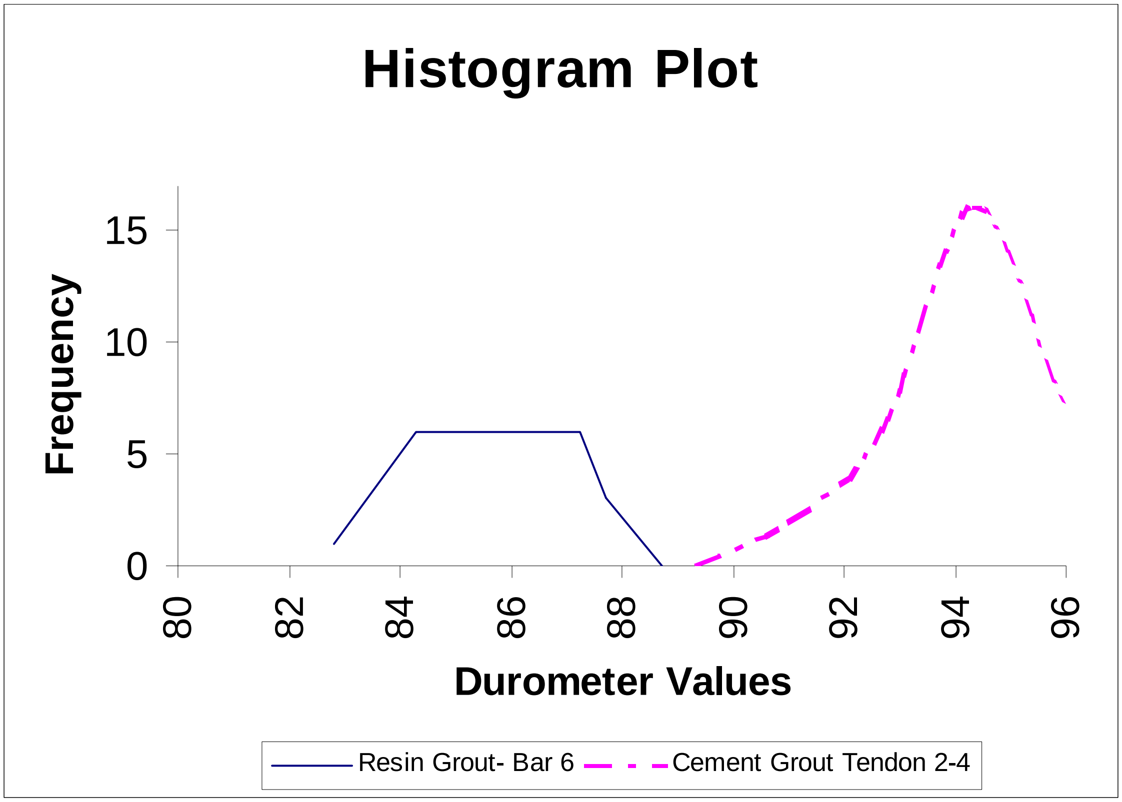

Figure 5. Histogram of Resin Grout and Portland Cement Grout Hardness Measurements.

After the grout was removed, a very slight amount of corrosion was evident on sample Tendon 2-4, within about 2 feet of the rock face. In spite of the apparently high porosity, the grout appears to have protected the steel from significant corrosion to date. The alkaline environment of the grout is apparently sufficient to protect the steel, but some corrosion may be possible due an ample supply of oxygen near the rock face, and the possibility of moisture and chloride intrusion. Chlorides may be present along the rock face as a residue from salt spray, and the possibility of chloride intrusion into the grout should be considered. Portland cement grout samples were submitted for chloride content tests, but results are not yet available.

DISCUSSION OF RESULTS

Corrosion

Examination of exhumed samples tended to verify results from NDT that recognized the occurrence of corrosion. Tendon elements protected by Portland cement grout were in very good condition compared to the resin grouted rock bolts. The free, unprotected, length of the rock bolts could not be accessed with NDT, however, the presence of corrosion along the grouted length was correctly indicated. Pits and craters were observed at a number of locations along the rock bolt samples, and craters appear to coalesce into areas of uniform corrosion extending for lengths of approximately four inches.

Figure 6. Pores Distributed Throughout Portland Cement Grout Sample.

One hundred and seventy-eight pit depth measurements were obtained from the surface of the rock bolt samples. Pit depths were measured with a pit depth gage having a sensitivity of 0.0001 inches. The average measured pit depth was approximately 0.015 inches with a standard deviation of 0.014. Figure 7 shows the cumulative distribution of pit depth measurements indicating that the maximum measured pit depth was 0.1 inches and 10 percent of the measured pit depths were greater than 0.031 inches. We observed that deeper pits are often associated with larger pit diameters, supporting the notion that pitted areas coalesce into areas of uniform corrosion.

Three of the rock bolt specimens exhibited a maximum loss of section corresponding to approximately 0.1 inches in diameter. This loss is consistent with existing mathematical models of service-life and with the observation from NDT that 70% of the rock bolts have experienced significant corrosion. Considering the initial diameter, level of prestress, and rate of metal-loss we estimate that rock bolts will not become overstressed from loss of section due to corrosion for another fifteen to twenty years.

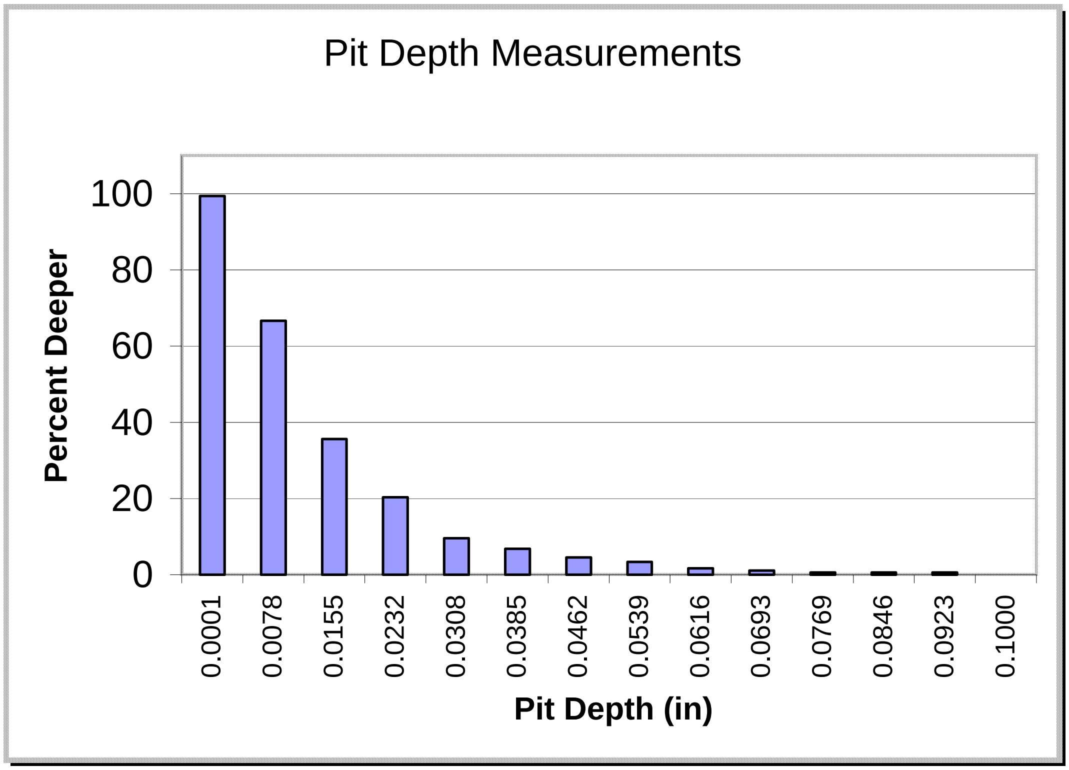

Figure 7. Cumulative Distribution of Pit Depth Measurements

Electrochemical measurements (half-cell potential and polarization resistance) are only able to assess the portion of the element in electrical contact with the surrounding electrolyte (rock mass). Bolt #6 and Tendon 2-4 were the only samples that included grout that could be compared with electrochemical measurements included in the Phase I NDT. The half-cell potential and polarization measurements for Bolt #6 indicate that the element is likely corroded and the grout condition is questionable. The distal end of Bolt #6 did not appear to be completely surrounded with grout and corrosion was evident, which is consistent with the results from NDT. Half-cell potential and polarization measurements for Tendon 2-4 also indicate that grout condition is questionable and that corrosion is likely. The high porosity observed for the exhumed grout sample may confirm the interpretation of grout quality from the results of NDT.

Prestress

Table 2 is a summary of lift off test results and comparison with the interpretation from NDT. Damping, or the rate of decay, of acceleration amplitude response observed from an impact test has been shown to increase with respect to level of prestress for rock bolts (Rodger et al., 1997). Loss of prestress is diagnosed from NDT by comparing the rate of decay observed for the sample population and identifying rock bolts associated with relatively low rates of decay as having an apparent loss of prestress. Thus, NDT results are described qualitatively in terms of “Good” or no apparent loss of prestress, or no good, “NG” corresponding to an apparent loss of prestress.

TABLE 2. LIFT-OFF TEST RESULTS

Bolt #	Lift-Off (Kips)	NDT Result	Correct NDT
3	36	Good	Y
4	38	Good	Y
7	17	Good (?)	N(?)
8	22	Good	N
9	20	NG	Y
G-1	7	NG	Y
6	Loose	NG	Y
17	Loose	G/NG	Y(?)

Reasonable agreement was recognized between results of lift-off tests and NDT. In general, the results indicate that a high percentage of the rock bolts have suffered loss of prestress. The comparison between NDT and lift-off test results is favorable for between 63% (5 of 8) and 88% (7 of 8) of the measurements. Some ambiguity exists with respect to interpretation of NDT results when an intermediate level of prestress remains, and this is apparent in the interpretation of results for Bolt #7. Large losses of prestress, or, at the other extreme, rock bolts with the majority of prestress remaining were correctly identified from the results of NDT.

Nondestructive tests repeated on six rock bolts including Nos. 4,6, 7 and 17, serve as a check on the consistency of NDT results from Phase I. Impact test results performed on bolt Nos. 4,6, 7 and 17 in 2004 and 2005 compare reasonably well, although the possibility that the bolt condition changed during the course of the year is evident for some of the data. The apparent loss of prestress observed for bolts 4,6 and 7 was consistent between readings, however conditions appear to have changed for Bolt #17. In 2003, Bolt #17 did not appear to have suffered loss of prestress, but the readings in 2004 indicate that it has. We also observed that the bearing plate at the anchorage of Bolt #17 was loose, and since this was not observed in 2003, it supports the conclusion that the prestress in Bolt #17 changed between readings.

Specific Reinforcement Conditions

Generally, condition assessment of rock reinforcements does not benefit from analysis of data to identify a specific feature along an element. Rather, the data are compared to one another to identify groups of responses that may be separated into either “good” or “questionable” condition. The interpretation is performed in terms of the character of the observed waveform including the initial rate of decay and the attenuation of the wave reflections. However, for the purpose of describing the measured response, interpretation of data from NDT is compared to physical observation of features observed along the lengths of exhumed reinforcement samples.

Table 3 describes the locations of reflectors observed from the results of impact tests conducted during Phase I. The location of the reflector, L_r, is computed using compression wave velocity, V_p, and observed reflection time, t_r, as:

$$L_r = (V_p \times t_r) / 2 \quad (1)$$

The compression wave velocity of steel is taken as 16,000 ft/sec and 12, 000 ft/sec for Portland cement grout. Reflections were observed from relatively proximal locations denoted as L_1, and from a more distal location, often corresponding to the length of the bolts, denoted as L_2. Direct observations are described in the comments column including loss of section from corrosion, the presence of couplings and rock conditions observed during drilling of replacement bolts as noted on the drillers logs. In most cases L_1 and L_2 are either correlated with direct physical observations, or with the known lengths of the bolts; to an accuracy within approximately three feet, i.e., corresponding to the wavelength inherent to the impact test.

The presence of couplings makes interpretation of reflections from impact testing difficult. Couplings appear to cause reflections in impact test data from Bolts #4 and Tendon 2-4. Although this is useful from the standpoint of verifying the meaning of reflections observed in the test data, this could be misinterpreted as a loss of cross section or other distress in the absence of prior knowledge of the coupling locations.

If rock joints or seams intercept the grout body this may also cause a reflection as evidenced in the data from Bolts #4, 6 and 16. These reflections are likely caused by a change in the geometry of the cross section of the grout body in the vicinity of the joint.

Table 3. Comparison of NDT Results and Direct Observations

Test #	L_1 (ft)	L_2 (ft)	L_T (ft)	Comment
G-1	~	14	15	Loss of cross section near anchor plate; observed soft rock between 9 and 11 feet during drilling for replacement bolt
4	5	15	30	Loss of cross section near anchor plate; coupling approximately 4 ft. from end; observed rock joint at depth of approximately 14 feet during drilling for replacement bolt
6	7	~	15	Poor grout quality; grout not observed until depth of nine feet
16	8	17	20	Loss of section near anchor plate; preexisting fracture approximately 4 ft. from proximal end; rock joint observed at depth of 10 feet during drilling for nearby bolt #17 replacement
2-4	7	~	60	Good grout condition; coupling observed approximately five feet from end

Given the details of the anchor head assembly, neither ultrasonic nor impact test data is useful for identifying loss of section directly behind the anchor plate. This is because the data are masked by a strong reflection from the anchor head location. Thus, although approximately

20% of the cross section from Bolt #4 was consumed by corrosion, this was not evident in the NDT data.

Further evaluation of test data from Bolt #16, as shown in Figures 8 (a) and (b), indicates that a preexisting fracture surface at four feet from the proximal end of the bolts may be evident in the impact and ultrasonic test data. However, these reflections are very subtle and could easily be overlooked without knowledge of the existence of this fracture surface.

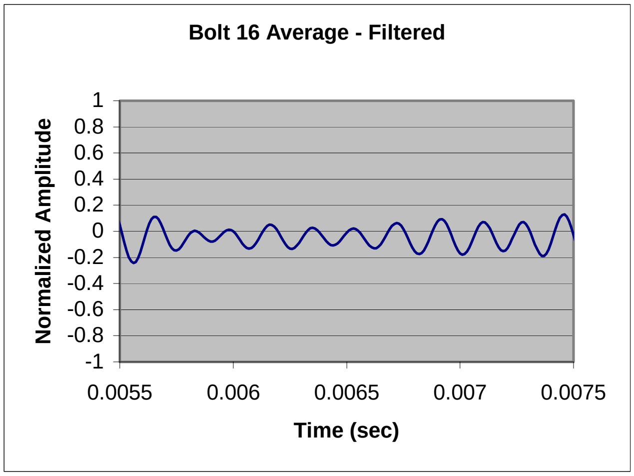

Figure 8(a). Wave from Impact Test on Bolt 16.

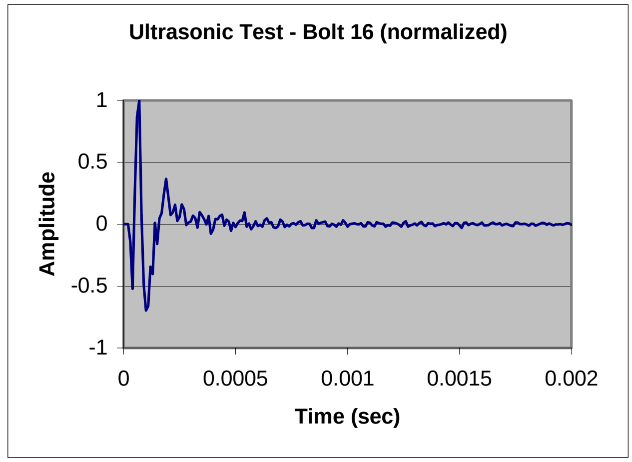

Figure 8(b). Wave from Ultrasonic Test on Bolt 16.

CONCLUSIONS

Reinforcement Condition

Tendons appear to be in better condition compared to rock bolts. Rock bolts have suffered a loss of prestress and some corrosion is evident. Tendons are fully grouted, passive elements and their useful life depends on the durability and integrity of the surrounding grout. The rock bolts are prestressed and are essentially end point anchorages. The useful life of the rock bolts depends on the durability of steel, grout and conditions at the anchorage. Thus, with respect to impacts on service-life, the rock bolts at this site are more vulnerable than the tendon reinforcements.

With respect to the rock bolts, corrosion is present, but the rate of metal loss appears to be close to expectations, and was apparently considered in the original design and corresponding selection of reinforcements and levels of prestress. Tendon elements appear to be passivated by the alkaline conditions provided by the Portland cement grout. Given the high porosity of the grout observed from the samples, chloride intrusion is a concern.

Loss of prestress was observed for four out of six elements examined with lift-off testing. Two of the elements have lost significant amounts of prestress and two others have lost an intermediate amount of tension, but still sustained at least 20 kips. This is consistent with results from NDT that identified at least thirty percent of the elements have lost significant prestress. In one instance, insufficient bond was observed during over coring, however, we do not know the extent to which this contributes to loss of prestress throughout the rock bolt population.

Utility of NDT

Results from NDT serve as useful indicators of overall reinforcement condition at the site. However, specific features along the lengths of the reinforcements are difficult to identify. Detailed knowledge of installation details including the location of couplings and joints, seams and fissures within the rock mass can be helpful for interpretation of results, but in general this information is not readily available. The interpretation of NDT data should be in terms of the character of the waveform obtained from impact testing, which can provide useful indications of stress conditions and grout quality inherent to the reinforcements. Electrochemical tests can also provide useful data relative to the occurrence of corrosion and integrity of corrosion protection. At this time, we strongly recommend that conclusions and assessments made on the basis of results from NDT be verified by more invasive testing.

Benefits of Condition Assessment

Compared to loss of service from corrosion, results from the condition assessment revealed that loss of prestress is the bigger concern relative to remaining service-life. Thus, a sound technical basis is established for planning future maintenance and rehabilitation activities at the site; ultimately resulting in a cost savings to the DOT.

ACKNOWLDGEMENTS

The Research Bureau of the New Hampshire Department of Transportation administered this research under the direction of Mr. Glenn Roberts, P.E. and Contract Nos. 13733L and 14282C. Dick Lane and Marc Fish from NHDOT reviewed the project and provided valuable comments throughout the course of this research. McMahon and Mann Consulting Engineer's, P.C. conducted the research under contract with NHDOT, and JANOD, Inc. was the specialty contractor who performed lift-off testing and over cored the reinforcements during Phase II. Phase II was organized as a pooled-fund study including the New Hampshire, New York and Connecticut Departments of Transportation, and the Federal Highway Administration (TPF-5(096)).

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A Coarse Aggregate Paradox for Indiana Highway Pavements, Less is Better

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Abstract

Coarse aggregates comprise an integral portion of highway pavements, both concrete and bituminous. Acceptance tests for aggregates include the following: Los Angeles Abrasion for strength; and sodium sulfate soundness loss, freeze-thaw loss, specific gravity and absorption as indicators of durability. The presumption is that a lower LA loss and lower sulfate soundness loss, lower freeze-thaw loss, lower absorption and higher specific gravity will yield a higher quality aggregate. This research shows that for certain aggregates in Indiana this adage does not hold true. The paradox applies to the frictional resistance of bituminous surface courses containing carbonate coarse aggregates. Aggregates that pass the minimum state requirements, but are of lower quality based on acceptance tests than are other aggregates, may provide better frictional resistance. It also develops that the acid insoluble residue test including a grain size analysis of the insoluble particles, provides an important evaluation test.

Use of Highway Aggregates

Acceptance tests for highway pavements fall into two basis categories: those evaluating 1) strength and 2) durability. The strength requirement insures that aggregates will maintain their gradation and not produce excessive amounts of fines during the handling process. The Los Angeles Abrasion test is used to evaluate the strength (and abrasion resistance) requirement. Figure 1 shows the coarse aggregate specifications for the Indiana Department of Transportation (INDOT). In reference to the Characteristic Classes, AP relates to concrete pavements and A to hot mix asphalt pavements (HMA). Class C corresponds to base courses. As indicated, a maximum of 40% LA loss prevails for aggregates used in pavements, whereas a maximum of 45% LA loss is allowed for base courses. This presumes that a lower strength material is acceptable for base courses than for pavements.

Aggregate durability, a measure of the ability to resist weathering effects, is measured by sodium sulfate loss, freeze thaw loss, brine freeze thaw loss and to some extent absorption. As observed for the Los Angeles abrasion case, a higher percent maximum loss value is allowed for Class C (base courses) than for Class AP and A. Again a lower requirement for aggregate quality is allowed for the base course material.

Deleterious materials are listed under additional requirements in Table 1. Soft and non-durable particles, plus low density chert cause surface popouts in concrete under freeze thaw conditions. A value of 8% maximum loss allowed for Class C, base courses as compared to AP for concrete pavements (3%) indicates the greater need to limit these weak materials for exposed concrete.

For sodium sulfate loss, note that a maximum of 12% loss is required for concrete and HMA pavements, but 16% loss is applied to base courses. As indicated in Note 2 of Table 1, a 50 cycle freeze thaw test on the unconfined aggregate can be substituted for the sodium sulfate test. Also a brine freeze-thaw test can be performed on the aggregate. Because it is a more destructive test than the 50 cycle freeze-thaw test in water, a greater allowable loss is allowed (30%). By contrast, 40% loss for this test is allowed for base course aggregates, underscoring once more the less stringent requirements for base courses as compared to pavements.

Table 2 provides the requirements for coarse aggregate in concrete as required according to ASTM C33. Standard C33 applies throughout the U.S. as contrasted to INDOT Specifications that are used for Indiana specifically. National standards are typically less stringent than regional standards because they are applied over a larger area showing greater variation. Note that the maximum allowable LA Abrasion loss is 50% for C33 and 40% for pavements based on INDOT Specifications.

The clay lumps, friable particles and low density chert (Sp.G < 2.40) are non-durable materials that deteriorate mechanically under freeze-thaw conditions. The primary problem caused by these weak particles is the occurrence of popouts of the coarse aggregate in exposed concrete surfaces during freeze-thaw conditions. Note that for the INDOT specifications these limitation values are somewhat lower than for ASTM C33.

Aggregate gradations are specified for different highway construction uses. The coarse aggregate gradations for INDOT are provided in Table 3. The gradation required for concrete aggregates is shown as Number 8 in this table. All aggregate pieces must be 1 inch or less in size.

Again for Table 3, the gradation for surface overlays over concrete pavements requires the Number 11 material. As indicated, 100% of the sample must be less than 1/2 inch in size. HMA is used to provide the surface overlays. One of the requirements of the surface is to have a high friction value to reduce the potential for skidding when vehicle brakes are applied. This subject is discussed further in the paper.

In summary, the point can be made that aggregates for pavements are required to have a higher quality according to the standard aggregate tests than do aggregates used in base courses. Also, the greater the exposure to climatic conditions, that is the more severe the weather, the lower the allowable loss based on these aggregate tests. From this it is presumed that aggregates with low values for LA Loss, low sodium sulfate loss, low freeze-thaw loss, low absorption values and a high specific gravity will be the highest quality aggregates for various uses of highway construction. In this paper we will point out that this is not always the case.

Hot-Mix Asphalt Pavements and Overlays

In 1997 INDOT adopted the Superpave mixture design method for hot-mix asphalt pavements (HMA). Prior to Superpave, frictional requirements were based on average daily traffic volumes (ADT) and divided into low (LV), medium (MV) and high (HV) volume categories. Since 1997, Superpave has been based on ESAL values (equivalent single axel loads). Categories include: less than 3 million, 3 to less than 10 million and greater or equal to 10 million ESALs.

Average daily traffic is a count of the number of vehicles that pass over a particular pavement point for a period of 24 hours, averaged over 365 days (Wright, 1995). One equivalent single axel load (ESAL) is equal to one pass of a standard eighteen kip (80 KN) axel. ESAL are converted from ADT and take into account many other factors such as traffic growth, lane distribution, design period, total repetitions per load group, equivalent axel load factor (EALF) per load group, and average number of axels per truck (Wright, 1995). While ADT corresponds to the number of vehicles passing over a pavement, the ESAL value is dependent on both the number of vehicles as well as vehicle weight.

It has long been understood that the coarse aggregate in HMA overlays provide much of the frictional resistance for the surface course. The bituminous binder has a much lower contribution to skid resistance than does the paste and fine aggregate portion of a concrete pavement. Therefore, the contribution of the coarse aggregate for a HMA overlay is crucial.

Crushed stone carbonate aggregates constitute the primary materials in Indiana for both concrete and HMA pavements. Bedrock in the state has a sedimentary origin and sedimentary rocks other than carbonates have a low quality with regard to pavements. Shale, siltstone and weak sandstones do not provide for high quality aggregates for pavements.

Of the two types of carbonates, dolomite is preferred over limestone in surface overlays because of its greater Mohs hardness (4 rather than 3) and typically higher strength. Many limestones lose their frictional resistance (polish) when exposed to vehicular traffic. Therefore, there is a built-in preference in the specifications for the use of dolomite aggregate as bituminous surface overlays.

Table 4 shows the relationship between coarse aggregate type and traffic amount in ESALs for INDOT. The coarse aggregate types listed are air-cooled blast furnace slag (ACBF), steel furnace slag, sandstone, crushed dolomite, polish resistant aggregates, crushed stone (limestone) and gravel. Note that crushed stone (limestone) cannot be used for overlays in the two highest categories of ESALs. Gravel also cannot be used in these categories.

By contrast crushed dolomite can be used for the middle category (<10,000,000) and can be mixed with ACBF or sandstone in a 50/50 % ratio for the >10,000,000 ESAL category. Polish resistant aggregates can also be used in a similar way for these two categories. These are aggregates that have demonstrated through special test procedures to be polish resistant. The sandstone category refers to a well cemented orthoquartzite rock which is available in certain quarries in southern Illinois. It is a massive rock with a low LA Abrasion loss.

Indiana Definition of Dolomite Aggregate

Indiana specifications require a relatively pure dolomite for use as a coarse aggregate for the surface course of pavement overlays. By definition the dolomite must consist of a minimum of 10.3% elemental magnesium. As shown below, this corresponds to an aggregate containing 78.1% of dolomite mineral. A comparison to the calculation based on MgO in dolomite is provided below as well.

a) Mg in Dolomite = $\dfrac{Molecular\,Weight\,of\;Mg}{Molecular\,Weight\,of\;(CaMg)(CO_3)_2} = \dfrac{24.31g}{184.4g} = 0.1318$

$$\text{\% Dolomite} = \frac{\%\,Elemental\,Mg}{0.1318}$$

Therefore, 10.3% elemental magnesium corresponds to 78.1% dolomite, whereas 13.2% corresponds to 100% dolomite and 50% dolomite corresponds to 6.6% Mg.

b) MgO in Dolomite = $\dfrac{Molecular\,Weight\,of\;MgO}{Molecular\,Weight\,of\;(CaMg)(CO_3)_2} = \dfrac{40.32g}{184.4g} = 0.2186$

% Dolomite = $\dfrac{\%\,\mathrm{MgO}}{0.2186}$

Therefore, 10.93% MgO corresponds to 50% dolomite, whereas 21.865 MgO corresponds to 100% dolomite and 78.1% dolomite corresponds to 17.07% MgO.

Laboratory Testing for Frictional Resistance

The British Pendulum Tester (BPT) is the most common method used in the United States to measure frictional resistance of a pavement aggregate in the lab. The BPT measures the coefficient of friction for a given surface and is reported as coefficient of friction multiplied by 100.

The test works by releasing a pendulum with uniform force by gravity from a given height. A rubber slider is attached to the end of the pendulum, which upon release comes in contact with the specimen surface. When the pendulum is released and swings down making contact with the surface, it pushes a pointer up along a calibrated measuring device and leaves it at the highest point reached by the pendulum. The less friction that is encountered by the rubber slider, the higher the pendulum will reach on the calibrated dial resulting in a lower value.

The BPT can be used to take initial, intermediate, or terminal polishing readings. Readings are reported as British polishing numbers (BPN). Initial readings are reported before a specimen undergoes polishing in a polishing machine and are designated with a zero subscript (BPN_0). Values are reported as BPN_0 for initial readings before polishing, and BPN_{10} for terminal readings after ten hours of polishing.

The British polishing wheel is used in conjunction with the BPT and is intended to simulate the polishing effects a pavement or aggregate undergoes in the field by vehicular traffic. Curved coupons for the British polishing wheel are prepared by affixing coarse aggregate with epoxy and later attaching the coupon to the polishing wheel. A smooth, pneumatic tire and the polishing wheel with the attached coupons are rotated in contact while water and carbide grit are fed to the coupon surface. Readings are taken before and after ten hours of polishing.

Discussion

Mineralogy of an aggregate is perhaps the most important factor which influences skid resistance. It is not the minerals alone that provide frictional resistance of a pavement, but it is the differential hardness between minerals that offers sufficient skid resistance. Although harder minerals are preferable, even the hardest rock types will polish if a differential hardness between minerals does not exist. However, a harder mineral will provide a longer rate of polishing, which is desirable. It is the difference in polishing rates between minerals with different hardness that gives aggregates a rough, skid resistant texture.

West and Cho (2000) point out that skid resistance of a pavement is partly dependent on the impurity of the limestone or dolomite aggregate, which can be determined by elemental magnesium content, specific gravity, and total acid insoluble residue. Dolomite is considered more impure with lower elemental magnesium content and higher total insoluble residue, and limestone is more impure with both higher elemental magnesium content and total insoluble residue (West and Cho, 2000). West et al. (2001) point out that this relationship can be seen in the specific gravity of the aggregate. It is suggested that a dolomite aggregate will have a higher frictional resistance when showing specific gravities lower than the 2.8 to 2.9 value (West et al., 2001).

Consequently, for limestone and dolomite aggregates, insoluble residue testing is important. Limestone and dolomite consist mostly of calcium and/or magnesium carbonate, which react with dilute hydrochloric acid. In the insoluble residue test (ASTM D3042), the aggregate is dissolved in dilute hydrochloric acid, and the remaining insoluble portion is usually made up of quartz (silica), feldspar (clays), or other insoluble minerals. The presence of quartz or other insoluble minerals in a limestone or dolomite is significant as it provides aggregates of this rock type with sufficient skid resistance.

In addition, in a recent study by West and Cho (2000), it was observed that a higher percent of insoluble residue smaller than the #200 sieve (<0.075-mm) resulted in an increased frictional resistance. It is speculated that the tiny clay particles that make up the portion smaller than the #200 sieve, break away from the carbonate matrix creating an irregular surface and providing the needed micro-texture for good skid resistance (West and Cho, 2000). West and Cho also report that although the total percentage of acid insolubility correlates well with terminal polish value (BPN_{10}) and wear index (WI), this correlation is stronger with limestone aggregates than for dolomite aggregates. Wear index is the difference between the initial friction value (BPN_0) and the final polish value ($WI = BPN_0 - BPN_{10}$), and is indicative of polish resistance (West et al., 2001).

Concluding Statement

Detailed discussion and analysis of the skid resistance of HMA overlays can be found in the M.S. thesis by the second author (O'Brian, 2004). Only a few conclusions are emphasized here.

For aggregates that pass the INDOT specifications it is not necessarily true that those with the highest quality values will provide the best aggregate for all specific uses. Each use has its own requirements and aggregate tests are not an exact measure of quality for all situations. In this study it has been shown that for frictional resistance of coarse aggregates for HMA surface overlays, an increased level of impurities provides a higher skid resistance because they provide a surface roughness to the aggregates. These impurities tend to cause a higher clay content, lower specific gravity and higher soundness loss.

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Table 1. INDOT Coarse Aggregate Classes.

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

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

Table 2. Limits for Deleterious Substances and Physical Property Requirements of Coarse Aggregate for Concrete.

NOTE 1—See Fig. 1 for the location of the weathering regions and Note 9 for guidance in using the map. The weathering regions are defined as follows:

(S) Severe Weathering Region—A cold climate where concrete is exposed to deicing chemicals or other aggressive agents, or where concrete may become saturated by continued contact with moisture or free water prior to repeated freezing and thawing.

(M) Moderate Weathering Region—A climate where occasional freezing is expected, but where concrete in outdoor service will not be continually exposed to freezing and thawing in the presence of moisture or to deicing chemicals.

(N) Negligible Weathering Region—A climate where concrete is rarely exposed to freezing in the presence of moisture.

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

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

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

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

Table 3. INDOT Coarse Aggregate Gradations.

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

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

Table 4. INDOT Surface Aggregate Specification (INDOT 1999).

Coarse Aggregate Type	Traffic ESAL		
	< 3,000,000	< 10,000,000	≥ 10,000,000
Air-Cooled Blast Furnace Slag	Yes	Yes	Yes
Steel Furnace Slag	Yes	Yes	Yes
Sandstone	Yes	Yes	Yes
Crushed Dolomite	Yes	Yes	Note 1
Polish Resistant Aggregates	Yes	Yes	Note 1
Crushed Stone	Yes	No	No
Gravel	Yes	No	No

Note 1: Polish resistant aggregates or crushed dolomite may be used when blended with ACBF or sandstone but cannot exceed 50% of the coarse aggregate by mass (weight), or cannot exceed 40% of the coarse aggregate by mass (weight) when blended with steel furnace slag.

Specification of Excavated Rock for Embankment Use

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Abstract

Good rock is like pornography. I know it when I see it, but no one seems to agree on a workable definition. This is a continuing dilemma in Pennsylvania road building. While quarried rock is available to meet aggregate and rock lining needs, rock from project excavation is preferred to improve embankment stability and provide sub-drainage. During design, key attributes including rock type, size, gradation, soundness, and durability can be documented for both project excavations and embankments. However, it has been more difficult to develop construction contract provisions that effectively implement design intent.

Over the years, various attempts have been made to control what rock goes where. One approach has been to exclude poor-quality rock from certain uses. Poor-quality rock has been defined by rock type or by rapid deterioration upon excavation. This became somewhat problematic in the field during construction, because of inspection and testing concerns. Another approach has been to accept only the best rock from excavation for certain uses. This best rock has been identified by rock type or has been left as a field decision. Even with constraints established by special provision, 'best available rock' was not enforceable to the extent anticipated during design.

Now both the Pennsylvania Department of Transportation and the Pennsylvania Turnpike Commission specify multiple classifications of rock on some projects. These classifications cover quarry-quality rock; other hard, sound rock on the project; any rock but poor-quality rock; and poor-quality rock. While the specifications used by both agencies are similar, there are differences. The specifications are still evolving to better address the construction issues of field identification, segregation, special handling, multiple handling, measurement, and payment.

INNOVATIVE AGGREGATE RESOURCE EVALUATIONS USING ELECTRICAL RESISTIVITY IMAGING

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ABSTRACT

Aggregate producers have traditionally relied on surface mapping and samples taken from boreholes or test pits to assess the potential aggregate resources on sand and gravel properties. This sampling approach leaves large data gaps which requires interpolation between reliable sources of geologic information. Electrical resistivity imaging (ERI) bridges this data gap and allows for more accurate estimates of aggregate resources to be made.

Electrical resistivity is an intrinsic property of soils that is primarily characteristic of the grain size distribution, moisture content, mineralogy and dissolved solids in the pore water of the soil. ERI is a geophysical technique that measures the apparent electrical resistivity in the subsurface. ERI data acquired in the field along profile lines using an array of electrodes are modelled to produce continuous, true depth geoelectric cross-sections of the subsurface. These cross-sections are then compared to grain size data obtained from strategically located boreholes to determine a correlation between electrical resistivity and aggregate resource material. By acquiring a series of parallel profile lines across a property, a high density of subsurface information can be obtained at the site.

The results of this process are used to identify the most favourable areas for aggregate extraction and produce more accurate resource volume and tonnage estimates for the property. Our experience in Southern Ontario indicates that this method works best in conditions where the moisture content is relatively constant, such as above the water table where the soils are at residual saturation, or below the water table where the soils are fully saturated. The benefit to the aggregate producer is a more accurate estimate of the potential resource than can be made using traditional approaches. Aggregate producers now have the information necessary to develop optimized mining plans and maximize the efficiency of their operations.

INTRODUCTION

In Ontario, aggregate producers face a difficult task in trying to meet the increasing demands of a prosperous economy. Licensing new aggregate properties close to markets is met with more and more opposition, citing environmental issues and impacts on the local community as primary concerns. Since 1992, the Greater Toronto Area (GTA) has run a 3:1 deficit in replacing depleted aggregate supply.[1] The result is a need for aggregate producers to be more thorough in their site selection process and be more efficient at extracting aggregate from their licensed properties.

Aggregate resource assessments are integral to the selection and purchase of a potential aggregate producing property, the licensing process, as well as planning the ongoing extraction at an active sand and gravel pit. Ideally, equipment and infrastructure associated with aggregate production should be placed in areas on the property that are least favourable for extraction so that the producer does not have to shut down production and re-locate the infrastructure to complete the extraction of resources on the property.

Traditionally, aggregate resource assessments are carried out through an extensive drilling and test pitting program. The lateral and vertical extent of the potential resource is then inferred based on relatively sparse point source data, often 100 to 300 metres (300 to 1,000 feet) or more apart. While this method can work in simple geologic settings, it is less reliable in settings where the geology is more complex.

This paper presents an innovative approach to assessing aggregate resources that combines a relatively new geophysical technique (ERI) with high quality borehole sampling methods. The paper utilizes actual data obtained as part a resource evaluation carried out by Golder Associates on behalf of a confidential client (with their permission). Some aspects of the resource evaluation have been altered slightly in an effort to preserve confidentiality, without compromise to the integrity of the methodology.

The site is presented on Figure 1; a typical sand and gravel pit property in Southern Ontario, Canada, where glaciofluvial and outwash deposits from moraines are a common source for aggregate extraction. This site was licensed for aggregate extraction above the water table, with the water table being on the order of 40 metres (130 feet) or more below ground surface.

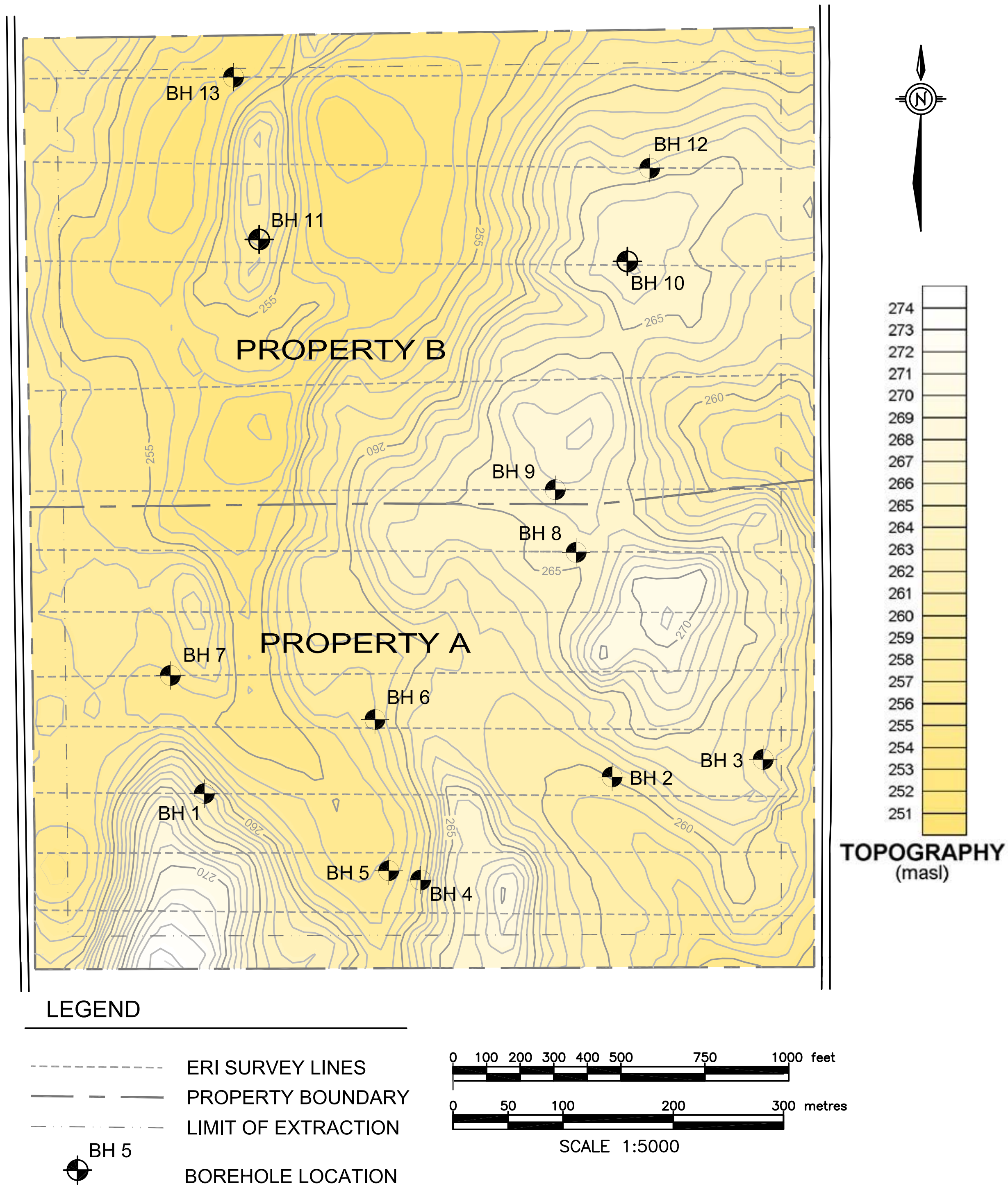
TRADITIONAL RESOURCE ASSESSMENTS

Traditional aggregate resource assessments commonly employ the air hammer (Becker Hammer) drilling technique for obtaining soil samples. It is a relatively rapid drilling technique that can penetrate large gravel and cobbles, however, the samples are highly disturbed and stratigraphy is generally not well preserved. These highly disturbed samples can lead to the misinterpretation of stratigraphic conditions. In addition, an extensive borehole drilling program is generally required in order to obtain a 'representative' volume of data for the property. Test pit data is also used in traditional aggregate resource assessments. While the stratigraphy can be more readily observed in test pits, the method is limited to shallow depths of investigation on the order of 7.5 metres (25 feet).

Estimating the vertical and lateral extent of the potential aggregate resource on the property using this approach requires the assumption that the stratigraphy is consistent and that the resource is continuous between boreholes or test pits. Large gaps between boreholes and test pits can therefore lead to errors in the interpretation of geologic conditions.

While this traditional approach to aggregate resource assessments may work at sites having relatively uniform stratigraphy, it is often not an adequately accurate approach when the sand and gravel deposits are stratigraphically more complex. This turned out to be the case at this

Figure 1
Site Plan and
ERI Survey Line Locations



particular site, and our client asked us to develop an investigative approach that could better assess these complex deposits, where traditional methods employed by a previous consultant proved to be inadequate.

ELECTRICAL RESISTIVITY IMAGING

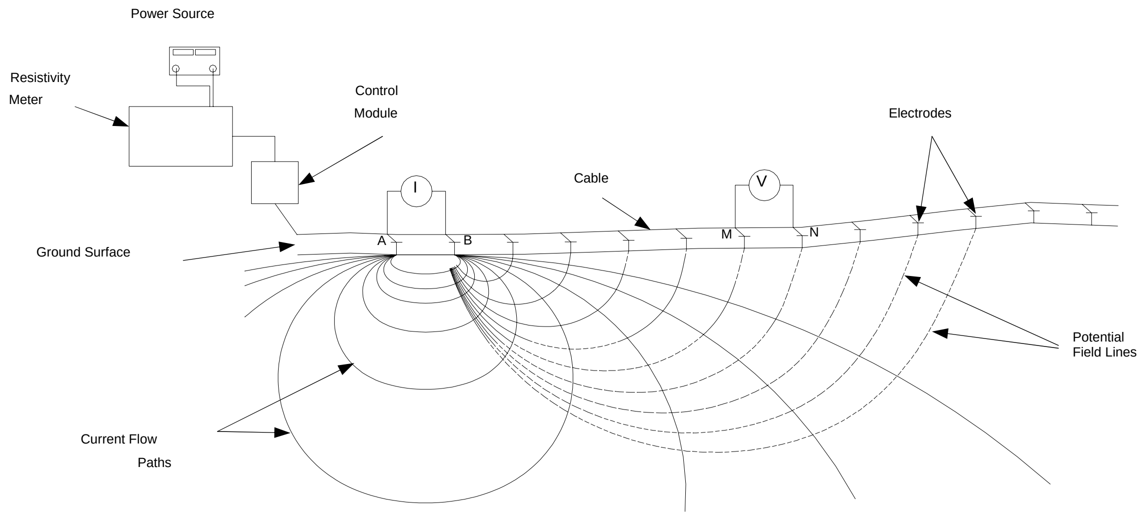
Resistivity methods, both inductive (i.e. electromagnetic) and galvanic (i.e. direct current), can potentially distinguish granular deposits from fine grained deposits.[2] The recent development of commercially available multi-electrode resistivity measurement equipment, as well as robust resistivity inversion software, allows geoscientists to conduct detailed, large-scale resistivity surveys with greater efficiency than ever before. This has given rise to the widespread use of the electrical resistivity imaging (ERI) geophysical method for many different types of subsurface investigations, including the mapping of granular materials.

The ERI method measures the electrical resistivity of the subsurface to infer soil/rock types and stratigraphy. The physical principles for this technique are the same as that established for direct current (DC) resistivity, in which the apparent resistivity of the subsurface is measured for increasing electrode separations by applying a current to the ground using two electrodes and measuring the potential difference (voltage) between two different electrodes. Apparent resistivity of the subsurface is calculated from the potential to current ratio multiplied by a constant, which is a function of the electrode spacing and survey geometry. The depth of investigation is a function of electrode separation, with larger electrode separations providing information from greater depths at the expense of reduced resolution.

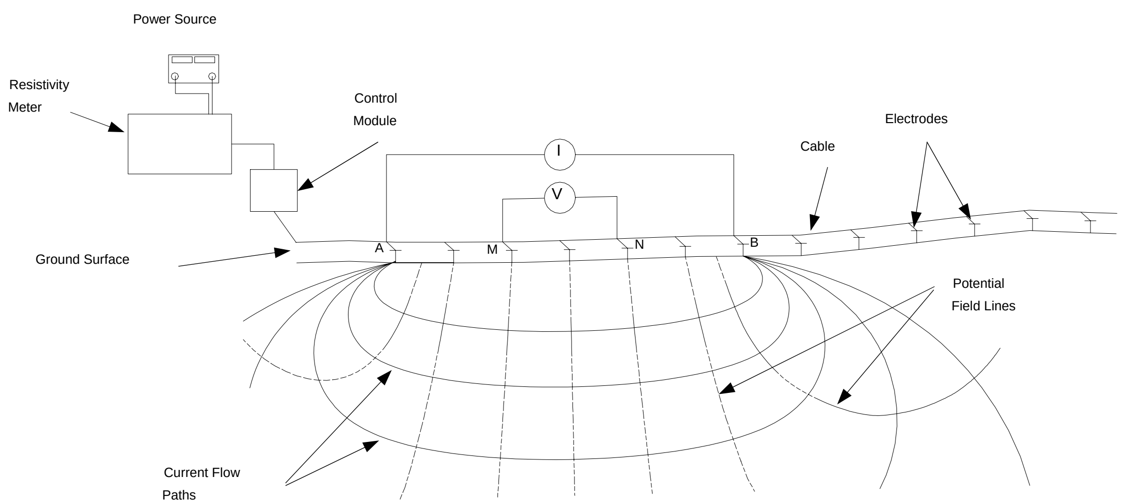
A schematic showing the typical ERI field setup for Wenner and dipole-dipole electrode geometry is presented on Figure 2. We have used both of these electrode geometries in our ERI surveys for resource evaluations with success. The survey results using either geometry are generally very comparable, but not exactly identical. The Wenner configuration typically yields data with a higher signal-to-noise ratio, while the dipole-dipole configuration is typically more effective in imaging abrupt lateral changes in stratigraphy.

ERI differs from the more traditional DC resistivity techniques in that a “spread” of electrodes (typically from 28 to 96) are staked along a survey line and connected to a resistivity meter by a cable fitted with multiple takeouts. The resistivity meter used in the survey is a computer-controlled device consisting of a current supply capable of producing switched +/- constant current and a high impedance voltmeter. This equipment allows for automated collection of high-density data along the entire survey line. A command file is setup in the resistivity meter, which defines the configuration and spacing to be used for each measurement, and controls the acquisition of the data. As data collection continues along the survey line, cables and electrodes from the start of the array are moved (rolled) to the end, reconnected, and the measurement process is repeated down the line using the next command file. These data are then transferred to a computer for processing and interpretation.

Figure 2
ERI Setup – Dipole-Dipole and Wenner Array



"DIPOLE-DIPOLE" ARRAY



"WENNER" ARRAY

The resulting field data can be contoured and plotted as a pseudo-section of apparent resistivity values versus apparent depth beneath the profile line. The field data can also be processed by least-squares inversion to yield a 2D, true depth, geoelectric model of the subsurface, using the computer program RES2DINV.[3]

ERI Data Acquisition

Prior to implementing a full scale ERI investigation, we recommend that a test ERI survey be carried out first in order to verify that the method will yield useful results at a particular site. Using all available information including available surficial mapping, previous borehole logs, test pits, and water well record information, test ERI line(s) are located in strategic areas of the site. Where possible, they are located near boreholes or test pits for comparison. The results from the test survey helps select optimal ERI survey line orientation, electrode configuration and line spacing. Complex geologic deposits may require closer line spacings while larger line spacings can be used for relatively uniform, stratified deposits.

ERI data is typically collected along parallel lines spaced between 50 and 100 metres (150 to 300 feet) apart across the entire property. The ERI lines are typically set up and surveyed using a differential GPS (dGPS) or a total station. In dry environments, it is common to pour a small amount of salt water around the electrodes to reduce the contact resistance between the soil and the electrodes. The depth of investigation is directly related to the electrode spacing and the reading geometry specified in the command file used; depths of investigation on the order of 30 to 45 metres (100 to 150 feet) are common. Assuming an electrode spacing of 5 metres (16 feet) and using a 56 electrode resistivity system, approximately 750 metres (2,500 feet) of ERI line coverage can be obtained by a field crew of 2 or 3 people in a single day.

The elevation survey data is used to correct the ERI data for topography and to help generate a digital topographic model of the property that will be integral to modelling the resource volume at a later stage. The ERI survey line coverage obtained at this particular site is presented on Figure 1. The line spacing was approximately 50 metres (165 feet) on Property A and approximately 100 metres (330 feet) on Property B. At both properties, the electrode spacing used was 5 metres (16.5 feet).

ERI Data Processing

The resistivity data are merged with the topographic data into single data file for inversion using RES2DINV. In RES2DINV, the user first reviews and, if necessary, edits out any bad field data points prior to initiating the inversion process. RES2DINV is very flexible and allows the user to customise all inversion parameters and has several different modes of inversion. User specific inversion parameters can be saved and recalled to facilitate consistent custom processing of data sets. It also has default inversion settings, which in many instances yields reasonable inversion results with little or no modification.

A typical 2D resistivity inversion model for one of the survey lines at this site acquired above the water table is presented on Figure 3. In this model section, the zones of higher resistivity values

Figure 3
ERI Model Resistivity and Borehole Locations

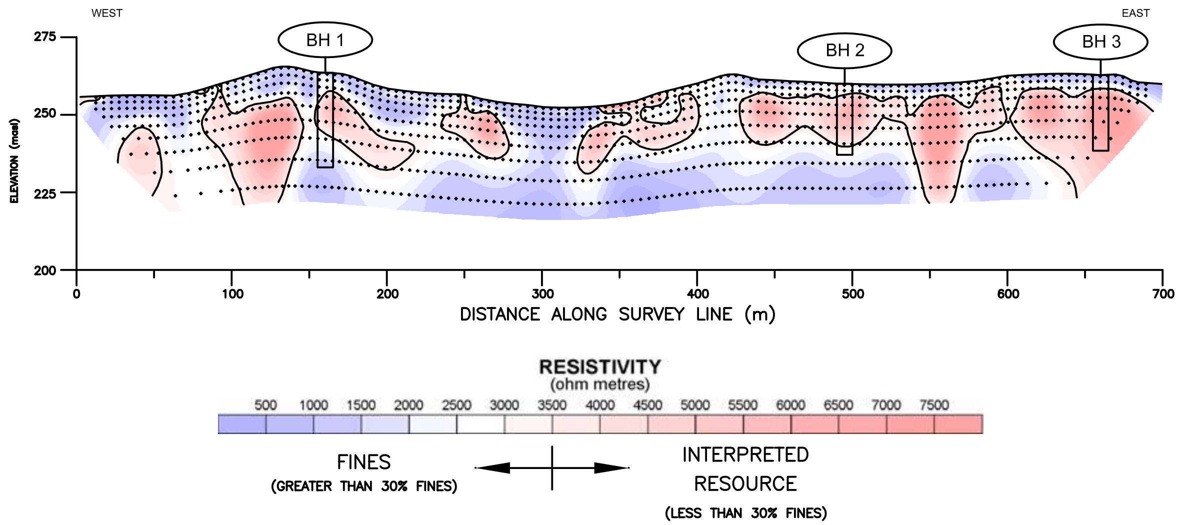
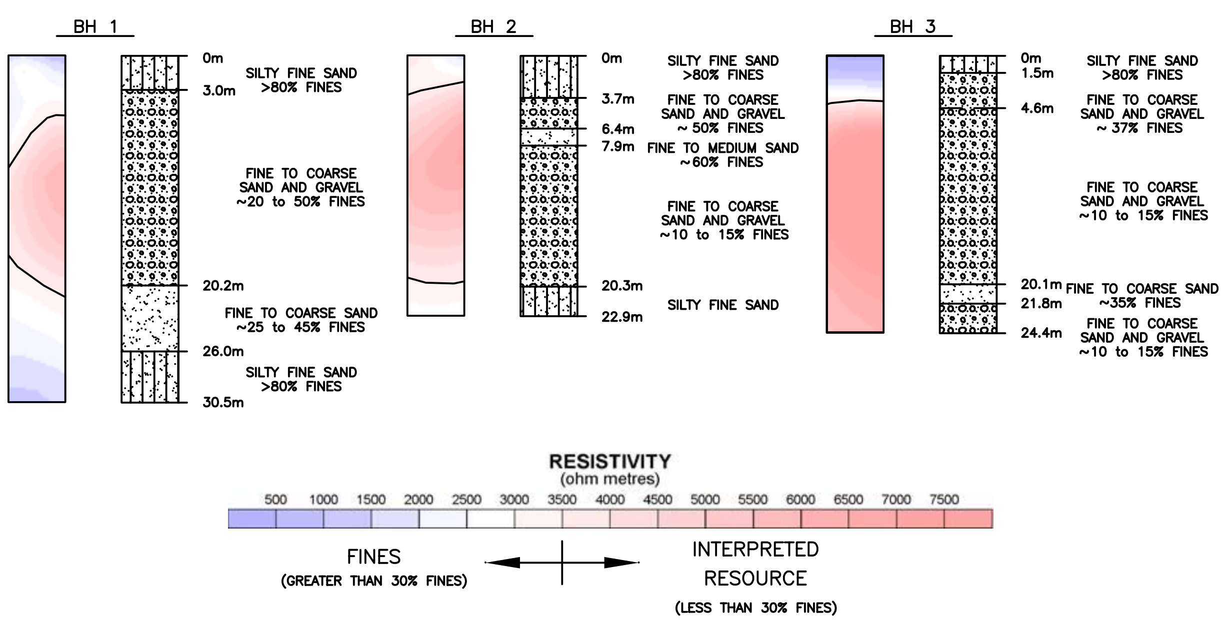


Figure 4
Comparison of ERI Model and Borehole Results



(>3,500 ohm-m) are interpreted to indicate the presence of coarser-grained resource material, while the zones of lower resistivity (<3,500 ohm-m) are interpreted to indicate finer, non-resource material. Determining a specific cut-off resistivity value (3,500 ohm-m) between resource and non-resource material at the site is a key aspect of our methodology and is discussed in a later section of the paper.

CONFIRMATORY DRILLING INVESTIGATION

The ERI field work is followed by a confirmatory drilling investigation with the purpose of obtaining samples from specific features identified on the ERI survey lines. This is an important step in the process of correlating the resistivity results to the presence or absence of potential aggregate resources on the property. In our view, it is important to obtain high quality relatively undisturbed soil samples for the assessment of stratigraphy and grain size analysis. Two drilling methods we find very successful are the CME continuous coring system and Rotasonic drilling. Both methods can yield good quality soil cores in the range of 100 to 150 mm (4 to 6 inches) in diameter. Borehole locations are selected based on their relationship to resistivity features in the ERI models. Both high resistivity and low resistivity features are drilled to obtain samples from an unbiased range of materials. Samples representative of a range of coarse and fine grained material are submitted for laboratory grain size analyses.

Figure 4 presents the stratigraphy and grain size results from drilling compared with ERI model results. At this site, drilling and grain size testing indicated that the high resistivity zones in the ERI model correlated well to the presence of potential aggregate resource as found in the boreholes.

CORRELATING EARTH RESISTIVITY TO GRAIN SIZE DISTRIBUTION

Physical Property Considerations

Using resistivity as part of a resource evaluation requires that a relationship between earth resistivity and grain size distribution can be established for the site. There are a number of physical properties that affect the earth resistivity of soil material at a fine scale, namely: porosity, saturation, pore water resistivity and particle resistivity. Porosity is the volumetric fraction of pore space in the material. Saturation is the fraction of that pore space that is filled with fluid. The pore water resistivity is a measure of how the fluid conducts electricity and is mainly a function of dissolved solids and ions in the water. The particle resistivity is a measure of how the particles conduct electricity and is mainly a function of particle mineralogy. Grain size and grain size distribution are also important factors, as they can affect porosity and saturation.

In natural systems, such as the subsurface soils comprising glacial and or fluvial deposits, the properties of a particular soil material combine in a complex way to yield the earth resistivity that we can measure directly in the lab on a core sample or infer from the modelling of ERI data we measure at surface in the field. As a result, the earth resistivity of soils can commonly vary by several orders of magnitude and in extreme cases, many orders of magnitude.

With all of these physical properties potentially contributing as variables affecting earth resistivity, under what subsurface conditions can resistivity be used as a relatively direct indication of grain size distribution? We have identified two environments where subsurface conditions are suitable for the assessment of potential aggregate deposits using resistivity, and fortunately, these particular subsurface conditions are commonly encountered at aggregate properties in Southern Ontario. These conditions can basically be summarised as deposits above the water table, and deposits below the water table.

In both of these situations in Southern Ontario, the pore water is relatively fresh so the pore water resistivity is relatively high and constant throughout the site. The soil particles themselves are comprised either of sand and gravel derived from limestone, dolostone and crystalline rocks that have a high electrical resistivity or mineralogical clay that has a low resistivity. Therefore, in these circumstances, pore water resistivity is relatively constant and particle resistivity is a function of clay content.

Aggregate Deposits Above the Water Table

In the case of aggregate deposits above the water table, where the soils are at residual saturation, the main factors that affect resistivity are the residual water content and how well the pore water is interconnected. Under residual saturation conditions, fine grained materials and in particular clay, will have a higher water content that is better interconnected than in the coarse grained material. Therefore, coarse grained material with lower clay content has a relatively higher resistivity than fine grained material in these conditions. In this environment, areas of relatively high earth resistivity are an indication of the presence of relatively coarse grained material with a good potential for aggregate production.

Aggregate Deposits Below the Water Table

In the case of aggregate deposits below the water table, where the soil is fully saturated, the main factors that affect resistivity are porosity and clay content. Because coarse grained material has a lower porosity and clay content than fine grained material, coarse grained material will have a relatively higher resistivity than fine grained material in these conditions. In this environment, areas of relatively high earth resistivity are again an indication of the presence of relatively coarse grained material with a good potential for aggregate production, although the resistivities below the water table are several orders of magnitude lower than those measured above the water table.

Establishing the Correlation

With this basic understand of these natural systems in mind, we can determine if there is a relationship between resistivity and aggregate potential, and what that relationship is, by comparing the ERI model resistivity values to the grain size distribution of samples taken from cores at boreholes along the ERI survey line. We typically establish this relationship by graphing % of fines measured by grain size testing of soil samples taken from cores along an ERI profile line to the model resistivity corresponding to that location on the ERI model section.

Examination of the distribution of points on this graph, as presented on Figure 5 for this site, indicates that zones where the model resistivity is higher than 3,500 ohm-m correlate reasonably well to areas where the % fines content (fine sand or smaller sized particles) is lowest, less than 30% of the total particle distribution in the sample.

Therefore, in making an interpretation of areas most favourable for extraction of aggregates on this site, we identified zones on the ERI cross sections where the model resistivity was higher than 3,500 ohm-m, effectively using this as a cut-off threshold between what we infer to be resource and non-resource material in the subsurface.

Limitations

Even in the two types of subsurface environments where we have applied this method successfully, there are some notable limitations, the most significant of which are as follows.

- The grain size distribution has to be relatively unimodal. If there is a bimodal grain size distribution, such as might be the case for clayey gravel till, the resistivity tends to be dominated by the fine mode (i.e. the clay) and the gravel can go undetected.

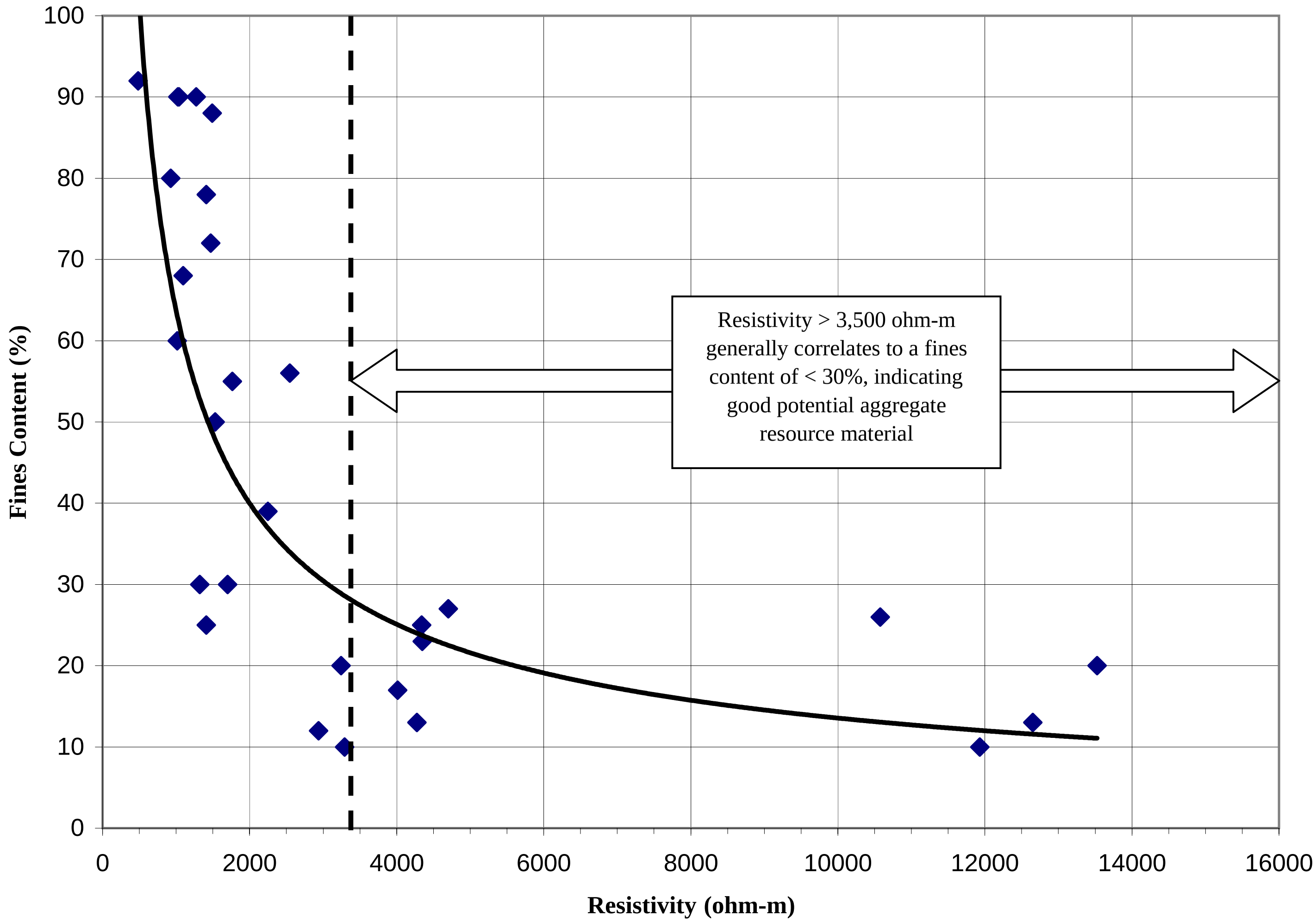
- Perched water tables are problematic. If there are perched water table conditions, low resistivity zones in perched areas above the water table may in fact be coarse grained, but because the material is not at residual saturation, the zone will have a low resistivity and the coarse grained material can go undetected.

- Small pockets and thin layers are hard to detect with ERI. The resistivity method measures an average resistivity over a volume, and the size of that volume increases with the depth below surface. As a result of this, there is a limit to how small a pocket of material and how thin a layer of material can be detected as the depth below surface increases.

- Fine sand can yield a deceptive resistivity response. Well-sorted fine sand that is essentially devoid of silt and clay can have a resistivity response that is quite similar to medium and coarse sand, making these materials difficult to distinguish from one another. In many cases, fine sand may be of little interest to an aggregate producer and may be considered a non-resource material, whereas medium to coarse sand is considered a resource.

Each set of site circumstances are unique and must be considered in developing a suitable investigation approach. The ERI method must be used as one part of an overall systematic approach to a resource evaluation, and should be implemented by geoscientists with experience and an understanding of the method and its limitations.

Figure 5
Correlation of ERI Model Resistivity to Percent Fines Content in Core Samples



INTERPRETING AND MODELLING POTENTIAL AGGREGATE RESOURCES

Interpretation of Potential Resource

Using the 3,500 ohm-m contour as a guideline, areas where potential aggregate resource is present in the subsurface are identified along each of the ERI model sections acquired at the site. In the case of the ERI model section presented on Figure 3, the resource does not appear to be continuous along the survey line. Within each of the areas where potential resource is inferred to be present, the top of resource and base of resource is interpreted from the ERI model and confirmed by drilling, where available.

In practice, the base of the resource used in volume and tonnage calculations may be limited by other factors than the actual lower limit of the deposit. For example, it may be limited by the lower extraction limit allowed on the property’s permit, or a practical lower limit in circumstances where operational side slopes need to be maintained at the edges of the property. In some cases, the producer will want aggregate volume and tonnage estimates for several scenarios such as vertical side slopes (i.e. theoretically available volume and tonnage) or 4:1 side slopes (i.e. practically available volume and tonnage allowing for 3:1 side slopes and benches).

Modelling the Resource

The resource limits identified by this process on the ERI sections are then translated from 2D into 3D, real world coordinates. The ground surface topography, lateral limits of the resource, top of resource, and bottom of resource, together form a 3D data set that can then be used to construct a 3D model of the potential resource available on the site.

Using these 3D data sets as input data, a series of 3D surfaces are created by gridding using the computer software Oasis montaj. This software allows us to generate contour and isopach maps, compute volumes between surfaces, perform other mathematical operations on the 3D surfaces (such as intersections) and trim 3D surfaces to 2D (plan) limits.

Modelling the potential resource on the property is carried out with the following objectives in mind.

- Identify where the aggregate potential resources are located on the property.
- Estimate the total volume and tonnage of potential resource on the property (both theoretical and practical).
- Estimate the volume and tonnage of overburden will need to be stripped in order to access the resource.
- Identify the most favourable areas for aggregate extraction.
- Identify the best place to locate a wash plant and other infrastructure so as to not “sterilize” potential resources.

Volume and Tonnage Estimates

Identifying the areas of potential resource is based on the lateral extent of the potential resources as interpreted on the ERI model sections, once these data have been translated from section to plan view. The volume of potential resource is estimated from the resource model by calculating the volume between the top of resource and bottom of resource layers, within the lateral limits identified. Tonnage of potential resource is estimated from the volume, assuming a “bulking” or density factor, which is usually between 1.65 and 1.85 tonnes / m^3. The volume and tonnage of overburden to be stripped is estimated in a similar manner, however, the volume between the ground surface and the top of resource layers are used in the calculation.

Favourable Extraction Areas

To identify the most favourable areas for extraction, a resource to overburden thickness ratio is calculated. This is done by first calculating a resource isopach (i.e. resource thickness) and an overburden isopach (i.e. overburden thickness). The resource isopach is then divided by the overburden isopach, and contoured. The resulting contour map is essentially an “extraction ratio” map. The areas of favourable extraction are identified on Figure 6; typically areas where the resource to overburden extraction ratio greater than 2:1. These areas are most economical for aggregate extraction, as the cost of stripping the overburden is relatively low, and the value (i.e. tonnage) of the aggregates that can be extracted is relatively high.

The producer now has valuable information about location of economically mineable resource within the property that takes into account the interpreted resource and non-resource thicknesses, the offsets from property and environmental boundaries, and required side slopes. The increased level of detail provided in this innovative approach to aggregate resource assessments is critical in allowing the producer to develop an efficient mining plan.

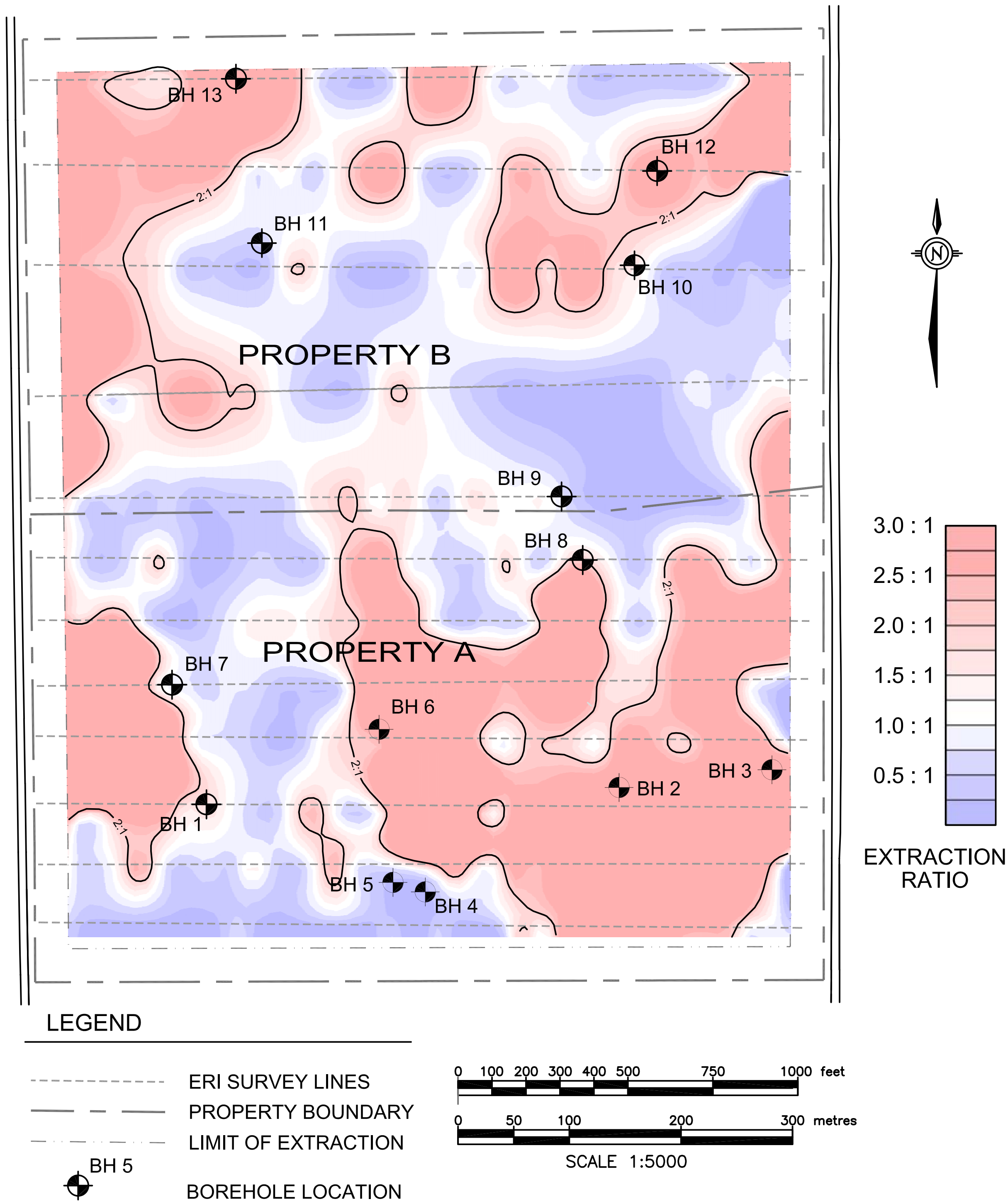
Developing Mining Plan

With this information at hand, the producer can confidently develop a mining plan that will maximize the efficiency of the mining operation. Everything from planning the phases of extraction to selecting optimum locations for infrastructure can be incorporated into the mining plan. The producer will know how to best plan the phases of resource extraction. They can locate their infrastructure in areas that are proximal to the extraction, minimizing the potential to “sterilize” available resources. Greater consideration for the natural environment is assured by limiting the amount of stripping of non-resource material in search of mineable resource.

CONCLUSIONS

Aggregate producers are facing increased pressure to maximize the resource extracted at their properties and make their operations more cost effective. Traditional aggregate resource estimates that involve extensive drilling and test pit programs are costly and are not well suited in moderate to complex geologic environments. Errors in interpreting the resource at this stage

Figure 6
Model Results Showing
Areas of Favorable (2:1) Extraction



can lead aggregate producers to purchase less desirable properties and develop less effective mining plans.

Geologic conditions in the subsurface can be interpreted with higher confidence between boreholes and test pits by utilizing the ERI geophysical method as part of the resource evaluation. In our experience, there is a repeatable correlation between the presence of potential granular aggregate resources and electrical resistivity response. By understanding this correlation and its limitations, we are able to model the potential aggregate resources at a level of detail that would be very difficult to match using more traditional approaches.

Our innovative aggregate resource assessment approach has been applied successfully at a number of aggregate properties in Southern Ontario. Confirmatory drilling at these properties has shown that ERI is a useful tool for delineating potential resource at these properties and is now an integral component of our aggregate resource assessments. Aggregate producers now have the tools they need to help make informed decisions on purchasing potential new properties and maximizing the efficiency of their operations at existing properties.

ACKNOWLEDGMENTS

We would like to acknowledge our confidential client for their vision and support in helping Golder Associates develop this innovative approach to aggregate resource assessments and appreciate their consent in presenting excerpts of data from one of their sites in this paper.

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Characterization of South Carolina Aggregates Using Micro-Deval Abrasion Test

Prasada Rao Rangaraju[1] and Jonathan Edlinski[2]

ABSTRACT

The objective of this study was to characterize the toughness and durability of 23 different aggregate sources in South Carolina using the micro-Deval abrasion resistance test and explore any correlations with results of the traditional LA abrasion test and sodium and magnesium sulfate soundness tests. In addition, results from all the tests were correlated with observed field performance. Also, the effect of aggregate gradation on the losses obtained in micro-Deval and LA abrasion tests, the rate of aggregate degradation in the micro-Deval test, and the influence of build-up of degraded material in the micro-Deval jar on the total loss observed were studied.

Based upon the results of this study, the loss observed in micro-Deval test showed a better correlation with the field performance of aggregate, compared to other test methods evaluated in this study. A maximum acceptable micro-Deval loss of 17% was found to be satisfactory to distinguish "good" aggregates from "poor" or "fair" aggregates. Evaluation of aggregate in sodium and magnesium sulfate soundness tests indicated a good correlation between the losses observed in the test methods. However, neither of the soundness test results correlated well with either the micro-Deval loss or the observed field performance.

The losses obtained with different gradations in the micro-Deval test correlated well with each other. However, finer gradations from a given aggregate source typically yielded higher losses in the micro-Deval test compared to coarser gradations. No such influence of aggregate gradations on the loss obtained was observed in LA abrasion test. Investigation into the influence of build-up of degraded material in the micro-Deval jar on the observed loss was inconclusive.

INTRODUCTION

Traditionally, the South Carolina Department of Transportation (SCDOT) has used the Los Angeles (LA) abrasion and impact test (AASHTO T-96) to measure the degradation resistance of coarse aggregates, and sodium sulfate test (AASHTO T- 104) to determine the long-term durability/soundness of aggregates. Depending on specific application, the maximum acceptable loss in the LA abrasion and impact test ranges between 45% and 60%. For HMA surface course, the LA loss limit is specified at 55%. This test has been criticized for lack of its correlation with field performance [1-3]. The reasons include:

(i) Generally, the moisture content of aggregates in field is closer to being saturated than oven-dry condition and most aggregates tend to be weaker and softer when wet. However, the LA test is conducted on an oven-dry aggregate, which is not representative of the field conditions.

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(ii) Generally, aggregates are subjected to more abrasive loads than impact loads. However, the LA test predominantly reflects the impact resistance of aggregates rather than abrasion resistance.

Also, with the advent of the Superpave HMA mix design system and the associated specifications for aggregates, many aggregates in South Carolina that were once considered acceptable for HMA, are now considered unacceptable due to "poor" LA test results. Because many of these aggregate sources had been used successfully over the years, this raised concerns in the aggregate and HMA industries as to whether the LA test is truly related to the performance of an HMA pavement. Similar concerns were raised about the use of sodium sulfate soundness test to predict the long-term durability/soundness of aggregates.

These concerns have prompted the SCDOT to investigate alternative degradation and abrasion test for coarse aggregate that may be more related to field performance of HMA and PCC pavements. One test that has received much attention is the micro-Deval abrasion test. The micro-Deval test is described as the "wet ball-mill test" in which graded aggregate samples are placed in a stainless steel jar with water and small steel spheres. The jar is rotated for a specified amount of time and the aggregate is then evaluated for material loss. Developed in France in the early 1960's, the micro-Deval test has been studied extensively in Canada, and more recently in the United States [3-7].

The micro-Deval test was one of the several tests that were evaluated in the National Cooperative Highway Research Program (NCHRP) study 4-19 to predict the field performance of aggregates. One of the principal recommendations of the NCHRP study was that the micro-Deval test was a better indicator of field performance than the LA abrasion and impact test [3]. The NCHRP study indicated a very strong correlation between field performance of various aggregates in HMA pavements and the micro-Deval abrasion value for these aggregates. Based on this study, a micro-Deval abrasion loss of 18% or less was found adequate for delineating aggregates that historically demonstrated either "good", "fair" or "poor" field performances. No such correlation could be found for the LA Abrasion test. More recent studies by Cooley and James at the National Center for Asphalt Technology (NCAT) on micro-Deval testing of aggregates from southeastern states had reiterated the findings from the NCHRP study [4]. The NCHRP investigation also explored correlations between various soundness/durability test results, micro-Deval test results and aggregate performance histories. Based on that study, a strong correlation was observed between micro-Deval test results, soundness/durability test results, and performance histories [3]. In particular, the magnesium sulfate soundness test was found to be a very good predictor of field performance for durability. Similar findings were observed in TxDOT study [6].

Due to its dissatisfaction with the poor precision and correlation with field performance of tests such as the LA abrasion and sodium sulfate soundness, the Ontario Ministry of Transportation had performed several studies on test methods for determining aggregate toughness/abrasion resistance and durability [1, 2]. These studies have indicated that for PCC, HMA pavements as well as granular bases, micro- Deval test served as a better indicator of aggregate quality than other degradation test. In contrast, studies by Oregon Department of Transportation have indicated that micro-Deval test procedure did not characterize aggregate any better than LA abrasion and impact test for evaluating the resistance of aggregate to studded tires [7]. For predicting the long-term durability/soundness of aggregates, the NCHRP study along with other studies found that magnesium sulfate soundness test provided a better characterization of durability of aggregates than sodium sulfate test [3,6].

Based on the concerns with the tests normally used to predict aggregate durability in South Carolina, a necessity to evaluate the micro-Deval abrasion test had developed. In addition, influence of aggregate gradation on the observed loss in micro-Deval test and the effect of accumulation of degraded material on the observed loss in the micro-Deval test were not evident in the literature. This paper presents the results of the study conducted to address these issues.

SCOPE AND EXPERIMENTAL PROGRAM

Aggregates from 23 different sources that are approved for use on SCDOT projects were selected for this study. Of the 23 sources, 20 sources were local to South Carolina; two sources were from North Carolina and one from Georgia. Table 1 presents a summary of all the aggregates used in this study along with a brief description of rock type.

Aggregates from each of the sources were tested to determine percent loss for the micro-Deval test, LA abrasion and impact test, sodium sulfate and magnesium sulfate soundness tests. In addition, micro-Deval and LA abrasion and impact tests were conducted on three different gradations of aggregates from each source to determine the influence of aggregate size on loss. Also, a series of modified micro-Deval tests were conducted on selected aggregates, in order to investigate the rate of aggregate degradation that occurs in the test. In addition, the influence of the accumulation of degraded material in the micro-Deval jar on the observed loss at the end of test was also explored using three different aggregates.

TEST PROCEDURES

Micro-Deval Abrasion Test

The Micro-Deval abrasion tests were conducted on all aggregates according to AASHTO TP 58-00 procedure. In this method, 1500 grams of an aggregate sample is prepared by washing and soaking in water for one hour prior to the test. The prepared aggregate sample is then placed in a stainless steel jar along with 2 liters of water and 5000 g of 9.5 mm-diameter steel balls. The sealed jar is loaded on to a ball-mill roller and rotated at 100 ± 5 rpm for a period of 120 minutes, 105 minutes or 95 minutes depending on the gradation of the aggregate being tested. The loss is determined by sieving the aggregate sample on a 1.18 mm sieve (#16) and expressing the mass of the material passing through as a percentage of the original sample mass.

In order to determine the influence of aggregate gradation on the loss observed in this test procedure, micro-Deval tests were conducted on three different gradations for each of the aggregates tested. The three gradations are recognized in the AASHTO TP 58 procedure under sections 7.2, 7.3 and 7.4. These gradations are identified in this paper as “MD-A”, “MD-B” and “MD-C”, respectively. Table 2 shows the distribution of the aggregate sizes for each gradation.

TABLE 1 Results of LA Abrasion and Impact Tests, Micro-Deval and Sulfate Soundness Tests

Aggregate ID	Rock Type	LA Abrasion Loss (%)			Micro-Deval Loss (%)			Average Magnesium Sulfate Loss, %	Average Sodium Sulfate Loss, %	Field Performance†
		Average LA-A *	Average LA-B	Average LA-C	Average MD-A *	Average MD-B	Average MD-C			
SC-1	ML	-	44.8	50.4	31.7	31.3	34.9	12.3	2.2	poor
SC-2	Gr	52	53.4	54.4	10.8	17.0	19.3	8.3	7.6	fair
SC-3	Gr	38	40.0	41.8	5.8	8.6	10.3	2.2	1.7	good
SC-4	Gr	34	30.7	31.9	4.3	6.4	7.4	0.8	0.5	good
SC-5	Gr	52	46.5	44.3	7.0	11.0	12.4	1.8	1.6	good
SC-6	Gr	20	17.7	18.9	4.5	4.8	6.7	3.2	3.0	good
SC-7	Gr	24	25.2	25.0	4.0	9.5	10.5	4.1	3.0	good
SC-8	ML	-	32.1	33.5	21.7	23.6	22.6	18.9	16.2	poor
SC-9	Gr-Gn	16	16.5	17.1	14.4	18.9	17.3	3.8	2.0	fair
SC-10	Gr	35	37.8	41.0	6.2	9.5	9.6	1.3	1.3	good
SC-11	Gr	20	18.5	21.2	9.0	10.0	9.7	1.1	0.8	good
SC-12	Gr	54	55.1	56.6	22.8	31.3	37.3	2.8	2.6	fair
SC-13	M-Sch	39	33.7	29.1	19.2	18.7	18.0	11.9	11.4	fair
SC-14	Gr	40	37.4	42.9	5.9	9.5	11.3	3.5	3.2	good
SC-15	Gr	29	26.3	25.1	4.0	6.0	6.4	4.0	3.8	good
SC-16	Gr	49	50.2	47.8	9.2	15.0	15.0	1.6	1.4	good
SC-17	Gr	54	53.9	55.4	9.5	16.3	17.4	1.2	1.1	fair
SC-18	Gr	53	53.1	49.5	8.5	15.0	17.3	0.8	0.8	good
SC-19	Gr	52	52.2	52.9	8.6	12.3	12.8	5.5	4.8	good
SC-20	Gr	47	50.0	55.5	11.3	22.1	26.4	2.7	2.5	good
SC-21	Gr	33	31.0	32.9	6.1	11.9	12.7	1.4	1.1	good
SC-22	Gr	52	51.2	47.8	7.9	12.4	13.6	1.1	0.8	good
SC-23	Gr	30	30.8	32.4	10.5	9.8	11.3	3.6	3.1	good

* Values provided by SCDOT; † Rating provided by SCDOT based on field performance. Each of the values is an average result of three tests

TABLE 2 Gradation of Aggregates Used in Micro-Deval and LA Abrasion and Impact Tests

Gradation of Aggregates in Micro-Deval Abrasion Test				
Passing Sieve Size, mm	Retained Sieve Size, mm	MD-A* Mass, g	MD-B* Mass, g	MD-C* Mass, g
19.0	16.0	375	-	-
16.0	12.5	375	-	-
12.5	9.5	750	750	-
9.5	6.7	-	375	750
6.7	4.75	-	375	750
Gradation of Aggregates in LA Abrasion and Impact Test				
Passing Sieve Size, mm	Retained Sieve Size, mm	LA-A† Mass, g	LA-B† Mass, g	LA-C† Mass, g
37.5	25	1250	-	-
25	19	1250	-	-
19	12.5	1250	2500	-
12.5	9.5	1250	2500	-
9.5	6.3	-	-	2500
4.75	2.36	-	-	2500

* MD-A, MD-B, and MD-C gradations correspond to requirements in sections 7.2, 7.3 and 7.4 of AASHTO TP 58-02

† LA-A, LA-B, and LA-C gradations correspond to requirements in Table 1 of AASHTO T 96

Throughout the experimental study, the micro-Deval testing equipment was checked for accuracy using the Brecchin aggregate from Ontario as control aggregate. The loss observed for the Brecchin control aggregate ranged between 16.5 % and 17.9%, which was acceptable as per MTO specifications.

Studies on Rate of Aggregate Degradation in Micro-Deval Test In order to measure the rate of aggregate degradation that occurs during the micro-Deval test regime, two slightly modified techniques were applied to the standard micro-Deval test procedure. For this purpose, aggregates satisfying the MD-C gradation were selected.

The first technique implemented multiple tests on multiple samples of aggregate. In this technique, a series of micro-Deval tests were conducted on multiple samples, wherein, each test was stopped after a specified number of total revolutions. In this series, the total number of revolutions at which each of the tests was terminated and the loss measured ranged from 1500 to 9500, in increments of 1500 revolutions.

The second technique used a single aggregate sample throughout the testing cycle and the test cycle was stopped at regular intervals of 1500 revolutions of the jar, so that mass loss could be measured. Testing then resumed on the same sample once the mass loss measurements were taken. Since this method expelled degraded aggregate each time mass loss was measured, it then demonstrated how mass loss occurs when degraded material is not allowed to accumulate. This information is valuable to determine if the accumulation of degraded material in the micro-Deval jar interferes with the loss observed in the test, particularly in case of marginal to poor aggregates.

Comparison of results from both procedures yielded valuable information to study the rate of aggregate degradation in the micro-Deval test and how the degraded material in the jar may interfere with the efficiency of the test procedure. For this analysis, three aggregates that showed significantly different loss in standard micro-Deval test were selected. Of the three, one sample had low loss (10%), while the other two samples had high loss (> 30%). The low loss aggregate sample was a granite aggregate. Between the two samples that that had high loss values, one was a marine limestone, and the other aggregate was a granite aggregate containing high levels of mica.

Los Angeles Abrasion and Impact Test

The LA abrasion and impact test was conducted according to AASHTO T 96 procedure. This test procedure involves placing a washed and oven-dried sample of aggregate (5000 grams) of specific gradation, in a large steel drum along with a specified number of steel spheres. The number of steel spheres used in the test (ranging between 6 and 12) is a function of the gradation of the aggregate being evaluated. The aggregate sample is then subjected to abrasion and impact loading by rotating the steel drum at a specified rate of revolutions per minute. After 500 revolutions, the degradation in the aggregate is determined by sieving the aggregate sample over 1.70 mm sieve (No. 12) and expressing the material passing through as a percent of the original sample mass.

Based on the maximum aggregate size, the AASHTO T 96 recognizes four different gradations – A, B, C, and D. In the present research, LA abrasion tests were conducted on only A, B and C gradations. These gradations will be identified in this paper as LA-A, LA-B and LA-C. The size-distribution for these gradations is provided in Table 2.

It is obvious from comparing the gradations for the LA and micro-Deval tests in Table 2 that LA-A and MD-A are not equivalent to each other and hence no correlations are drawn between the results of the two tests based on the “A” gradation.

Although, LA-B and MD-B gradations are slightly different from each other in their maximum size of aggregate, attempt was made in this research to draw correlations between results of micro-Deval tests and LA abrasion and impact tests. LA-C and MD-C gradations are equivalent in their relative proportions of different sizes of aggregates. Therefore, correlations based on the “C” gradation were explored between the results of micro-Deval tests and LA abrasion and impact tests for all the aggregate sources.

Sulfate Soundness Test

The sodium sulfate and the magnesium sulfate soundness tests were conducted according to AASHTO T 104-94 test procedure. Both test procedures are identical to each other except for the soak solution in which the aggregate is immersed. According to this test, a sample of aggregate is immersed in a sulfate solution for 16 to 18 hours in order to saturate the void space in the aggregate with the solution. Thereafter, the aggregate is drained and dried in an oven to a constant mass. This results in the crystallization of the sulfates in the void spaces causing expansive stresses. This procedure is repeated for five cycles of immersion and drying in the sulfate solution. At the conclusion of five cycles, aggregate is thoroughly washed, dried, sieved and the weighted average loss is determined.

EXPERIEMENTAL RESULTS AND ANALYSIS

In this section, the results of micro-Deval tests, LA abrasion and impact tests, sodium and magnesium sulfate soundness tests for each of the 23 aggregate sources are presented. Also, correlations between results of different tests are presented. Table 1 presents the summary of all the experimental data collected in this research program.

Correlation Between Micro-Deval Loss and LA Abrasion and Impact Loss values

Figures 1 and 2 present the comparison between micro-Deval test results and LA abrasion and impact test results for "B" and "C" gradation of the aggregates, respectively. Also shown on the graph are the limits on the acceptable percent loss for LA abrasion and impact test (55%) as specified by the SCDOT for HMA and PCC pavements, and acceptable percent loss for micro-Deval test – 17% as recommended by MTO and 18% as recommended by the NCHRP study [1, 3]. Based on the results presented in these figures, a very poor correlation exists between the results of the two test procedures, regardless of the aggregate gradation. However, a general trend showing corresponding increasing losses in both test procedures can be observed. In particular, the aggregates that showed high micro-Deval losses (> 17%), but acceptable LA abrasion and impact losses tended to be marine limestone, marble schist or granites that contained relatively high levels of mica. Except for SC-8 and SC-20, all the aggregates that had micro-Deval loss over 17% (for "B" gradation) also had a "fair" or "poor" field performance rating based on field performance. However, only one of these aggregates failed the LA abrasion and impact loss limit of 55%. Similar observations were made from results obtained using "C" gradation.

Influence of Aggregate Gradation on the Loss Observed in Micro-Deval and LA Abrasion and Impact Tests

Figure 3 shows the micro-Deval test results of all aggregates, for each gradation – MD-A, MD-B and MD-C. Based on the results presented in Figure 3, 20 out of 23 aggregates tested in this study yielded higher loss with MD-C gradation compared to the MD-A or MD-B gradation. Similarly, 20 out of 23 aggregates yielded higher MD-B losses compared to MD-A gradation. The difference between the losses observed with MD-C gradation and MD-A or MD-B gradation ranged between 0.5% and 6%. Similar ranges of differences were observed between MD-B and MD-A gradations. It should also be noted that as per AASHTO T 58, the MD-C gradation is subjected to only 95 minutes of testing, compared to either 105 or 120 minutes for MD-B or MD-A gradations, respectively. This indicates that the aggregate size does have an influence on the micro-Deval loss determined for each of the aggregate sources.

Figure 4 shows the results of LA abrasion and impact tests for each of the three gradations – LA-A, LA-B and LA-C, for all aggregates. Based on the results presented in Figure

4, no specific trend could be observed on the influence of aggregate gradation on the loss obtained in the LA abrasion and impact test.

The difference in the trends observed between the micro-Deval and LA abrasion and impact results reflect the fundamental difference in the mechanism between the two procedures. In micro-Deval test, the predominant mode of degradation is due to abrasion, which is a function of the specific surface area of the aggregate. Therefore, the finer gradations, such as the MD-C gradation exhibited higher loss compared to coarser gradations such MD-A and MD-B. However, the LA abrasion and impact test is predominantly an impact test and its results are not as dependant on the specific surface area of the aggregate as the strength of the aggregate and other factors such as angularity of aggregates.

Rate of Aggregate Degradation in the Micro-Deval Abrasion Test

Figures 5(a), 5(b) and 5(c) show the results of the tests conducted to determine the rate of degradation of the aggregate in the micro-Deval test, using SC-7, SC-12 and SC-1 aggregates respectively. SC-7 represents a good quality aggregate with a micro-Deval loss of 10.5% for the MD-C gradation, while SC-12 and SC-1 represent marginal aggregates with losses of 37.5% and 34.9%, respectively, for the MD-C gradation.

Based on the results shown in Figure 5, it appears that for good aggregates with low overall micro-Deval loss values, the rate of loss of material appears to be uniform throughout the test. However, with marginal aggregates there appears to be a high initial loss followed by gradual reduction in the amount of loss observed.

Figure 5 also presents data from micro-Deval tests conducted on single and multiple samples, to evaluate the influence of accumulation of degraded material on the further abrasion of aggregate. Based on data presented, it appears that in case of good aggregates (SC-7) there is no appreciable influence of the degraded material on further abrasion observed in the test. However, with marginal aggregates it appears that the influence of accumulation of degraded material in the jar on further abrasion is more profound. In case of SC-12 (granite with high levels of mica) appreciable difference could be observed between the loss observed with single and multiple samples in the test. However, with SC-1 (marine limestone) the difference between the observed loss in the micro-Deval test with single and multiple samples was not significant. It is therefore likely that the influence of accumulation of degraded material in the jar on the observed loss in the micro-Deval test may be a function of the total loss observed at the end of the micro-Deval test, as well as the mineralogy of the degraded material that accumulates in the micro-Deval jar.

Correlation Between Loss Observed in Micro-Deval Tests and Sulfate Soundness Tests

Figures 6 and 7 illustrate the correlation between the losses observed in micro-Deval abrasion test for MD-C gradation and the sulfate soundness tests, for sodium sulfate and magnesium sulfate respectively. Also, indicated on the plot are the limits for the acceptable loss in sulfate soundness tests (as per the SCDOT and MTO specifications) and NCHRP and MTO recommended limits for acceptable loss for the micro-Deval test.

Based on the results, there is no significant correlation between the sulfate soundness loss and the micro-Deval abrasion loss among the aggregates tested. In case of sodium sulfate soundness test, the observed loss for all aggregates is less than the SCDOT specified value of 15%. In case of magnesium sulfate soundness tests, the observed loss for all aggregates is less than 12% (MTO specification for maximum acceptable loss), with exception of SC-8. However, several of these aggregates that pass the sulfate soundness tests failed to meet the maximum acceptable loss requirement in micro-Deval test (17%).

Figure 8 shows the results of correlation between sodium sulfate loss and magnesium sulfate loss of all aggregates tested in this research study. With exception of one aggregate (SC-1 a fossiliferous marine limestone), the R^2 value of the correlation is 0.98, indicating a strong correlation between the results of these two test procedures.

DISCUSSION

Based on the results and correlations observed in Figures 1 through 4, it appears that the loss obtained in micro-Deval test procedure has no significant correlation with LA abrasion and impact test, regardless of the gradation of the aggregate. Vast majority of the aggregates tested in this study satisfy the existing specifications for loss in the LA abrasion and impact test (< 55%). However, significant number of aggregates failed to meet the maximum acceptable loss in micro-Deval test as recommended by NCHRP 4-19 study and MTO specifications (six out of the 23 aggregates failed to meet the NCHRP recommended limit of 18% and seven out of 23 aggregates failed to meet MTO specified requirement of 17% for MD-B gradation). Among the all the aggregates evaluated in this study, only two out of 23 aggregates (with MD-B gradation) and three out of 23 aggregates (with MD-C gradation) were incorrectly categorized by the micro-Deval test.

All aggregates tested in this study satisfied the existing specification on maximum acceptable loss in sodium sulfate soundness test procedure (< 15%). In case of magnesium sulfate soundness test, with exception of SC-8, all aggregates met the MTO specified maximum loss of 12%. Based on these findings it appears that neither of the two sulfate soundness tests adequately characterized the true field performance of the aggregates.

CONCLUSIONS

Based on the micro-Deval abrasion test, LA abrasion and impact test, sodium and magnesium sulfate soundness tests conducted on 23 different sources of aggregate in South Carolina, the following conclusions are drawn:

1. The micro-Deval test provided a more accurate characterization of aggregate performance, compared to other tests evaluated in this study.
2. The micro-Deval abrasion loss of aggregates did not correlate well with the LA abrasion and impact loss.
3. The micro-Deval abrasion loss of aggregates did not correlate well with loss in either sodium sulfate soundness test or the magnesium sulfate soundness test.

4. For a given aggregate source, finer aggregate gradations generally yielded higher losses compared to coarser gradations in the micro-Deval testing. However, a good correlation exists between the micro-Deval losses obtained for different gradations.
5. The loss observed in the sodium sulfate soundness test correlates very well with the loss observed in the magnesium sulfate soundness test.

Based on the findings from this study, it can be generally concluded that micro-Deval test is a better test in predicting the field performance of aggregates compared to LA abrasion and impact test or the sulfate soundness tests.

ACKNOWLEDGEMENTS

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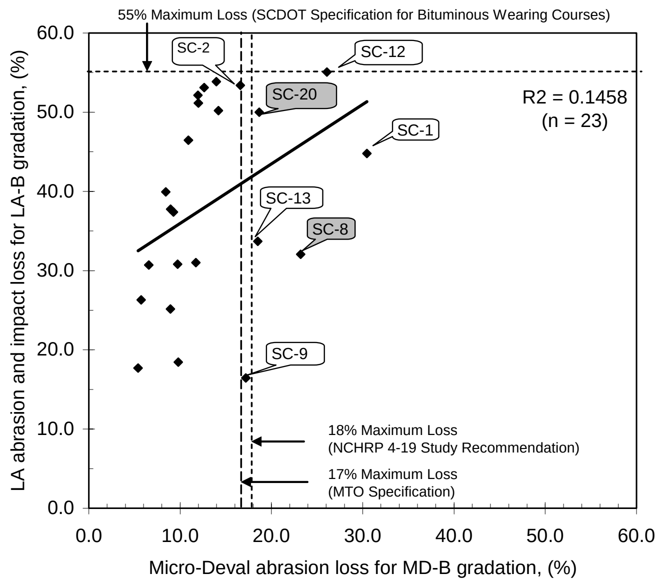


FIGURE 1 Correlation between micro-Deval loss and LA abrasion and impact loss for "B" gradation. (shaded bubbles indicate "good" aggregate and clear bubbles indicate "fair" or "poor" aggregates).

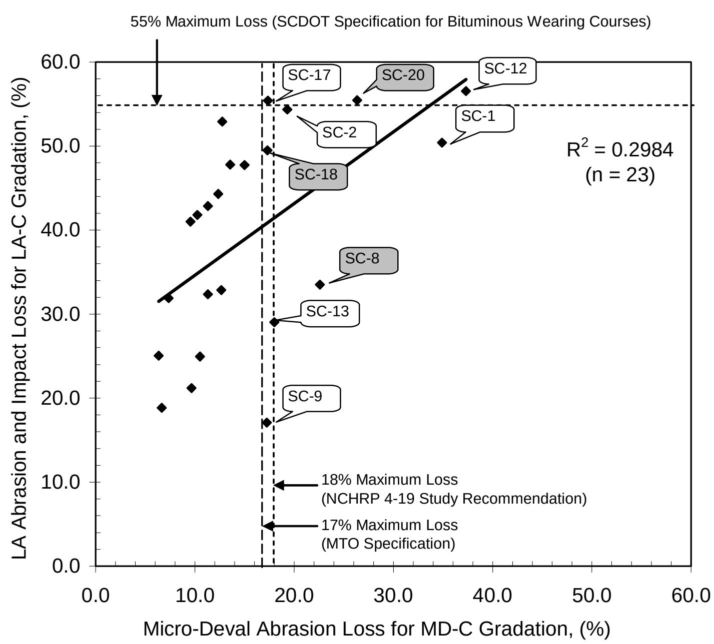


FIGURE 2 Correlation between micro-Deval and LA abrasion and impact loss for "C" gradation. (Shaded bubble – good aggregate; Clear bubble – "fair" or "poor")

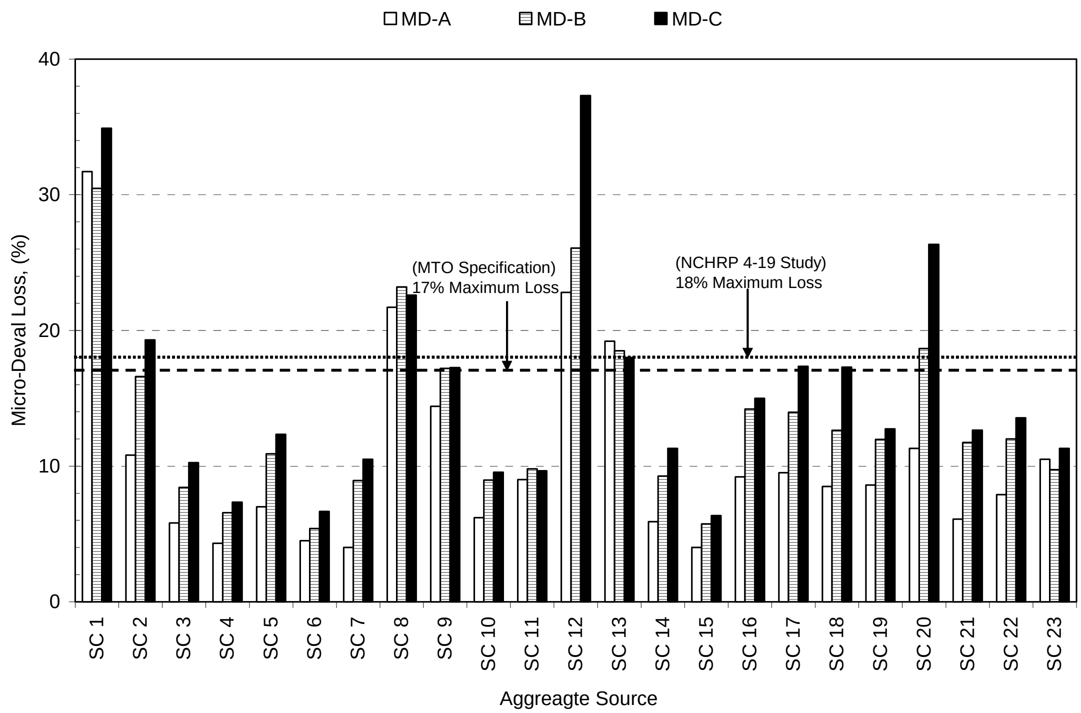


FIGURE 3 Micro-Deval abrasion loss of MD-A, MD-B, and MD-C gradations of all aggregates.

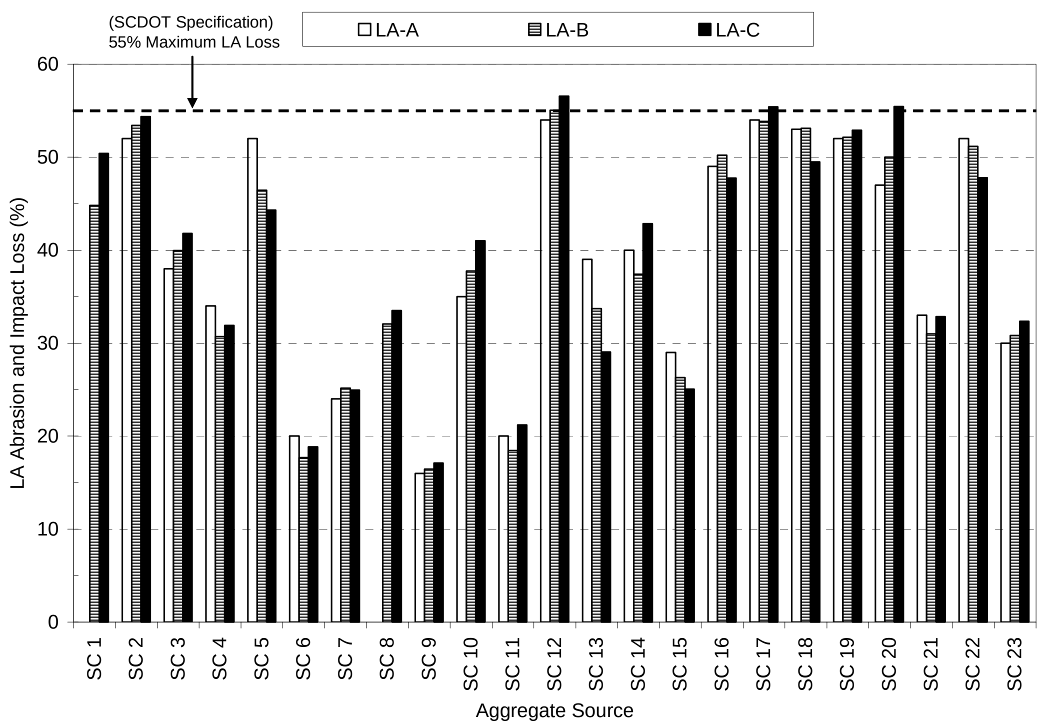
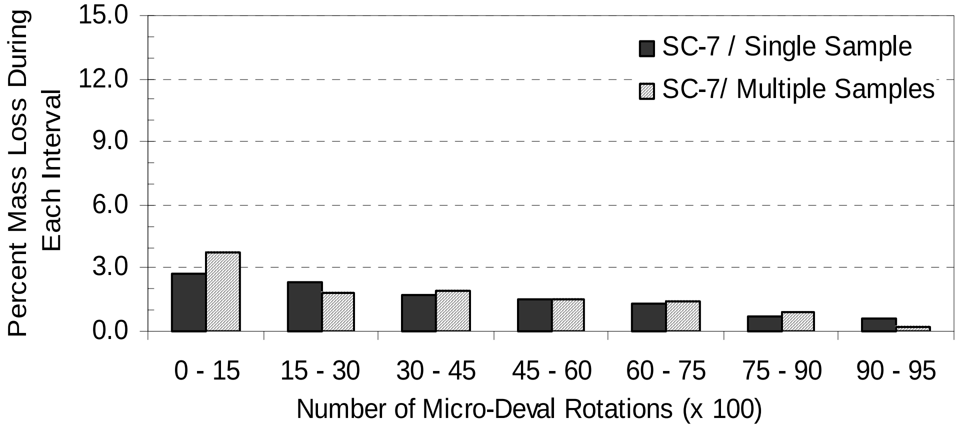
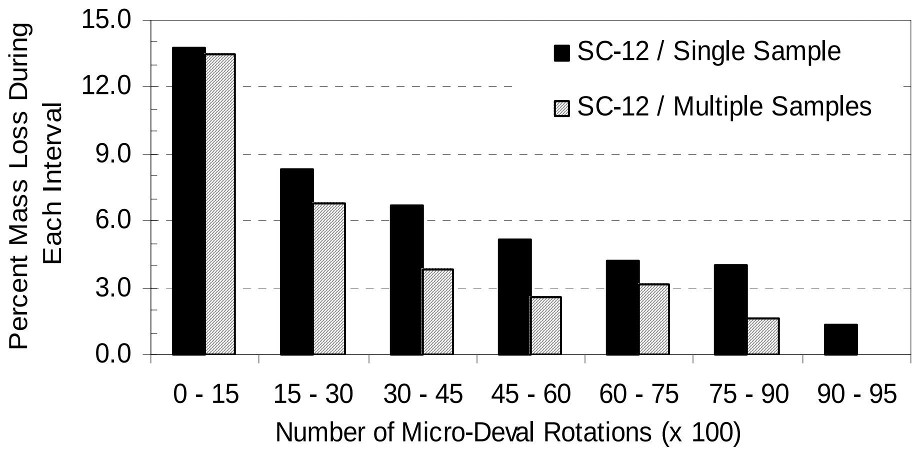


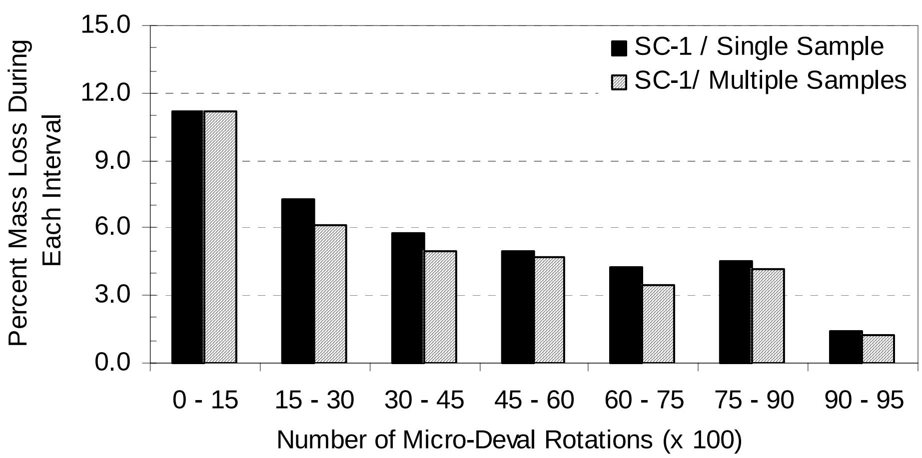
FIGURE 4 LA abrasion and impact loss of LA-A, LA-B, and LA-C gradations of all aggregates.



(A) SC-7



(B) SC-12



(C) SC-1

FIGURE 5 Comparison of rate of aggregate degradation for single and multiple samples in micro-Deval test.

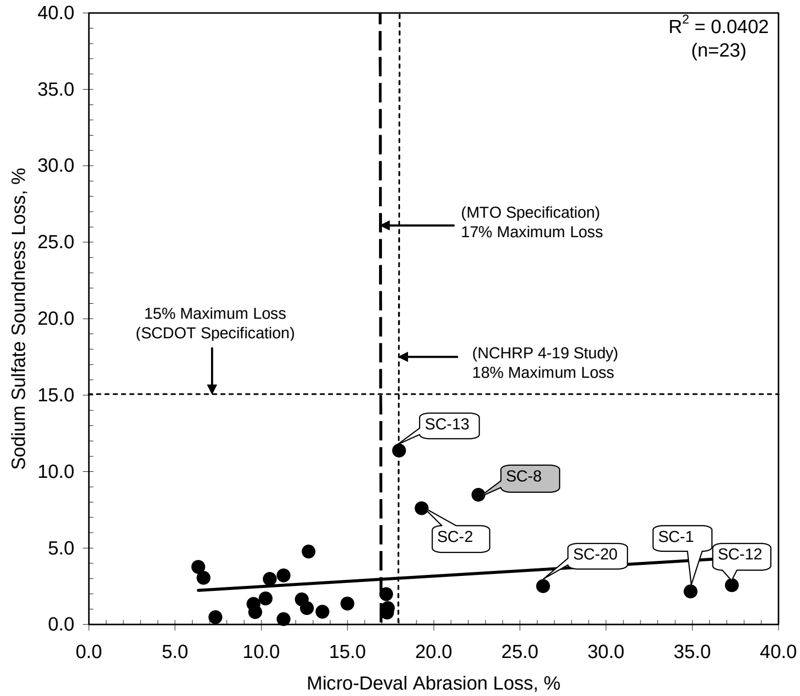


FIGURE 6 Correlation between micro-Deval abrasion loss (C gradation) and sodium sulfate soundness test loss. (Shaded bubble – "Good"; Clear Bubble – "Fair" or "Poor")

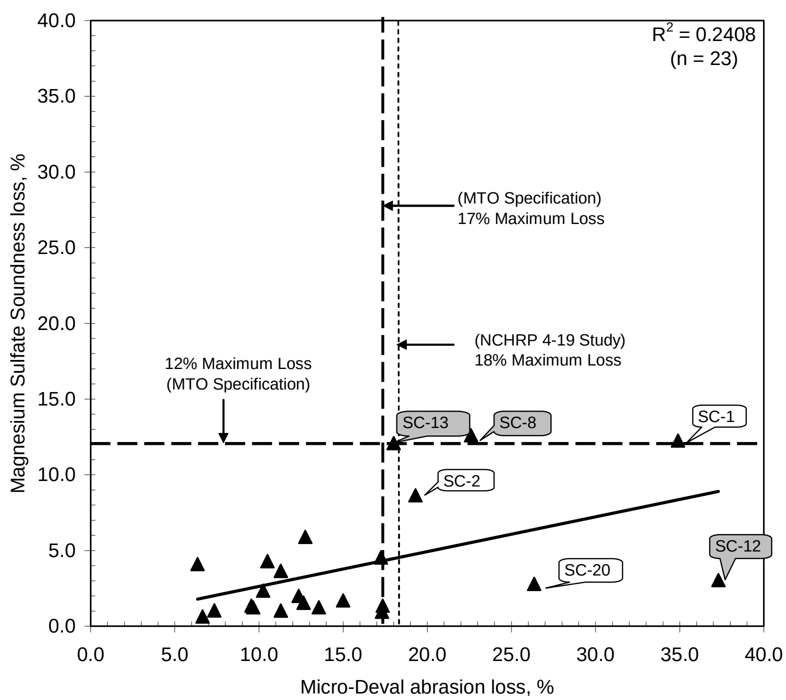


FIGURE 7 Correlation between micro-Deval abrasion loss (C Gradation) and magnesium sulfate soundness loss. (Shaded bubble – "Good"; Clear Bubble – "Fair" or "Poor")

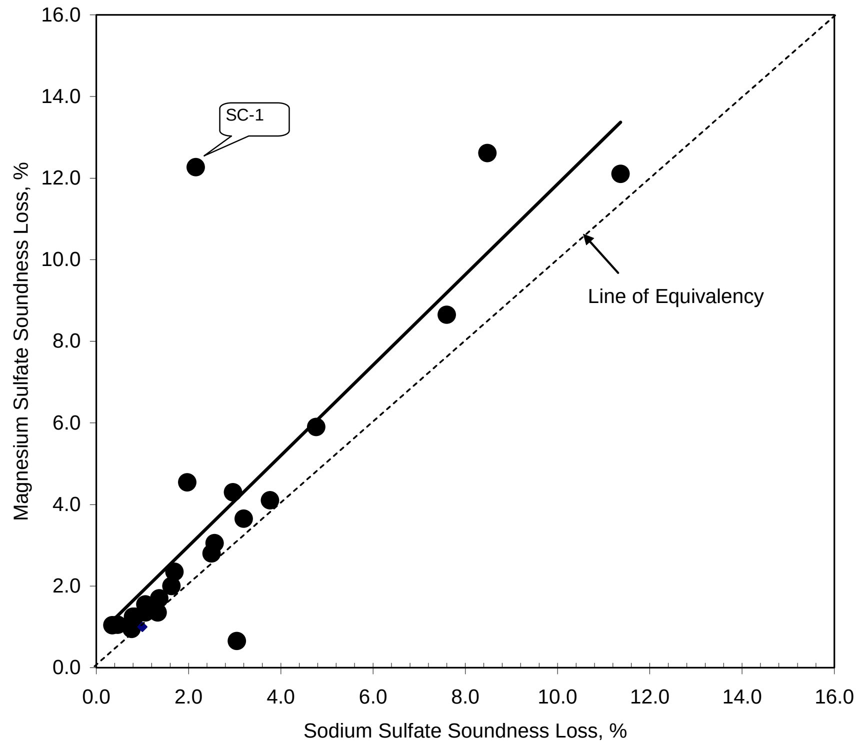


FIGURE 8 Correlation between sodium sulfate soundness test loss and magnesium sulfate soundness test loss.

Determination of a Rock Bulking Factor for Highway Construction

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Abstract:

The rock bulking factor is a relative measure of the change in volume (bulking) of rock after it is excavated. When applied to highway construction, the rock bulking factor is used as a design tool to balance rock excavation quantities and rock fill requirements for a given elevation grade and alignment. In the past, the Ministry of Transportation has used a rock bulking factor of 1.5, which has provided satisfactory estimates for most work within the province. However, during the construction of several projects involving large rock cuts in northern Ontario, additional material was often required due to shortfalls in available rock quantities. These errors prompted an investigation into the accuracy of the rock bulking factor used on MTO projects. In general, rock bulking factors currently used by highway designers are based on past experience or taken from estimates from the quarrying industry. There are no published reports that deal specifically with bulking factors for highway rock fill construction.

Two experimental full-width embankments, 40 m long and 3 m high were constructed in separate studies to examine the variability of the bulking factor for different rock types and to establish a more accurate value for design purposes. The first embankment was constructed using blasted rock from a mafic gneiss quarry in the vicinity of Parry Sound, Ontario. The second embankment was constructed from granite gneiss and marble-breccia taken from rock cuts made as part of highway realignments near Minden, Ontario. At each site, measurements of the total mass and volume of the finished embankment were accurately determined. The rock bulking factor for each embankment was calculated by comparing its average density with the average in-situ density of the source rock used in its construction. The rock bulking factors for the two projects were 1.353 and 1.376 respectively. Based on the two studies, a new rock bulking factor value of 1.35 has been adopted for the design of future rock excavation projects in Ontario involving competent bedrock. For less competent rock types such as shale, shaley limestone, chlorite schist and deeply weathered rock, further investigation is still needed.

Introduction

In a simple experiment, two full-scale rock fill embankments were constructed to examine the bulking factor for different rock types encountered in Ontario highway projects. The purpose of this study was to examine bulking factors using a single method of construction and compare the results with the current “bulking factor” value of 1.5 used by the Ministry of Transportation of Ontario (MTO) for estimating the amount of rock material available for use in fills.

In general terms, the “bulking factor” is understood to be the change in volume of a given mass of material in the ground to its expanded volume following excavation. Strictly speaking, the bulking factor is defined as the ratio of the volume of excavated material to the volume of original in-situ material. Volumetric increase is accompanied by a corresponding decrease in the overall density that occurs with the creation of void spaces resulting from blasting and recombination of loosened fragments.

In underground mining and tunnel construction, bulking factors are used to estimate the volumes of excavated rock in order to manage the movement of materials in limited openings at depth. In surface mining operations and quarries, various bulking factors are assumed for estimating the storage space requirements for muckpiles, or manage the available space required for stockpiling processed materials. Where it is desired to optimize construction, test fills may be built to determine a bulking factor for specific materials and construction equipment. Carrying out this work for construction of rock fill dams may prove to be cost effective, or even necessary (USACE, 1994).

When applied to highway construction, a rock bulking factor is used as a design tool to balance the available quantities of excavated rock from “cuts” with the quantities of material needed in “fills” to complete a roadway to a given elevation and alignment. This allows the overall design of the highway to be optimized while minimizing costs associated with importing additional material external to the work. Contractors also apply a rock bulking factor when planning cost estimates for the proposed work.

On MTO construction projects, values for the rock bulking factor are set out in the Construction Design Estimating and Documentation Manual (MTO, 1991). This document identifies a value of 1.5 (volumetric expansion of 50%) for all bedrock types other than for shale, where a value of 1.2 is used (20% net volumetric expansion). The source of the 1.5 bulking factor value is not known. Most likely, it is an estimate based on information gathered from past construction experience and other published data on rock properties. For the majority of MTO contracts involving relatively small volumes of rock excavation, typically less than 100,000 m^3, this value has been adequate. For highway construction, test fills are not normally constructed. Volumetric errors, typically under-runs in the original estimates, are usually compensated for with extra rock being obtained from outside the proposed excavation limits, i.e., overbreak from the walls and shattered rock from beneath the cut. However, with larger rock cut quantities a small error in the bulking factor at the design stage can lead to significant quantity changes during construction.

The need to review the MTO rock bulking factor became apparent following the construction of new highway alignments for the expansion of Highways 400, 11, and 69 in northern Ontario during the late 1990’s. This work involved several projects in the Parry Sound area that required numerous large rock cuts with individual contracts requiring excavation of more than 1,000,000 m^3 each. Severe shortfalls in available rock quantities occurred on these projects, which initiated a review of potential causes of the discrepancies. Such factors as a poorly estimated bedrock surface, the need for additional material as fill in deep swamps and the rock bulking factor where included among these items.

Project Sites

The first embankment was built near Parry Sound using dense, mafic bedrock found in this region. A second rock fill embankment was constructed from more felsic rock near Minden, Ontario (Figure 1). At each site, the mass and volume of the embankment were accurately measured to determine the overall density of the fill. This value was then compared to the in-situ density of the bedrock used in its construction. The ratio of these two density measurements was used to calculate the rock bulking factor for each embankment.

Ontario may be subdivided into two geological regions based on major bedrock types: Precambrian rocks of the Canadian Shield and the Palaeozoic rocks lying to the south (Figure 2). The Precambrian terrain is composed mainly of large expanses of intrusive granites and gneisses along with intermixed metasediments, predominantly quartzite, amphibolite, paragneiss and metacarbonate and east-west trending metavolcanics known as greenstone belts. This region is also dominated by numerous lakes, fault controlled river valleys and poorly drained areas of muskeg filled with deep accumulations of organic materials. In contrast, the southern portion of the province is underlain by relatively flat lying, horizontally bedded sedimentary bedrock mainly of Ordovician to Devonian age. Almost two thirds of Ontario's 21,000 km of highways are constructed through Precambrian bedrock. With the exception of the Niagara Escarpment, there are few rock cuts of significant size within the Palaeozoic.

Operating quarries were selected as the most appropriate locations for carrying out the projects. This setting offered the advantage of calibrated weigh scales and the ability to control the placement of materials. Obtaining a bulking factor directly from general construction is more difficult and less accurate. Detailed excavation and fill quantities, including overbuilding, are generally not available. On MTO contracts, rock made available from excavations becomes the property of the contractor who makes the decision to use it in the most appropriate manner, e.g., placement in fills, crushed for aggregate production, haul road construction or managed as surplus material. Where there is insufficient rock to complete a job, the contractor must obtain additional materials either from within the right-of-way or outside the contract limits as borrow material. These options make it difficult to measure the general use of rock on a contract unless detailed records are kept and follow-up surveys of all material locations are taken.

The two embankments were initiated as separate projects. The first test embankment was identified as a research need in view of ongoing concerns with large rock excavation contracts near Parry Sound. This project was constructed at the Foley Quarry near Parry Sound. Bedrock at this location consists of interlayered gneissic granite, mafic gneiss, and amphibolite gneiss of the Central Gneiss Belt within the Grenville Province (1425 to 1350 Ma) (Easton, 1992). The embankment was constructed from bedrock drilled and blasted to produce feed for granular base aggregate production. Detailed results on the construction and calculations related to this project are published in Materials Engineering and Research Office Report MERO-004 (Senior and Rogers 2003).

The second embankment was included as part of a highway rehabilitation project and was constructed at the Morrison Pit in the vicinity of the town of Minden. Material was obtained from a rock cut widening located within the existing contract limits, approximately 200 m north of the Morrison Pit. Bedrock at this site consists of interlayered granite, felsic gneiss and marble breccia from the Central Metasedimentary Belt in the Grenville Province (1270 to 1220 Ma) (Easton, 1992). Detailed results on the construction and calculations related to this project are published in Materials Engineering and Research Office Report (Draft) (Senior et al, 2005).

Construction

Work at the Parry Sound site took only a few days to complete, as the work was tendered under a separate contract specifically for the completion of a rock bulking factor embankment study. The excavated rock was extracted from relatively homogeneous bedrock used for production of granular base course and hot mix aggregates. In comparison, the Minden site project was included as a single lump-sum item within a capital construction contract that took place as part of the roadway improvements along Highway 35. Construction of the embankment took several weeks to complete at this site, as work was dependant on the contractor’s schedule. Bedrock at the Minden site was more variable.

At each site, a rock fill embankment was built in accordance with current Ontario provincial standards for subgrade construction of a two-lane roadway (OPSS 206, Construction Specification for Grading). This specification governs the method of construction including maximum block size and grade tolerances of the finished surfaces. Final grades and embankment sideslopes were trimmed according to Ontario Provincial Standard Drawing OPSD 201.010, requiring a 3% slope from centreline to the edge of the shoulder and 1.25:1 grade for the sideslopes. Each finished embankment was at least 40 m in length and had a minimum width between the shoulder roundings of 9 m. The final elevation of the embankment was a minimum of 3 m above the highest original ground point.

Construction started with building of an access ramp in order to reach the elevation of the final surface before construction of the actual embankment began (Figure 3). The remainder of the embankment was constructed by end dumping rock and bulldozing materials into place in a single, continuous lift (Figure 4). Materials spilling over the sideslopes outside the footprint area of the embankment were removed or placed back into the fill. Oversized rock fragments, i.e., those exceeding one third of the height of the fill vertically and one half of the height of the fill horizontally, and other material that could not be accommodated into the embankment were removed entirely. All side slopes were trimmed and the final upper surface of the embankment was brought to proper crossfall (Figure 5).

Total station surveys to generate elevation profiles were taken at three different stages of construction. An initial survey was taken during the layout prior to placement of any material in order to establish original ground elevation. A second survey was taken following completion of the fill which, when compared to original ground, gave an accurate measurement of the constructed volume of the embankment (Figure 6). All materials used in construction of the embankment were removed (Figure 7) and a third survey was taken. When compared to the original ground survey, this measurement was used to determine the volume of excavated materials to allow adjustments for any possible over or under-excavation.

During removal, the materials were weighed using on-board scales that kept a tally of the weights of each individual load. The on-board scales were calibrated against the large certified platform scales located within each facility, before, during and after the embankment was removed. For calibration, haul vehicles were loaded with stockpiled aggregates and weights were compared between the two scales. The loader scales were adjusted as necessary. Removal was as close to the original ground surface as possible (Figure 8).

Measurements

Bedrock Density, Parry Sound Site

At the Parry Sound site, a bulk relative density was determined for bedrock using two different methods. The first method employed visual estimates of the percentage of various rock types present in the source material along with laboratory density measurements of each rock type determined from representative field specimens. All laboratory density measurements were determined using test method LS-604. The weighted average calculated from this approach is given in Table 1.

Table 1. Bedrock density, Parry Sound site (visual)

Rock type	Percentage occurrence (%)	Relative Density
Gneissic granite (pink)	20	2.644
Biotite-hornblende gneiss	70	2.759
Amphibolite	10	2.981
Weighted average		**2.758**

The second method determined the bulk relative density from crushed granular aggregate samples taken from existing quarry stockpiles. These samples are considered to represent a homogeneous blend of the all the rock types present in the embankment. Test results from this method are given below in Table 2.

Table 2. Bedrock density, Parry Sound site (stockpile samples)

Sample	Relative Density
1	2.850
2	2.869
3	2.899
4	2.873
Average	**2.873**

The average bedrock density of 2.816 t/m^3 from these two determinations is used in further calculations.

Bedrock Density, Minden Site

Because of the variable nature of the bedrock at the Minden site, bedrock density was determined from a random selection of seventy-four hand specimens (ranging from 5-20 cm diameter) taken directly from the embankment. Figure 9 shows the distribution of the test results from the 74 samples. An average density of 2.703 t/m^3 was determined and used in further calculations.

It should be noted that the average bedrock densities do not take into account existing joints, fractures and other voids present within the bedrock mass. However, for the purposes of this study, the average density of representative samples taken from the embankment or stockpiles is considered practical and sufficient for the purpose. Qualitatively, the in-situ bedrock was observed to be massive with few or no fractures. Joints were extremely wide spaced and apertures very tight without any infilling materials.

Calculations

Embankment Density

For each embankment, the volume of the completed fill, the volume of over-excavated materials, the total mass of materials removed during excavation and the corrected mass are listed in Table 3. The embankment density is based on the corrected mass and initial volume. The corrected mass contains a mass adjustment for over excavation of the underlying material, which consisted of well compacted, crushed, dense graded, granular aggregate produced by the operating quarry. A conversion factor of 2.2 tonnes per cubic metre was used, based on field records of compacted road base granular materials meeting the gradation requirements of OPSS 1010 for Granular A. In addition, the mass correction for the Parry Sound site contains an adjustment for moisture due to heavy rains that occurred during the excavation and weighing phase of the project. A moisture correction of 0.5% was assumed, further reducing the mass of excavated materials by approximately 24 tonnes. Results obtained from provincial construction records are illustrated in Figure 10. Direct samples of the underlying material were not taken.

Table 3. Field volume and mass measurements

Site	Volume (m^3) (initial survey)	Volume (m^3) (over-excavation)	Mass Removed (tonnes)	Corrected Mass (tonnes)	Embankment Density (tonnes/m^3)
Parry Sound	2233.0	100.2	4890.2	4646.9	2.081
Minden	1850.0	71.0	3789.82	3633.6	1.964

Bulking factor

The calculations based on the above measurements for the two bulking factors are shown in Table 4.

Table 4. Rock bulking factor calculations

Site	Bedrock	Embankment Density (t/m^3)	Bedrock Density (t/m^3)	Bulking Factor
Parry Sound (Foley Quarry)	gneissic granite, mafic gneiss, amphibolite gneiss	2.081	2.816	1.353
Minden (Morrison Pit)	granite, felsic gneiss, marble breccia	1.964	2.703	1.376
Average		**2.023**	**2.760**	**1.365**

Discussion

In a review of bulking factors conducted by the MTO Construction and Operations Branch (MTO, 2000) it was discovered that most agencies do not take a highly developed approach to determining fill quantity estimates when dealing with rock excavation (Table 5). In some Canadian jurisdictions, a mass haul diagram is provided to bidders without providing a bulking factor. It was also reported that a majority of U.S. Departments of Transportation purchase rock at a specified grading rather than determine rock cut and rock fill quantities. In these cases, excess rock is either wasted on sideslopes or hauled offsite for further processing.

Bulking factors may also be estimated from published material data, such as density tables of various rock types for intact rock and for broken loose rock from the aggregate and blasting industry. Examples are given below.

The bulk unit weights for the materials given in Table 6 are published by the Aggregate Producers of Ontario (APAO) and include rock types typically found in commercial quarry operations located within Palaeozoic bedrock. The information is published with the clarification that these weights “may vary in accordance with moisture content, grain size, degree of compaction etc” (APAO, 2000). The source of the data is unknown.

Information available from the blasting industry is given in Table 7. Materials similar to those encountered in Ontario are included, specifically igneous and metamorphic rock types as found in the Precambrian as well as clastic and carbonate sedimentary rocks typical of the Palaeozoic. The reader should consult the source for a complete listing.

Table 5. Bulking factors used by other jurisdictions (MTO, 2000)

Jurisdiction	**Bulking factor**	**Comments**
Newfoundland	1.2 to 1.25 (suggested)	Assumptions up to designer
New Brunswick	Shale: 1.1 to 1.25 Granite: 1.4 to 1.5	
British Columbia	1.7 (suggested)	
Michigan	No set factor	Limited rock work in Michigan
Minnesota	No set factor	
New York	No set factor	Assumptions up to contractor. Rock and earth excavation combined into “Unclassified Excavation” item
Washington State	Granite: 1.72 (assumed)	Based on Excavation Handbook (Church, 1981)
US Army Corps of Engineers	No set factor	

Table 6. Bulking factors from aggregate industry data (after APAO, 2000)

Material	**Bulk Unit Wt (kg/m^3)**		**Bulking Factor**
	Broken	**Solid**	
Dolomite	1742	2895	1.66
Gypsum	1809	2783	1.54
Limestone	1555	2607	1.68
Sandstone	1511	2319	1.54
Shale	1579	2666	1.69
Slate	1653	2687	1.63

Table 7. Bulking factors from blasting industry data (after ETI, 1980)

Material	Density (kg/m^3)		Bulking Factor
	Solid	Broken	
Granite	2723	1762	1.55
Gneiss	2883	1842	1.57
Mica-schist	2723	1762	1.55
Slate	2723	1762	1.55
Marble	2483	1602	1.55
Limestone	2643	1682	1.57
Dolomite	2883	1842	1.57
Sandstone	2403	1522	1.58
Shale	2563	1682	1.52

Bulking factors were recently compiled in a detailed study of collapsed underground mine workings in Waihi, New Zealand (Richards et al, 2002) (Table 8). An average rock bulking factor of 1.41 was selected as being representative of a collapsed, loosely arranged rock mass with a relatively high void space. It was chosen as a middle value from various published bulking factors ranging from 1.3 to 1.8. Richards et al also suggested that lower bulking factors (1.15 – 1.3) would be expected for material stockpiles in open pit excavations, as a result of a wider range of particle sizes and fewer voids due to material breakdown as a result of handling.

Table 8. Bulking factors (Richards et al, 2002)

Source	**Bulking Factor**	**Comments**
Gilmour and Johnston (1912)	1.4	- based on maximum volume of quartz ore drawn off required to maintain working space
Church (1981)	1.5+	- for rocks similar to andesite
Blyth and De Freitas (1990)	1.5 to 1.8 1.25 to 1.4	- for unweathered, blocky igneous and metamorphic rocks - for weathered igneous and metamorphic rocks
Bell and Stacey (1992)	1.3 to 1.5	- for coal measures strata
Whittaker and Reddish (1993)	1.33 to 1.50	- *(no comments provided)*

Bulking factors for soil and rock materials related specifically to highway construction were published by the University of Durham (Table 9). This compilation introduces a “shrinkage” factor that incorporates bulking and compaction of a material at its final destination. Note that the “shrinkage factor” values for rock are greater than 1.0, i.e., the final volume is greater than the original volume.

The bulking factor values for rock given in Table 9 are similar to those used by the aggregate and blasting industries (Table 6 and Table 7). On the other hand, the shrinkage factor values for rock, which average about 1.35, are very similar to the bulking factor values determined by this project. Sandstones, basalts and granites are hard, strong rocks that break into angular fragments in the same way as gneisses from the Parry Sound area. In addition, these values are also similar to a bulking factor value of 1.33 published by the Maine Department of Transportation (MDOT) Highway Design Guide for rock excavation for common borrow (pers comm).

Table 9. Bulking/ shrinkage factors for various materials (University of Durham, 1997)

Material	Bulk Density (Mg/m^3)	Bulking Factor	Shrinkage Factor
Clay (Low PI)	1.65	1.30	-
Clay (High PI)	2.10	1.40	0.90
Clay and Gravel	1.80	1.35	-
Sand	2.00	1.05	0.89
Sand & Gravel	1.95	1.15	-
Gravel	2.10	1.05	0.97
Chalk	1.85	1.50	0.97
Shale	2.35	1.50	1.33
Limestone	2.60	1.63	1.36
Sandstone (Porous)	2.50	1.60	-
Sandstone (cemented)	2.65	1.61	1.34
Basalt	2.95	1.64	1.36
Granite	2.41	1.72	1.33
Bulking factor = Volume after Excavation/Volume before Excavation *Shrinkage factor = Volume after Compaction/Volume before Excavation*			

Conclusions

The experimentally determined bulking factors for end-dumped, partially compacted rock fill from two independent projects in a controlled setting were calculated to be 1.353 for the dense, mafic rock at the Parry Sound site and 1.376 for the less dense, felsic rock at the Minden site. Both projects determined very similar values even though different rock types of variable densities were used. The difference between the two bulking factor values is most likely related to particle size distribution and packing density within each embankment.

The bulking factors calculated by this exercise are significantly less than the standard value of 1.5 used in typical highway design in Ontario. Higher bulking factors (1.5+) are used by the blasting and aggregate industry for excavated rock. Generally, lower bulking factors (1.3 – 1.4) are given for excavated rock that is subsequently placed and compacted in fills.

On any given project, many different factors influence the actual bulking factor such as the variability of the rock type (mineral composition), the in-situ density of the bedrock mass (jointing, fractures, presence of voids etc.), the bulk density of the rock fill (blast fragmentation, particle size distribution, segregation, particle shape, moisture), as well as construction methods (layer thickness, compaction effort). Current MTO construction practice now includes placing of rock fill in maximum 1.5 m lifts and compacting each lift prior to the addition of any subsequent material. This constraint was not included in this project in order simplify the procedure and reduce costs. Construction of thinner lifts may result in increased rock fill density, thus reducing the bulking factors as determined by this study.

Based on the rock bulking factors values obtained from the two independent projects at Parry Sound and Minden, along with comparative data from other sources, the Materials Engineering and Research Office, MTO recommended that a bulking factor of 1.35 be used for new highway construction projects in Ontario. This value is not an average of the two experimental determinations, but a design value to be used for estimation purposes taking into account various assumptions, sources of error and construction methods limiting lift thickness within rock fills to 1.5 m. This new bulking factor is to be applied to most igneous and metamorphic Precambrian rocks of the Canadian Shield and hard, durable Palaeozoic carbonates and sandstones of southern Ontario. For other rock types such as shale, shaley limestone, chlorite schist and deeply weathered rock, it is probable that an even lower bulking factor may be required.

It is expected that a lower bulking factor will help improve design estimates and reduce construction errors, especially with large projects involving significant quantities of bedrock excavation. As a follow up to the recommendations to change the bulking factor, an amendment to the MTO design manual has been issued instructing highway designers to use a rock bulking factor value of 1.35 when estimating quantities.

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Figure 1. Project locations of the two embankments at Parry Sound and Minden.

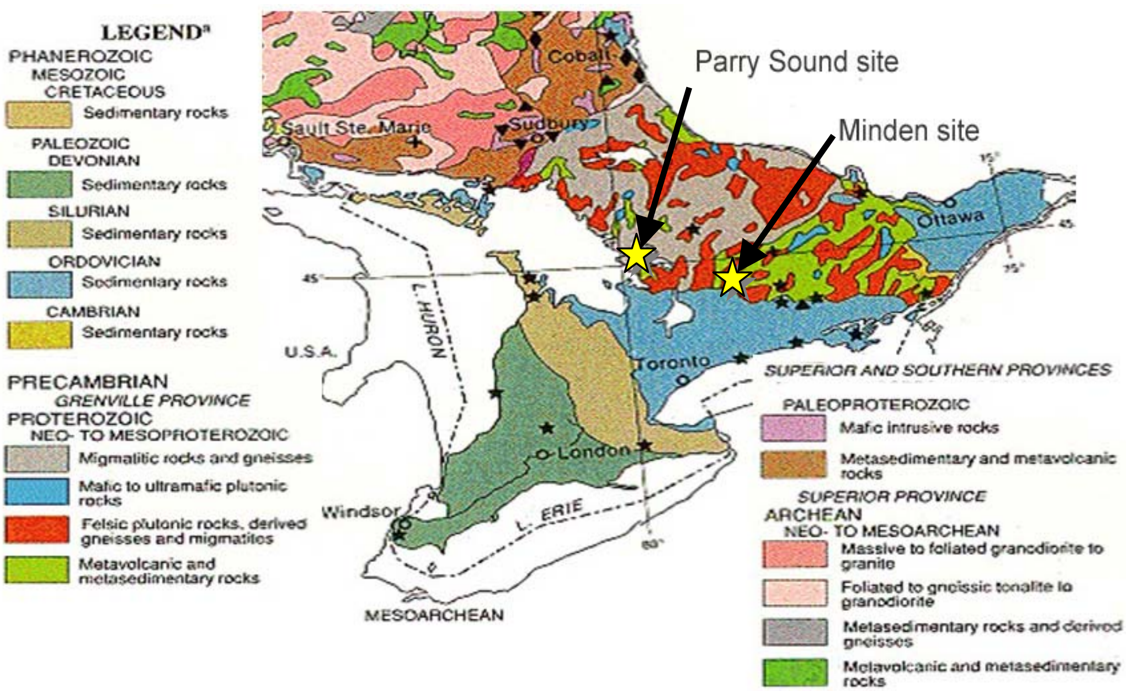

Figure 2. Geological map of southern Ontario showing project locations.

(Figure courtesy of Ontario Ministry of Northern Development and a Mines. © Queen's Printer for Ontario, 1995. Reproduced with permission.)

Figure 3. Building the embankment access ramp at the Parry Sound site.

Figure 4. Partially constructed embankment at the Minden site.

Figure 5. Completed embankment at the Parry Sound site.

Figure 6. Surveying the embankment at Minden to determine volume of the fill.

Figure 7. Removal and weighing materials at the Minden site to determine mass of the fill.

Figure 8. Return to original ground following excavation at the Parry Sound site.

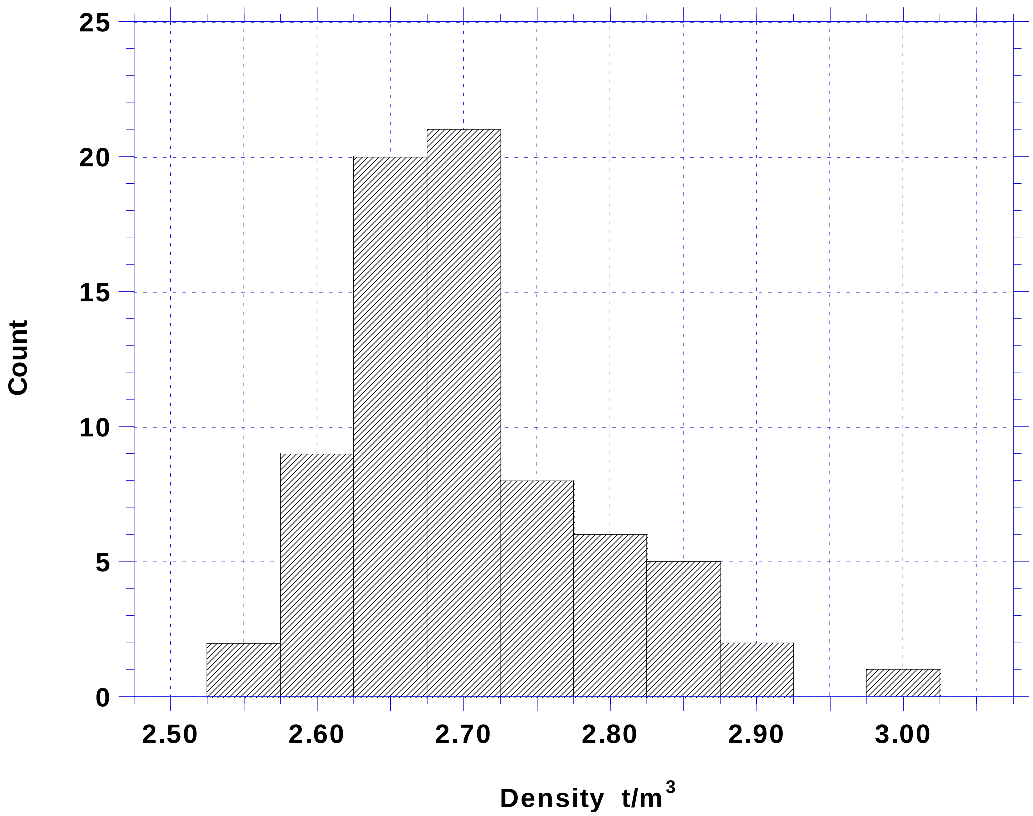

Figure 9. Density variation of rock types sampled at the Minden site.

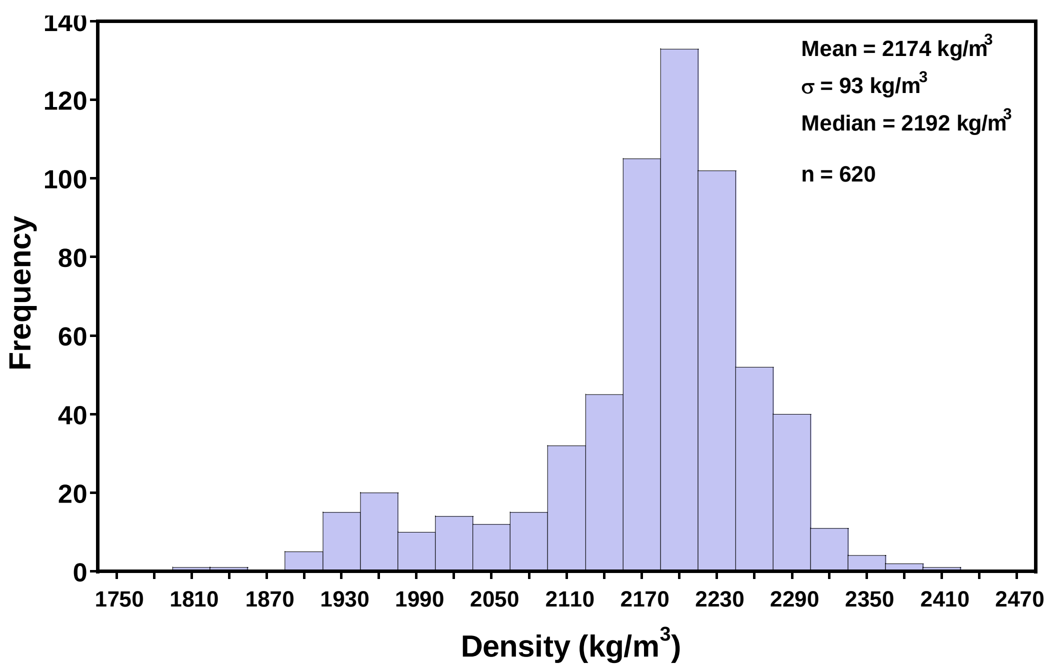

Figure 10. Field densities of dense graded granular road base materials.

Providing Structural Support and Reducing Long-Term Settlement in the Soft Silts and Clays above the Cooper Marl. Ashley Phosphate Road and Route 52 Flyover, Charleston, SC

Jeffrey J. Bean, P.E. and Robin Cheng, P.E.[1]

Introduction

Growth in the area of North Charleston, South Carolina has necessitated the expansion of several roadways and bridges. The project site is located at the Ashley Phosphate Road / Interstate 26 Interchange in North Charleston, South Carolina. See Fig. 1.

Fig. 1. General Plan View of the Interchange

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The project limits include portions of Exit 208 and 209 consisting primarily of the US 52 Flyover and Ashley Phosphate Road where both cross over I26. The highway upgrade includes replacement of two new bridges over I26, widening existing embankments and construction of new embankments, construction of new mechanically-stabilized earth walls and relocation of interchange ramps.

Foundation preparation for the proposed complex construction work has presented an admirable challenge for the engineering design teams as well as the Contractor. Typical foundation soils include soft clays and silts that will require ground modification prior to the construction of the overpass abutments to prevent differential settlement between the bridge structure and the approach ramps. The bridge approach embankment performance is being considered to be most critical to the South Carolina Department of Transportation. Both static settlement due to the proposed embankment construction for the bridge approaches and permanent seismically induced deformation resulting from earthquake forces and / or settlement due to seismically induced liquefaction for the mechanically stabilized earth wall will not be acceptable by the Department.

Given the critical performance of the embankment, the engineering design team, Earth Tech and S&ME, recommended several ground modification methods ranging from prefabricated vertical drains to accelerate and consolidate the clay strata below the new embankment, vibro replacement to improve embankment subgrades and vibro concrete columns to provide additional structural support for the embankment in areas where ground contamination was an issue. The ground improvement program will be described in more details in the following sections.

During the course of the construction work, an unanticipated denser clay strata was encountered creating an equipment refusal that warranted a change of the design approach to augment the vibro replacement stone columns and in some cases replacing the vibro concrete columns. Compaction grouting was performed below the hard strata and also below the completed stone columns. The composite ground improvement proved to be successful based on the settlement monitoring of the ongoing construction of the embankments.

Site Geology

The project site is located within the outer Atlantic Coastal Plain. The underlying bedrock is 2000 to 2500 feet deep overlain by several formations from the late Cretaceous Period and the Tertiary Period that includes the Cooper Group. The upper sediments are composed of Quaternary Period deposits of Recent to Pleistocene age overlying the Cooper Group.

The Cooper Group is generally described as Cooper Marl for engineering purposes. The Cooper Marl is described as a phosphatic limestone consisting of calcium carbonate, quartz, clay and phosphatic sand and pebbles, and small amounts of glauconite, shellhash, and mica. In geotechnical terms, it is typically classifies as a lightly – to moderately over consolidated, high plasticity, sandy silts or clays but can also be classified as a silty sand. Thickness of the Cooper Marl is approximately 300 feet. This is also the foundation bearing layer for which the ground improvement treatment are founded upon as the competent soils. The strength of the Cooper Marl ranges from 5 to 61 blows per foot and CPT tip resistance is in the range of 20 to 70 tons per square foot.

The upper sediments overlying the Cooper Marl at the site in the Charleston, South Carolina consists of layer of sands with varying amounts of interbedded silts and clay overlying clay layers. Clay layers encountered at the site ranged from soft to very stiff and in some cases the clay is very soft. Thickness of this upper sediments ranges from 26 feet to 43 feet. Strength of the upper sediments ranges from 4 to 42 blows per foot. Clayey sand was identified between the clay layer and Cooper Marl. The clayey sand contains high content of clay fines. The primarily clayey upper sediments exhibit plasticity indices as low as 18 for the low plasticity clay and as high as 102 for the high plasticity clays. Moisture contents for the clays range from 28% to 116%.

The surficial Fill varies from 2 feet to 4.5 feet thickness and was identified in a few boring locations. The Fill generally consists of silty/clayey sands to sandy clays. Strength of the Fill ranges from 14 to 28 blows per foot. Water level at the time investigation ranges from 4 to 10 feet below existing ground surface.

Water level at the time of investigation ranges from 4 to 10 feet below existing ground surface. A typical subsoil profile is shown on Fig. 2 indicating the generalized the subsoil consistency and strength parameters.

Ground Improvement Techniques

A total of three ground improvement techniques were employed for the foundation treatment prior to the construction of the new bridge approach embankment and MSE walls. The three ground improvement techniques include vibro stone columns, vibro concrete columns and prefabricated wick drains. Each of the specified ground improvement techniques were shown on the foundation plans at each bridge approach embankment location.

Due to highly variable ground condition and complex layering of silt, sand and clay soils, each ground improvement technique was designed for a particular required function to either accelerate consolidation of the soft and compressible soils by wick drains; to improve the relative density of the sandy soils and to increase bearing capacity of the soft and compressible silt to clay soils by vibro stone columns and vibro concrete columns. The vibratory improvement process would densify sandy soils to mitigate the risk of liquefaction-induced instability and also post seismic settlement. The wick drains were installed by another contractor and will not be discussed in this paper.

The compaction grouting was not originally specified in the Supplemental Specification for the ground improvement work but was introduced as a supplementary ground improvement technique to compliment the unexpected early refusal of the vibro penetration through the stiff to

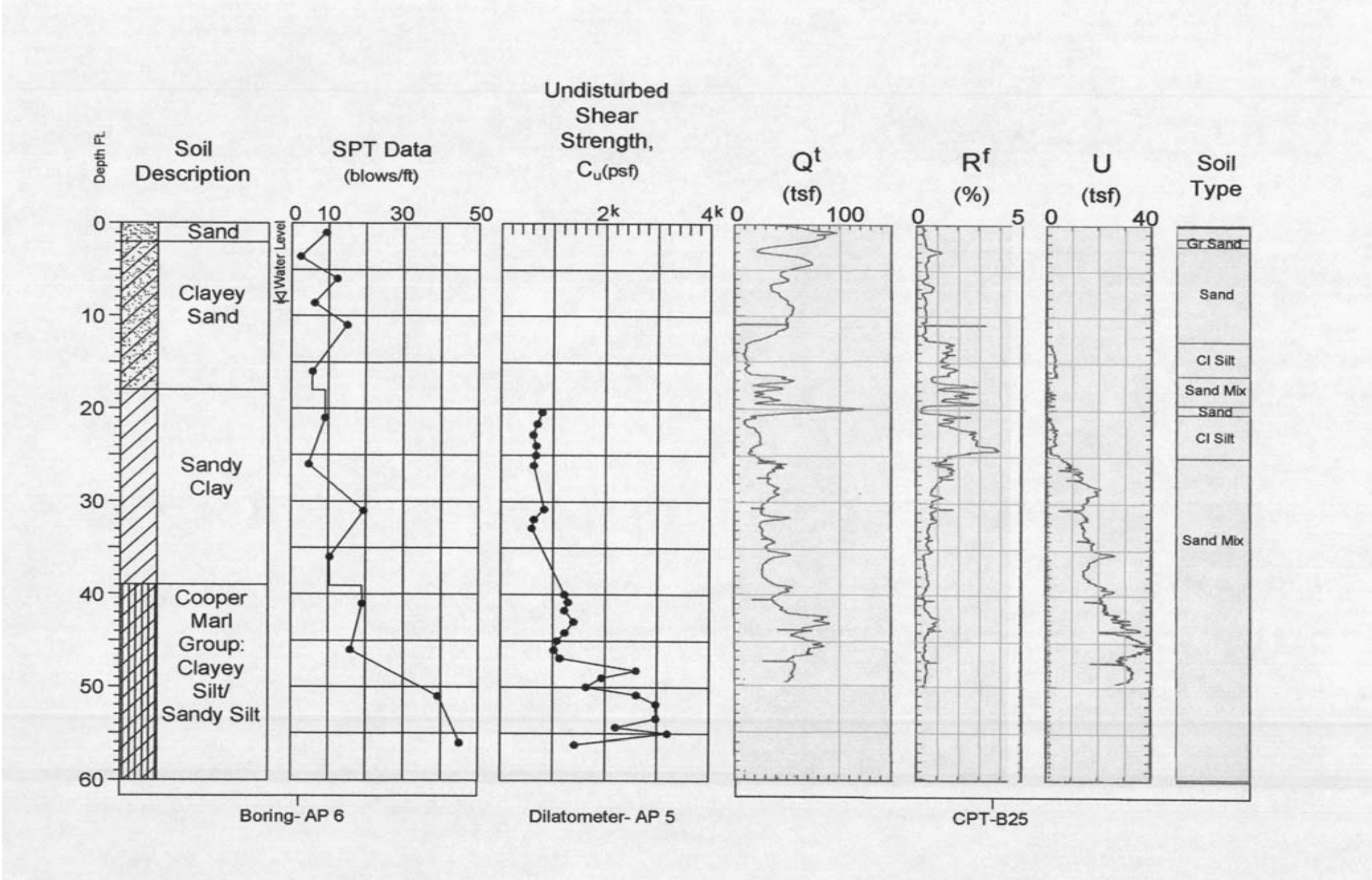


Fig. 2. Typical Generalized Subsoil Profile

increasing stiffer ground condition and the unexpected presence of buried utilities, road aprons and drilled shafts. The technique turned out to be an appropriate measure to mitigate schedule delay impact associated with the constructability issue.

Vibro Stone Columns

Vibro stone columns can be constructed using either a wet top feed vibro replacement method or dry bottom feed vibro displacement method. Both installation methods utilize a vibrating probe and produce a well compacted granular crushed stone backfill in a circular column whether replacing soft / unsuitable soils or displacing soft and compressible soils. Diameter of the stone columns can vary from 2.5 to 3.5 feet and are normally installed in square or triangular pattern of 7 to 10 feet spacing on center.

Given the relationship of the stone column diameter and the spacing, a unique replacement ratio using these two parameter would form the basis of engineering stone column design whether for reinforcement or drainage effect. The specifications of this contract required a minimum of 19% replacement ratio. The specifications also stipulated the method of installation to be dry bottom feed stone columns. Because of the variable layering of the soil conditions at this project sites, variation of the stone column diameter would change from larger diameter in softer soils and relatively smaller diameter in the denser layer based on a consistent installation procedure throughout a soil profile.

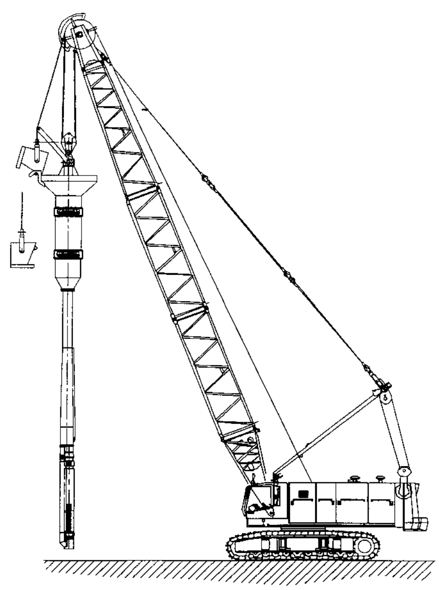
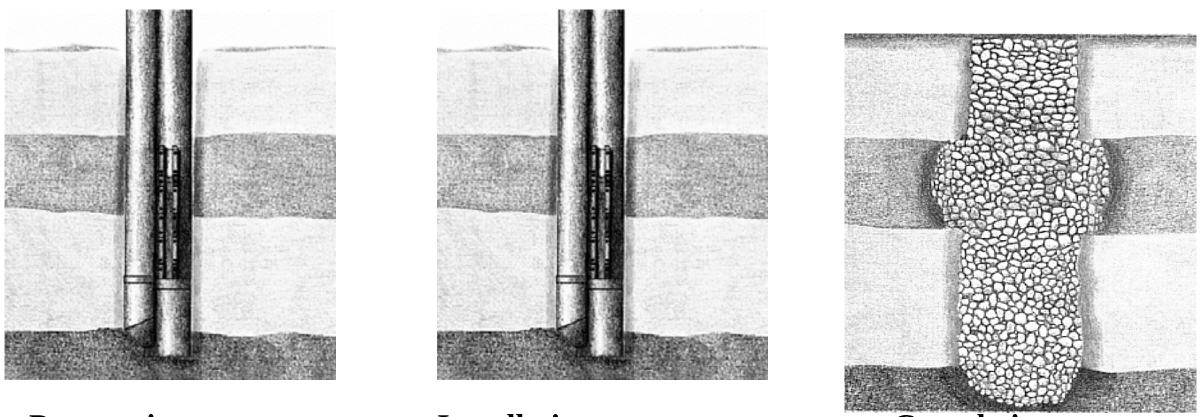


Fig. 3. Typical Vibro Stone Column Rig

The dry bottom feed vibro penetration unit is free hanging by a crawler crane as shown in Fig. 3. A rubber tired wheel loader picks up a shovel load of stone backfill and transfers it into the skip bucket. The skip bucket being lifted up and down via a secondary hoist guided by a frame attached to the hopper for stone backfill delivery.

The stone backfill is emptied into the hopper and in turns release into the pressure chamber by a closing mechanism. While the closing mechanism is closed, the system is pressurized from the closing mechanism to the bottom of the delivery tube. The positive pressure prevents any potential cave in condition or bottleneck effect. The sequence of operations can be seen in Fig. 4.



Penetration
The vibroprobe penetrates vibration and aid of compressed air to the required depth.

Installation
The stone column is installed by adding gravel through the separate gravel duct alongside the vibroprobe.

Completion
The surface is leveled by and eventually roller compacted.

Fig. 4. Typical Vibro Stone Column Installation

The specified backfill in this contract for the vibro replacement stone column is #57 sized crushed aggregate meeting ASTM C33 coarse aggregate requirements.

Vibro Concrete Columns

Vibro concrete columns are installed similarly to vibro stone columns using similar equipment. There are two different types of vibro concrete column installation. The concrete can be either by pumpable wet concrete with 3 to 4 inch slump or dry concrete with zero slump. A specially designed tremie concrete tube is attached to the follow up tube of the vibro probe connecting to the concrete pump located at a suitable location that would be accessible to the concrete truck. The sequence is outlined in Fig. 5.

The vibro concrete probe unit would penetrate to the design depth or at a refusal depth satisfactorily to the engineer. Pumpable concrete would be charged through the tremie concrete tube. The installation process would be by raising and lowering the vibro concrete unit by forming a bulb at the foundation-bearing stratum. In this case, the stiff to very stiff Cooper Marl formation consisting of sandy silt to silt soils. Upon completion of building the bulb, the vibro probe would be withdrawal at a controlled lift to form the specified diameter of the vibro concrete column.

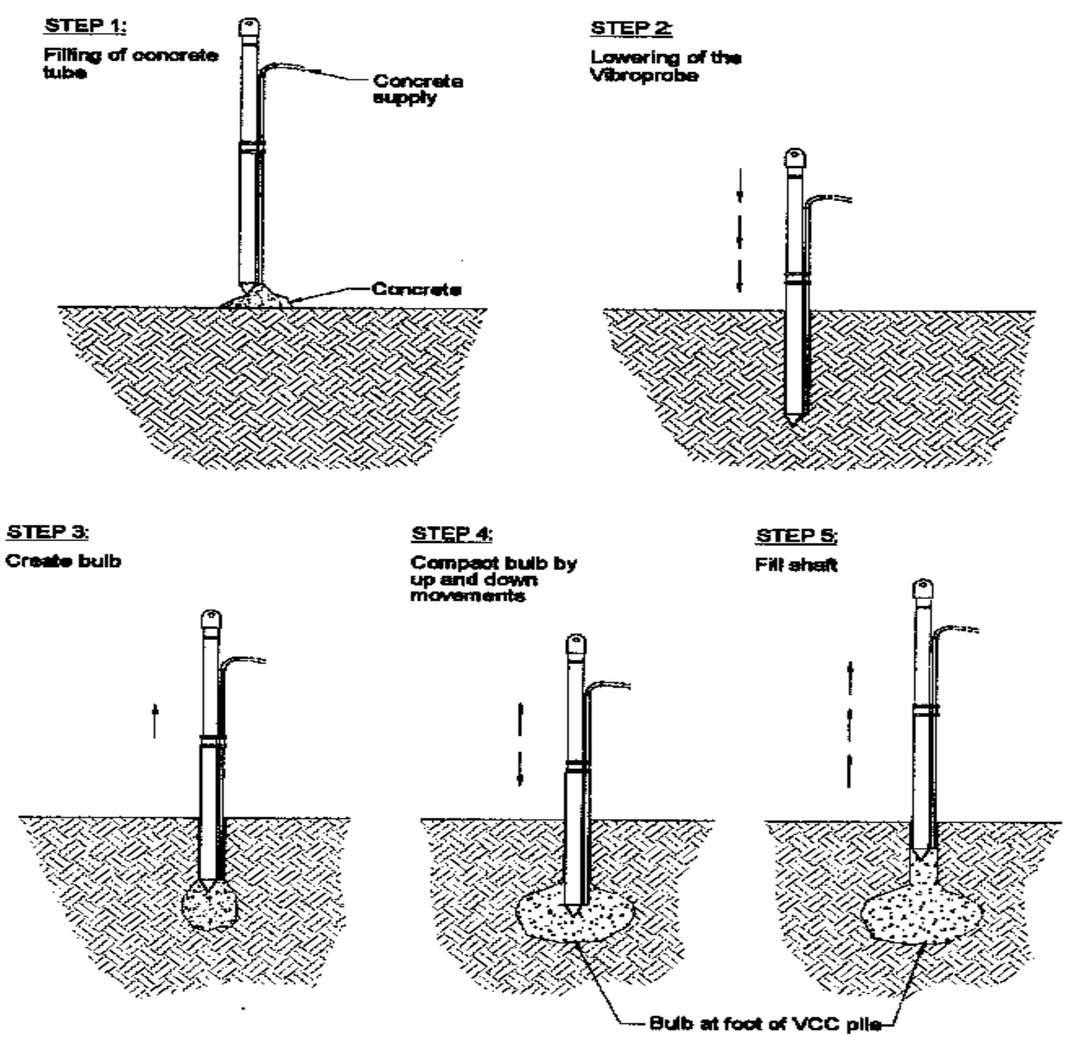


Fig.5. Vibro Concrete Column Installation Process

During the last 2 to 3 feet near the ground surface, the upper bulb would be formed by raising and lowering of the vibro concrete unit prior to completely pull out of the ground. The installation process is maintained in a consistent manner to prevent any unnecessary withdrawal or to cause a bottleneck condition. The vibro concrete unit would remain in the hole at all times. Coordination of delivery of concrete to the project site will be a key requirement for the project to ensure good quality vibro concrete column.

Compaction Grouting

Compaction grouting involves the injection of a very stiff grout (normally a soil – cement mixture) that does not permeate the native soil, but results in the controlled growth of a grout bulb mass that displaces the surrounding soil. The compaction grout is injected through grout pipes that are progressively inserted or withdrawn from a soil mass such that a grout column or series of bulbs is created over the treated depth interval as shown in Fig. 6. Compaction grouting is normally used for various different reasons of ground treatment. Engineers have designed compaction grouting to mitigate liquefaction potential of loose saturated sand soils, some engineers have designed compaction grouting for structural support such as underpinning of building foundations, and in many cases that the treatment is to increase the strength of the soft and compressible subsoils to minimize any unacceptable settlement.

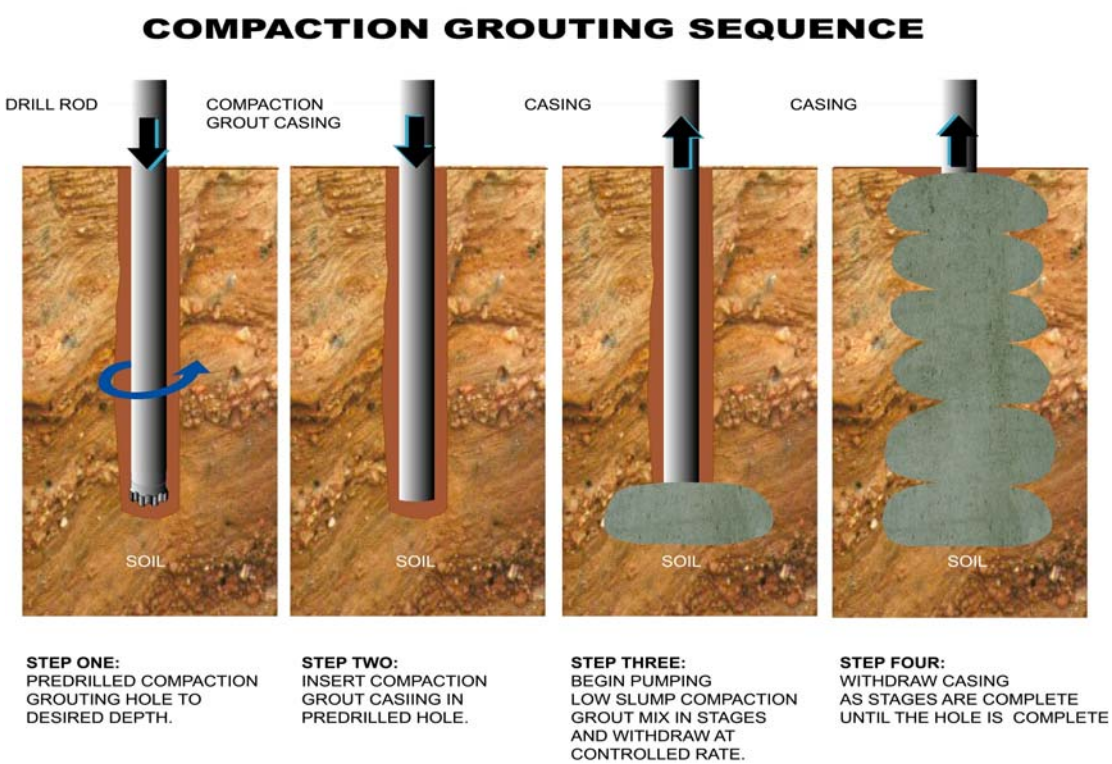


Fig. 6. Compaction Grouting Sequence

Compaction grouting was not originally specified in the Contract but was only introduced to supplement the vibro stone columns that met refusal at a higher elevation than anticipated by the Engineer and around sensitive ground structures. The subsoil condition for US 52 Flyover as indicated in the boring data is shown in Figs. 7, 8 and 9 reflects the anticipated ground conditions. During the course of the vibro stone column work, the subsoil condition at the US 52 Flyover at I 26 at about 22 to 25 feet show substantial resistance to penetration by the vibro probe unit. A review of the boring information; the cone penetration data FO7 and the dilatometer test FO8 indicate that the anticipated penetration depth for the vibro stone column should be in the order of 22 to 25 feet depth below the working grade.

However, the boring data as shown in FO9 indicate the anticipated depth for the vibro stone column would be in the order of 47 feet depth. Therefore another 20 feet was required to reach the design depth. A revised design was put in place that allowed compaction grouting to augment the vibro stone columns and in some cases replace vibro concrete columns.

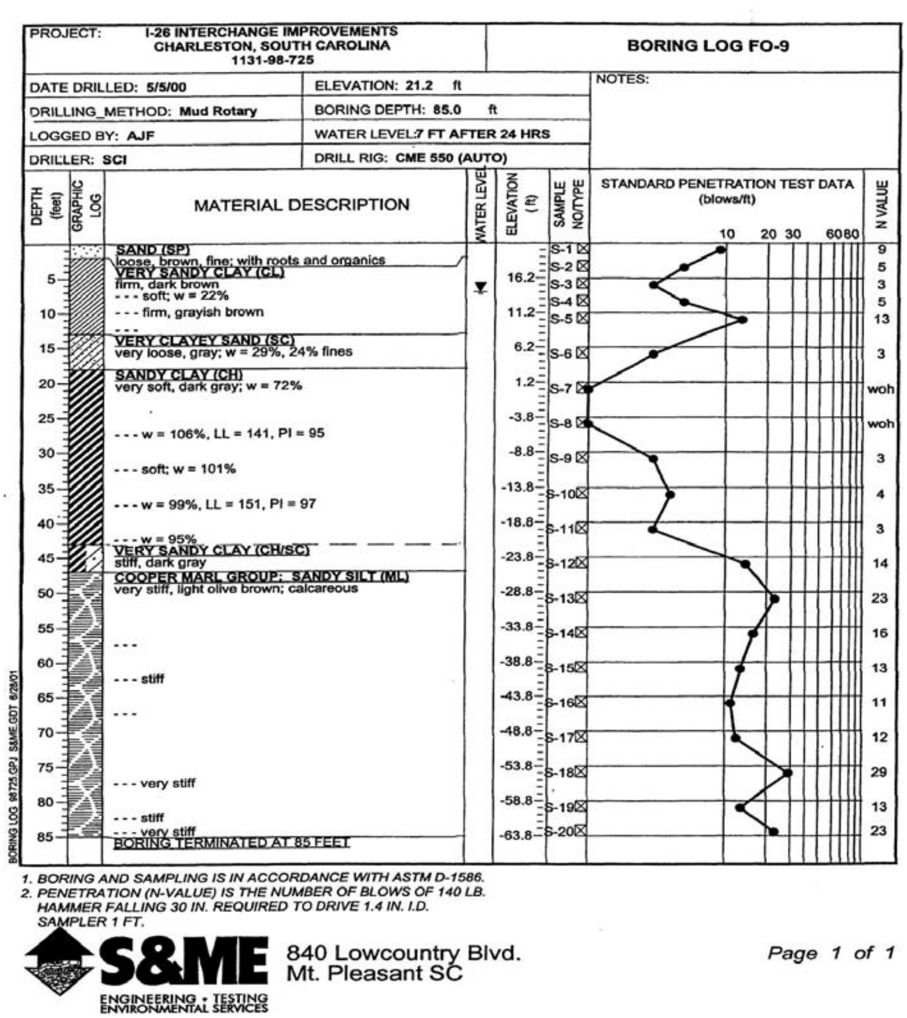


Fig. 7. Standard Penetration Test Boring at South End of US 52 Flyover

Compaction grouting for this project required that a hole be predrilled to a predetermined location prior to pumping grout through a tight-fitting steel casing place to the bottom of the hole. This would create a globular looking structural column. A large track-mounted drilling machine was utilized for this task. The drilling machine was set up so that it could also complete the grouting operation. This setup expedited production in that a second machine did have to be moved back and forth onto the same hole.

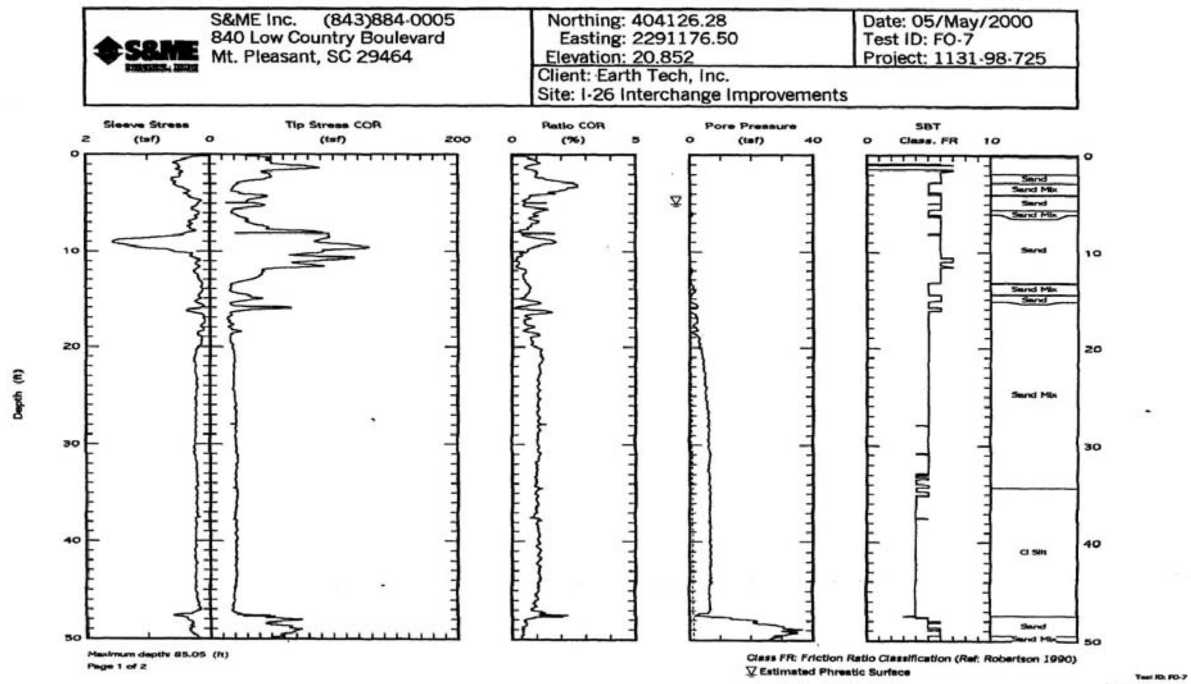


Fig. 8. CPT Boring at South End of US 52 Flyover

Initially a 5-½ inch diameter hole was drilled to total depth with air used to flush out the drill cuttings. Subsequent to the drilling a tight-fitting, internally flush threaded steel grout pipe was placed to the bottom of the predrilled hole so as to prevent collapse prior to the grouting operation. This pipe both temporarily cased the hole and acted as the grout pipe for later. The drill installed the grout casing so that it could be drilled back to the bottom of the hole if necessary. A carbide shoe was placed on the cutting edge of the grout pipe so as to drill though debris, which may have fallen in the hole after the removal of the drill pipe. The drill hole was designed to be just slightly larger than the grout pipe so that it fits snug when placed. This snug fit was sufficient to choke off the low-slump grout from coming up the annulus between the hole and casing thus forcing it into the voids, soft zones and solution features of the soils and creating a structural column.

The grout pipe was connected to the drilling machine with a mast height of such that permitted a minimum 20-foot continuous retraction without breaking joints. A continuous delivery hose was connected to the swing-tube pump, which delivered grout to the end of the grout pipe. That grout was delivered to the project via a redi-mix supplier. The pipe was raised with the drill, in two-foot lifts starting at the bottom of the hole until the top of the zone to be grouted was reached. This method allowed for controlled placement of grout in a discreet zone. The pipe also acted as a packer sealing off the upper portion of the hole while the grout was being placed at a lower zone. The method allowed the grout pressure and flow to be varied in the discreet zone. Essentially by leaving the pipe at the same location for a given duration pressure was developed in the grout delivery system and a larger column was created. A gauge in the grout delivery pipe at ground surface directly in front of the operator was available to monitor pressure. The pump operator recorded flow at the pump. The quantity of grout placed against the pressure was recorded for each stage.

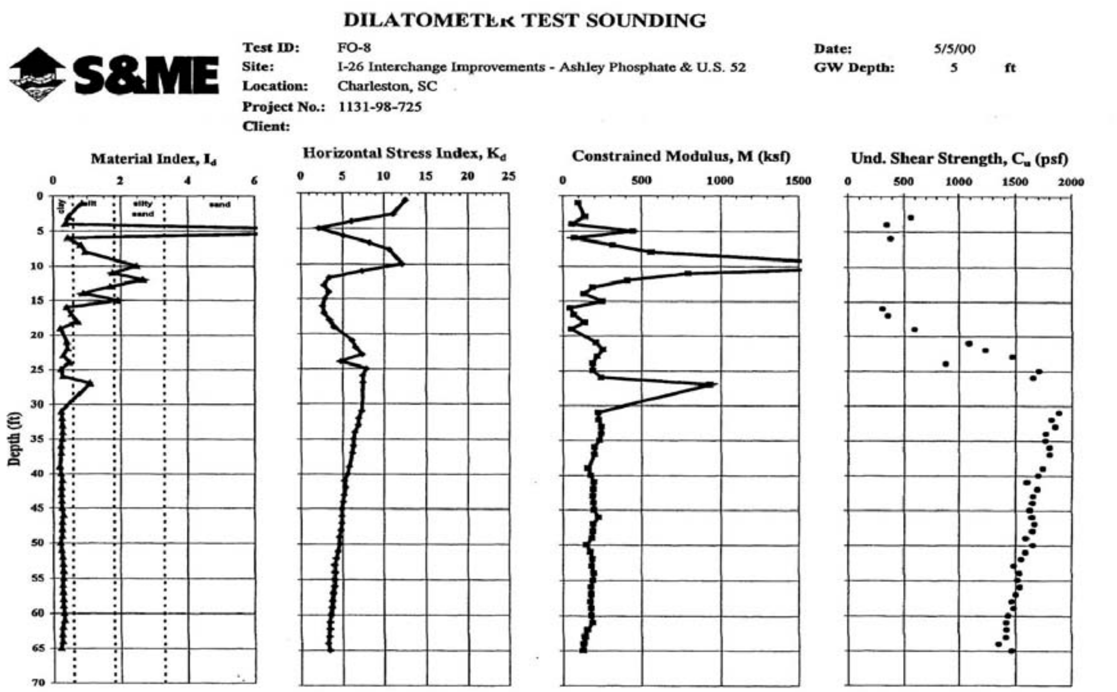


Fig. 9. Dilatometer Test Sounding at South End of US 52 Flyover

Treatment of a particular stage ceased when one of the following criteria were met:

1. A pressure increase of 200 pounds over static head was achieved or a maximum of 300 psi in the grout delivery system.
2. Ground or structural movement was detected.

3. Thirty cubic feet of grout was placed with no appreciable increase in pressure. This did not occur, however had it we would have thickened the mix or suspended pumping until the mix set.

Subsequent to grouting, the grout pipe and any temporary casings were removed and the hole backfilled to the surface.

The grout mix consisted of portland cement, fly ash, sand and water. A mix with design strength of about 3000 psi was used to assimilate vibro concrete columns. The slump was in the range of 3 – 5 inches for the compaction mix.

Primary Low-Mobility compaction Mix consisted of:

Portland Cement	400 pounds
Fly Ash	800 pounds
Sand	1700 pounds
Water	400 pounds

This mix was modified to create one cubic yard. The mix was then checked at the job for slump and batch tickets were collected to verify placement quantities.

Details of Construction

The ground modification work was spread out over two construction seasons or two phases. During the work, however traffic was maintained on the existing US 52 Flyover and Ashley Phosphate bridges while the new bridges were being constructed. The US 52 Flyover consisted of a single bridge with traffic flowing in one direction but the Ashley Phosphate Bridge consisted of two separate bridges with traffic flowing in opposite directions on each segment. These construction plan views are shown in Figs. 10 and 11.

Phase I work began at Ashley Phosphate Bridge at the southeast quadrant of Figure 10. Solid dots represent concrete columns and circles represent stone columns. This work began after the existing south half of the old bridge had been removed. During that period, traffic was shifted onto the remainder of the bridge and preceded in both directions. Ninety-nine vibro Stone Columns using the dry bottom feed system were installed on an eight-foot triangular grid pattern to depths of 30 feet below grade, which was the top of the Cooper Marl. Vibro Stone Columns installed through existing embankment were terminated at the excavation level for the MSE wall and backfilled with sand to the surface. Existing embankment material did not require treating. Five drilled shafts had been installed prior to the ground modification work, which forced some minor modifications to the layout of the stone columns. During the construction of the columns near the shafts, continuous monitoring of the shafts was conducted and no movement was detected. Some minor disruption to the existing paved shoulder on I26 was also noticed during construction, which forced a slight shift in the outermost rows of columns.

Phase I continued with the ground modifications at the US52 Flyover at the southern end of the proposed bridge. This work took place while the existing bridge was in use and presented no disruptions to the traffic pattern. Vibro Stone Columns using the dry bottom feed system were installed on an eight-foot triangular grid pattern to depths of 34 feet below grade, which as it turned out was not the top of the Cooper Marl. We discussed this problem earlier in relation to the boring data. The Cooper Marl was determined to be at about 47 feet. Five drilled shafts had also been installed prior to the ground modifications at this site, as was the case in all of phase one work, which was worked around so as to prevent any damage. Several vibro stone columns were replaced with compaction points in this case.

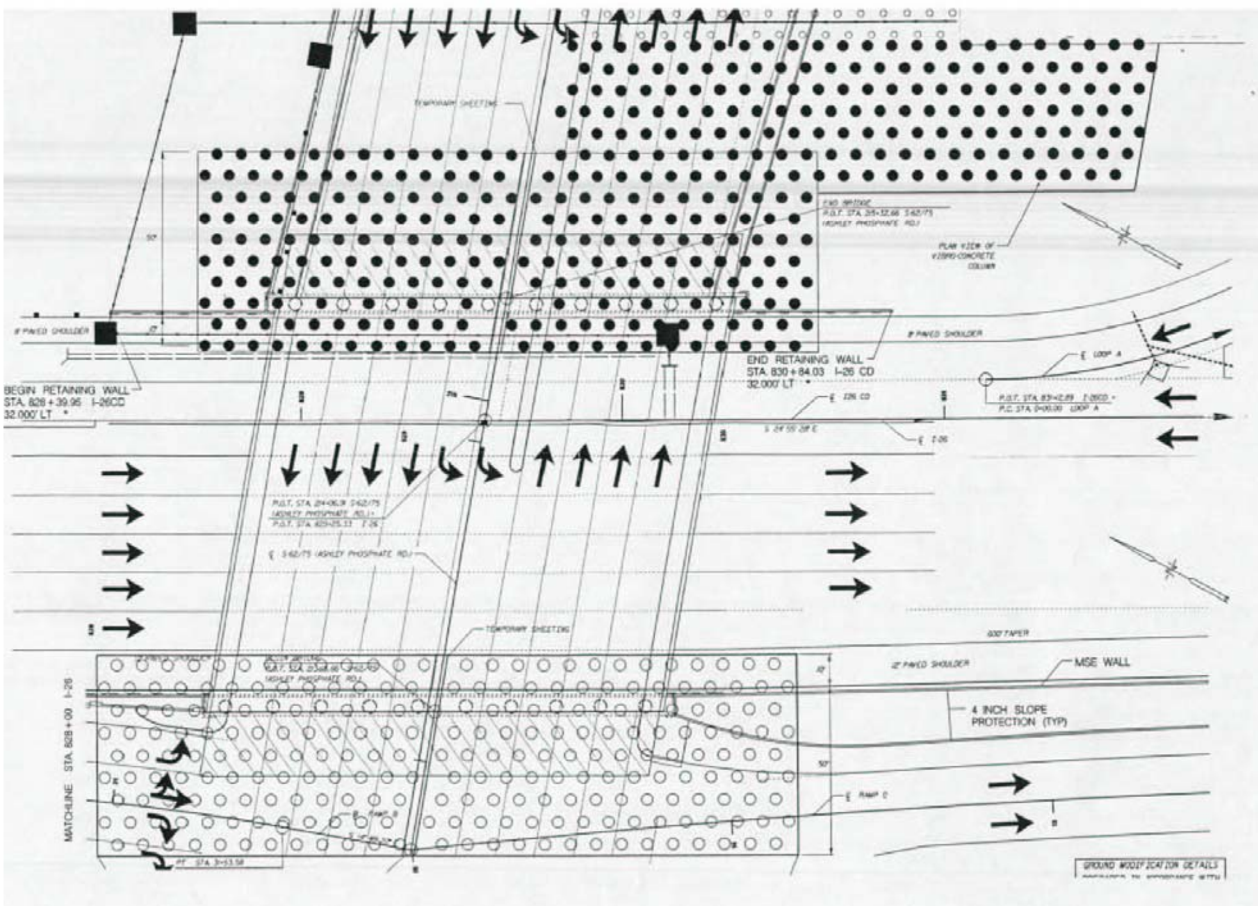


Fig.10. Ashley Phosphate Bridge Crossing Interstate 26

Compaction grouting was performed here to minimize schedule delays and provide for required structural embankment support. The center-to-center distance between holes was an eight-foot triangular grid similar to the stone columns. Holes were placed in between existing stone columns, which did not reach the Cooper Marl and in lieu of stone columns, which were not installed due to preexisting obstructions such as the caissons and road shoulder. In the case where a compaction grout hole augmented the stone column, the treated zone was from about 47 feet back to 33 feet, or the depth to the Cooper Marl back to termination of the stone column. In the case where the compaction grout hole replaced the stone column, the treated zone was from 47 feet to ground surface, however very low pressures was used as the ground surface was approached so that surface heave was minimized. Compaction grout points were designed for 24 inches in diameter or the same as the vibro concrete columns. At this location, 76 vibro stone columns were installed and 96 compaction points with 44 of the compaction points extending full depth.

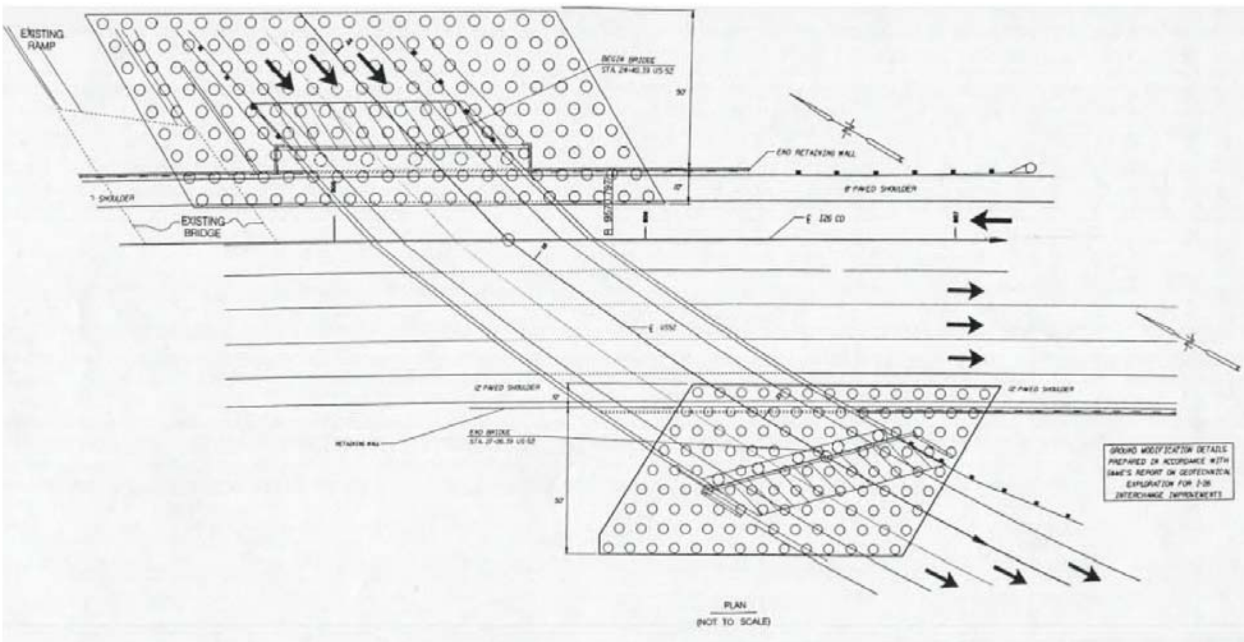


Fig. 11. US 52 Flyover Crossing Interstate 26

One hundred and forty-four vibro stone columns were installed to 26 feet at the northern end of the proposed US 52 Flyover Bridge. These columns penetrated to the top of the Cooper Marl. Predrilling was required through the existing embankment. This was the case for the majority of the columns, which were installed through embankment material. This was since this material was so dense that the vibroflot would not penetrate it. A 24-inch solid stem auger connected to a large track-mounted drill advanced the hole through the stiff embankment prior to vibro work. The existing drilled shafts at this location required some modification to the original stone column layout.

The final work in Phase I was at the other end of the end of the southern section of the Ashley Phosphate Bridge completed earlier in Phase I. This work consisted of the installation of vibro concrete columns. The vibro concrete columns were used instead of the vibro stone columns at this location since the ground water contained some low-level contaminants. Vibro concrete columns would prevent potential cross-contamination of aquifers but still provided some ground improvement and most importantly structural support for the embankment.

A cross section in Fig. 12 indicates vibro stone columns and vibro concrete columns profiles. A total of 175 vibro concrete columns were installed to a depth of 33 feet or the top of the Cooper Marl. The vibro concrete columns were installed with a 24-inch shaft and a 3-foot diameter bulb at the top and bottom as shown in Fig. 12. Some obstructions at this location prevented all of the columns from being installed. They included a 42-inch reinforced concrete pipe with an invert of 13 feet and 5 drilled shafts. To attempt to limit damage to these structures from lateral forces developed by the vibroflot, vibro concrete columns were placed no closer than 15 feet from the pipe. Ninety-six compaction grout points placed to 33 feet replaced the vibro concrete columns near the pipe while maintaining a 5-foot setback. The grout pressure was reduced from the invert to ground level. Some modifications were done around the drilled shafts to mitigate potential damage also.

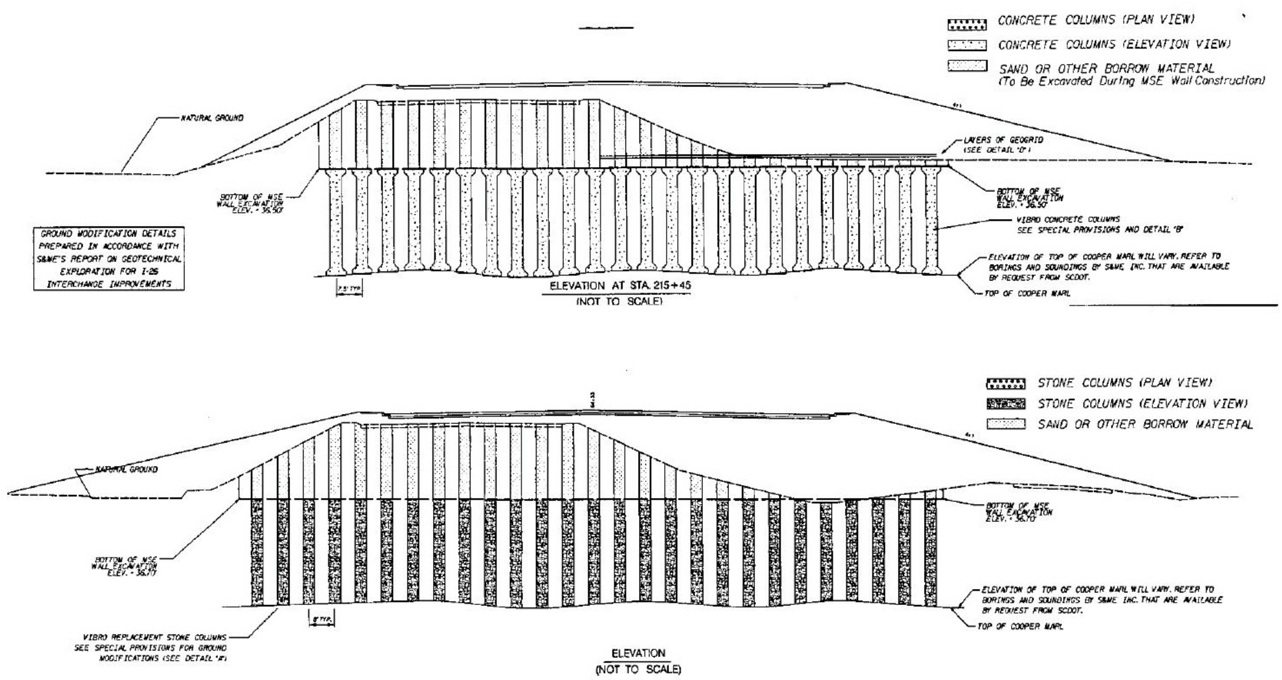


Fig. 12. Cross Section of Vibro Stone Columns and Vibro Concrete Columns

The General Contractor placed embankment material on the improved soil subsequent to the ground modification work. This embankment was monitored for approximately 6 months prior to paving. The construction manager monitored the settlement plates and reported this information to S&ME. Paving work and bridge completion was allowed after settlement had leveled off. Prior to beginning Phase II ground modifications, US52 Flyover was complete along with the southern section of Ashley Phosphate Bridge.

Phase II work commenced about 1 year after Phase I. Phase II work consisted primarily of ground modification at the northern section of the Ashley Phosphate Bridge. This time, the drilled shafts had been left till the ground modification work was to be completed, so there were no issues with navigating them. Still, however the 42-inch reinforced concrete pipe remained and had to be dealt with on the east end of the north section of Ashley Phosphate.

Work at that end proceeded first with the installation of 136 compaction points around the pipeline and 44 vibro concrete columns all to depths of 40 feet. The western section of the bridge was completed with the installation of 76 vibro stone columns and 54 compaction grout points to 28 feet. The compaction grout points replaced vibro stone columns due to the presence of an existing 18-inch reinforced concrete pipe.

Phase II was completed with the installation of 29 compaction points installed on the eastern-most end of the work on the north section of US 52 Flyover Bridge. These points were installed after the new bridge was in place. They represented just a small area of ground improvement. These points replaced stone columns that would have been too difficult to install due to difficult access with a crane. These final points went to a depth of thirty-five feet or the top of the Cooper Marl.

Subsequent to the Phase II ground modification, the general contractor completed the drilled shafts and the north section of the Ashley Phosphate Bridge.

Conclusion

The project sites for the two bridge constructed at Ashley Phosphate Road and US52 Flyover were underlain by variable upper sediments ranging from silty sand soils to clayey sand soils and silty clay material. The variable subsoil conditions in the North Charleston area have presented an admirable challenge for the engineering team to design a ground improvement program to meet the performance requirement of Transportation Department of South Carolina. A total of three ground improvement techniques were employed to provide additional bearing support and to minimize both static and post seismic induced settlement and liquefaction potential of the underlying subsoils. Vibro stone columns and vibro concrete columns were designed to improve the soils and compaction grouting was used to supplement the vibro stone column work and in some cases replacing the vibro concrete columns and stone columns.

Differential settlement between grade supported structures (pavement) and deep foundation supported structures (bridges) is a problem around the Charleston, SC area. Resulting settlement forces continual and costly repaving of many approach ramps.

The Ashley Phosphate/I-26 Interchange project was designed for total static post-construction settlement to be 1 inch or less in areas improved by vibro-replacement and 2 inches or less in areas improved with wick drains. This would be beneath embankments reaching 35 feet in height. Construction would follow a period of 3-6 months after placing embankment fill. During this time monitoring of the embankment fill was conducted to verify the initial settlement due to the surcharge leveling off. Less than expected initial settlement was recorded. Less than 1 inch of post-construction settlement has occurred after bridges and approaches have been up one year at portions of the work.

Compaction grouting was a viable alternative to vibro-replacement in areas where subsurface obstructions were encountered. This caused for a modified design, but seemed to ensure that schedule was maintained with only a small effect on cost.

When difficulties arise on the project, especially with respect to constructability issues, it is important that the owner, engineer, construction manager, general contractor and specialty subcontractor work quickly to mitigate the effect to construction cost and schedule impact delays while maintaining the intent of the ground improvement design. Those involved at this project partnered to quickly identify the problems, present solutions and provide timely turnaround to protect the project's integrity and maintain schedule.

Acknowledgement

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Repair of Voids Above Jack-and-Bore Pipeline Installations Under a Divided Highway

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Abstract: Two pipelines were installed approximately 10 years apart as bored crossings under a divided highway in an arid part of the southwestern United States. Sinkholes discovered locally above both lines alerted the pipeline company to a problem with subsurface voids and loose soils around the bored crossings. The sinkholes were initially filled with surface soils or flowable fill. A ground-penetrating radar survey was attempted in hopes of detecting locations and relative sizes of voids, but interference from nearby overhead 345 kV electrical transmission lines limited the technology's success. A vacuum truck with a plastic vacuum hose extension tube was used to create small-diameter potholes at 31 locations. The pothole procedure consisted of jetting with a water wand to loosen the silty and sometimes gravelly sandy soil so that it could be sucked out of the hole with the vacuum pressure. Loose soil was detected in places above the pipe, but abrupt loss of jetting water was the best indicator of voids.

Limited-mobility displacement grout (compaction grout) was needed to fill voids and compact loose soil above the buried pipelines. Ranging data of the road surfaces were obtained with a three-dimensional laser scanner before and after grouting in northbound lanes to provide a basis for documenting elevation change that might have been caused by the grouting. Arbitrary coordinates were used for the scanner surveys, the locations of which were related to distinctive features on nearby steel lattice electrical transmission towers.

Grouting was accomplished in vertical pipes in the northbound lanes and parts of the median and shoulder. Limited-mobility displacement grout was injected into 129 grout points spaced nominally at 5-ft (1.5-m) centers in two lines over the edges of each pipe. Grout-pressure criteria and local manometer readings on the ground surface were used to minimize expected ground heave. Both sets of scanner survey data showed that the grouting caused no detectible change in the road surface elevation. Error in the scanner data, which varied with distance from the scanner setup location, was attributed to windy conditions, including air blasts generated by passing vehicles, and an increasingly acute angle between the scanner laser and the road surface.

The 3D laser scanner survey revealed a slight depression over one of the pipelines in the southbound left lane, which was subsequently addressed by a supplemental grouting program. The largest amount of grout (143 ft^3 [4.05 m^3]) injected at a single point on the entire void-repair project was in the shallow depression revealed by the scanner survey.

Key Words: compaction grouting, laser scanner, highway, sinkhole, pipeline, void filling, displacement grouting, jack bore, limited mobility grout

Introduction

The purpose of this paper is to describe a method used to repair soil voids beneath an active divided highway surface along two pipeline alignments. The large-diameter pipelines were installed ten years apart as bored crossings under the divided highway. Sinkholes discovered locally above both lines alerted the pipeline company to a problem with subsurface voids and loose soils around the cased crossings. The sinkholes were initially filled with surface soils or flowable fill, but limited-mobility displacement (LMD) grout (also known as compaction grout) was needed to fill voids and compact loosened soil. In this process, soil compaction is achieved by controlled pressure-displacement, and verified by achievement of terminal injection pressure or by measurement of minute surface uplift. Laser scanner technology was used to document elevation changes on the surface of a divided highway where grouting was performed, and proved to be a valuable diagnostic application.

The roadway surfaces of the divided highway were scanned before and after grouting to provide a basis for documenting elevation change that might have been caused by the grouting. Arbitrary coordinates were used for the scanner surveys, the locations of which were related to distinctive features on nearby steel lattice electrical transmission towers.

Grouting was accomplished in vertical grout pipes in the northbound lanes and parts of the median and shoulder with a grout-pressure criteria and manometer readings at two locations on the road surface adjacent to the grout point to minimize any heave. Grouting was not done on the southbound lanes because a geotechnical investigation detected voids or soft soils only on the northbound lanes. The two sets of scanner survey data showed that the grouting caused no detectible change in the road surface elevation. Error in the scanner data, which varied with distance away from the scanner setup location, was attributed to windy conditions and air blasts generated by passing vehicles. The scanner survey revealed a slight depression over one of the pipelines in the southbound left lane, which was treated with a supplemental grouting program.

The remaining sections of this paper pertain to the grouting operations, the laser scanner operations and results, and conclusions regarding the utility of the scanner.

Subsurface Soil Investigation

A limited subsurface soil investigation was conducted upon discovery of a small sinkhole on the highway shoulder above one of the pipelines. Initial attempts to characterize subsurface voids with ground penetrating radar (GPR) were unsuccessful, because of interference from active overhead 345kV power transmission lines. Potholing was selected as the best alternative to GPR for locating subsurface voids, and was accomplished by vertically advancing a pressurized water nozzle, while simultaneously removing cuttings with a 6-inch (0.15-m) diameter vacuum hose fitted with a rigid plastic tube that could touch the active pipelines without concern for damage. The method was preferred to that of traditional equipment (e.g., a backhoe) to mitigate potential damage to the buried pipelines. Use of this potholing method allowed exposure of the pipeline casing or concrete coating and detection of voids and soft soil zones along the pipeline alignment. Pipeline depth and void locations were subsequently plotted in plan view to aid in

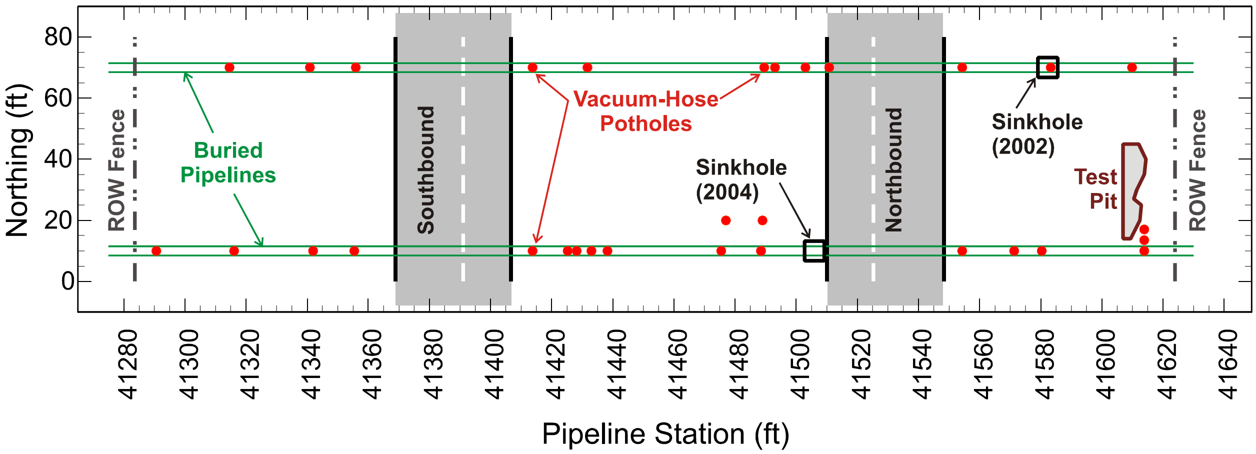

Figure 1. Layout of highway and buried pipelines showing locations of sinkholes, investigation potholes, and test pit.

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development of a remedial approach (Figure 1). Depth to the top of the pipeline was on the order of 10 feet (3 m).

It was determined that jack-and-bore construction methods had likely loosened native soils, consisting of loose to slightly cemented sand and gravel deposited as arid alluvial fan and flood plain sediments of a nearby river, and resulting in a voided condition. Subsequent infiltration of precipitation runoff had likely further degraded overlying subgrade through chimney erosion. Continued enlargement of some of these voids then lead to local subgrade collapse, resulting in sinkhole formation at the ground surface.

Grouting Operation

Limited-mobility displacement (LMD) grouting is a ground improvement technique that uses a mortar-like grout to locally displace loose soils and reduce void space [*Byle*, 1997]. It is a physical process that translates pump pressure to the surrounding soils, using grout as an injected medium. The grout occupies void space resulting from pressure displacement. The process can be controlled carefully to achieve specific objectives, including controlled lift of overlying structures and grades. Use of unsuitable equipment, material or procedures can result in undesired surface heave, or insufficient treatment. The procedure is often referred to as "*compaction grouting*" [*Brown and Warner*, 1973], however, the specific term's use is constrained by contemporary semantics [1980], and precludes fluid behavior.

A slightly higher degree of grout mobility was considered desirable for this project for the purpose of addressing a potentially voided condition. The grout mix used consisted of silty sand, approximately 8 to 10 percent Portland cement, up to 2 percent bentonite, and sufficient water to achieve a 2 to 6 inch slump. This mix design allowed the grout to be pumped with limited fluid characteristics, as was desired for intrusion into local voids.

Although the initial sinkhole was backfilled with soil and lean concrete slurry to mitigate a potentially dangerous condition at the highway shoulder, LMD grouting was used to address subsurface voids that had not yet exhibited surface expression. Grouting allowed for repair of subsurface voids without the need for excavation and recompaction of overlying soils. The grouting program was designed to fill voids and displace loose soils without heaving the surface of the ground or the highway. Lane closures were required for grouting at locations through the pavement.

Two rows of grout-injection points were established for each pipeline. The points on each row were staggered so that the points were approximately 5 feet (1.5 m) apart. Small-diameter steel casing was advanced vertically from the ground surface to a maximum depth that was approximately 2 feet (0.6 m) above the top of the buried pipeline. Grout was then pumped until one of three terminal criteria was achieved: 1) high grout injection pressure, 2) upward deflection of the overlying surface, or 3) grout return to the surface alongside the injection casing. Casing was then lifted to the next planned vertical treatment interval, and the process was repeated. Injected grout volumes were estimated by multiplying the number of grout-pump strokes by the average stroke displacement. Two manometers were used to aid in the observation of road-surface elevation change adjacent to the point being grouted. Equipment used for the LMD grouting operation is shown in Figure 2.

A total of 815 ft^3 (23 m^3) of grout were injected at the bored crossings. Injection quantities tended to vary, with higher relative injection quantities clustered at areas of suspected large voids. A plan-view summary of relative grout quantities injected is presented in Figure 3.

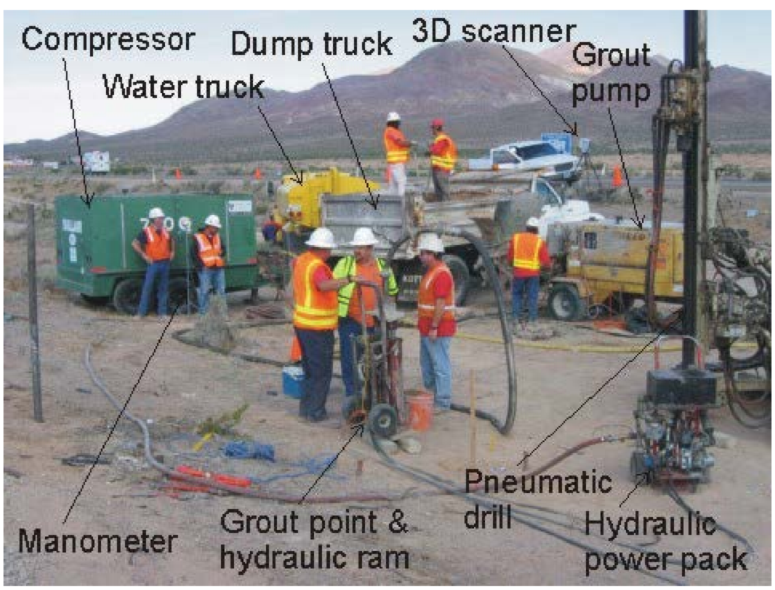

Figure 2. Grouting and scanning operations staged in median of divided highway.

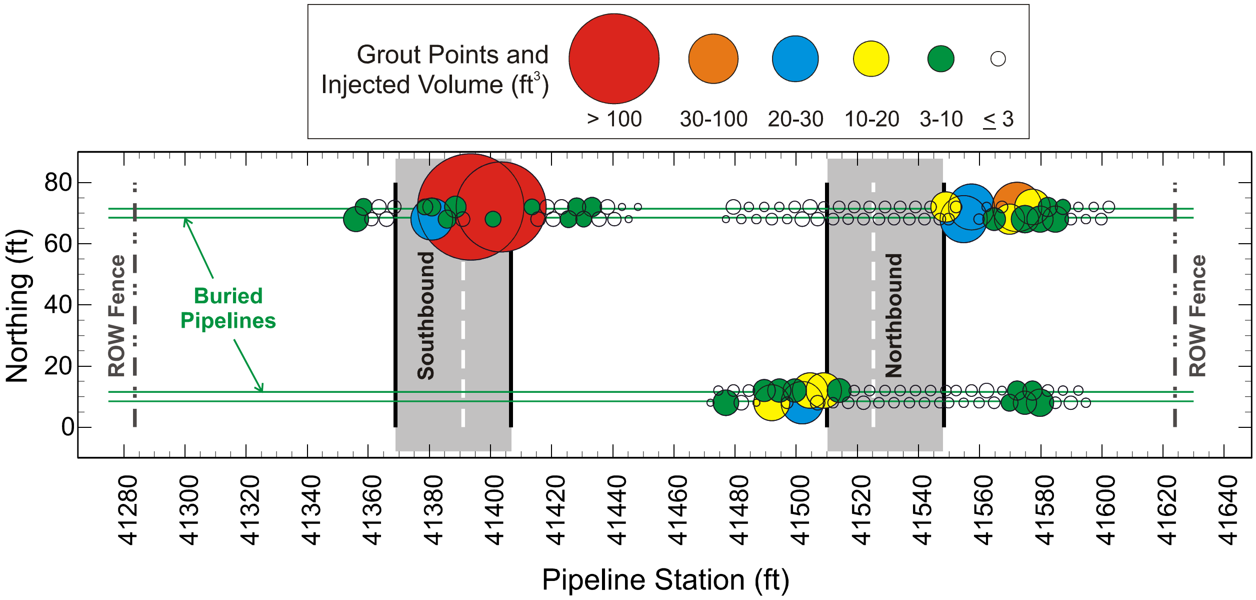

Figure 3. Layout of highway and buried pipelines showing locations of grout injection points and relative grout injection volumes.

Laser Scanner Operation

A Cyrax 2500 laser scanner was used for this project. It was set up in two locations, one location for the northbound lanes and one for the southbound lanes. The scanner location for the southbound lanes is visible on Figure 2. Lane closures were not needed to accomplish the laser scanner surveys. Electrical transmission lines passed overhead at the pipeline crossing of the highway, and distinctive locations on steel lattice towers were used for position reference, thereby avoiding the need for survey control points within the project area. The pre-grouting digital camera view from the laser scanner is shown on the left side of Figure 4, and the corresponding point cloud is shown on the right side.

An arbitrary coordinate system was used for the laser scanner data. The scanner software was used to generate ASCII text files of x, y, and z values after spurious points on passing vehicles had been removed. The pre-grout scan of the northbound lanes consisted of 209,445 points, whereas the post-grout scan consisted of 122,668 points. Similarly, the pre-grout scan of the southbound lanes consisted of 129,686 points, whereas the post-grout scan consisted of 238,481 points. The data were plotted using geographic information system (GIS) software by converting the points to grids, and smoothing the grids. Maps of the pre- and post-grout scanner elevations are shown on Figure 5 for the northbound lanes and on Figure 6 for the southbound lanes.

Errors in the scanner data increased with distance because the scans were conducted on windy days and passing vehicles generated air blasts which had some effect on the tripod-mounted scanner unit located on the shoulder of the highway, as shown on Figure 2. Comparative elevation profiles were drawn at five locations on the northbound lanes and five locations on the southbound lanes. The profile locations are shown on Figures 5 and 6. Representative profiles

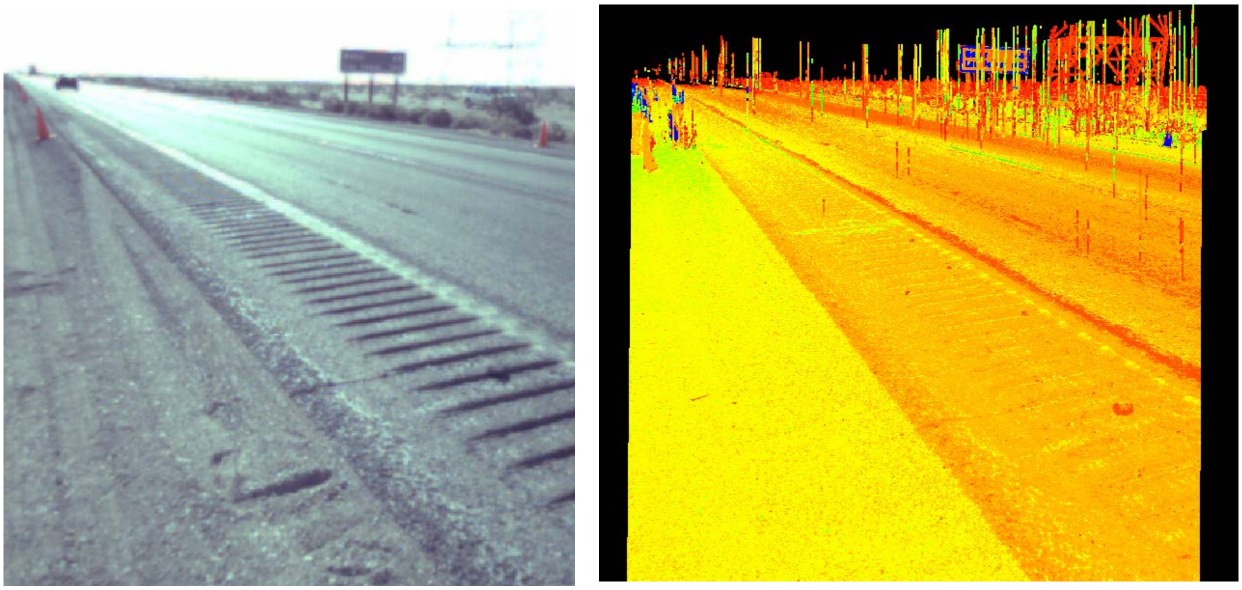

Figure 4. Digital camera view (left) and 3D laser scanner point cloud (right) of the northbound lanes. Vertical lines on point cloud are reflected from passing vehicles.

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are shown on Figures 7 and 8. Figure 7 is the middle profile on the northbound lanes, and four of the raised-dot lane markers are visible at Northing Stations between 65 and 75. Figure 8 is the right profile on the southbound lanes, and shows a slight depression at approximately Northing Station 130.

Discussion and Conclusions

The subsurface soil investigation conducted with the vacuum pothole procedure created small holes that were easily backfilled. The plastic extension tube on the vacuum hose could touch the active pipelines without causing concern for damage. Loose soil zones could be detected by the potholing procedure, but the best indication of shallow voids was abrupt loss of jetting water.

The laser scanner results were valuable in demonstrating that elevation changes caused by grouting operations at the highway crossings of two buried pipelines were within the limits of measurement errors. The laser scanner results supported the conclusion that grouting did not cause elevation change of the road surface. Factors that contributed to the error in the measurements included windy conditions and air blasts generated by passing vehicles, especially highway tractor-trailer vehicles, and the fact that some elevation points were on passing vehicles. The vertical lines on the 3D scanner point cloud (Figure 4) represent points on passing vehicles. Computer software removed the points from the registered model space manipulations of the data. Points on tires very close to the pavement surface were not removed from the data set representing the pavement, further contributing to measurement errors.

The post-grout scanner elevation points were not at exactly the same locations as the corresponding pre-grout points. Consequently, direct comparison on a point-by-point basis could not be accomplished with the equipment and procedure used for this project. Comparative profiles were constructed by extracting points along narrow bands of easting, which correspond to profiles parallel to the lanes, as shown on Figures 5 and 6. The elevation values along narrow bands of easting were plotted against northing station, as shown on Figures 7 and 8.

The shallow depression in one of the southbound lanes that shows clearly at approximately Northing Station 130 on Figure 8 was undetected by visual observations. The shallow depression is approximately 0.07 feet over a distance of about 20 feet (21 mm over 6 m). Grouting was originally performed only in the northbound lanes because neither subsurface voids nor loose soils were detected around the pipelines in the vicinity of the southbound lanes. However, a supplemental grouting program was performed for the area of the shallow depression, and a large void was revealed through injection of an additional 250 cubic feet (7 m^3) of grout. Perforation of the highway pavement and subgrade during casing installation had revealed that subsurface chimney erosion had locally propagated to within 2 feet (<1 m) of the pavement surface.

The laser scanner elevation data provided useful and reasonably accurate information to document limits in possible elevation changes in the highway surface as a result of LMD grouting operations. It also revealed a previously unrecognized shallow depression over one of the two pipelines that triggered supplemental grouting program. Injected grout quantities for the associated void were locally the largest for the overall project.

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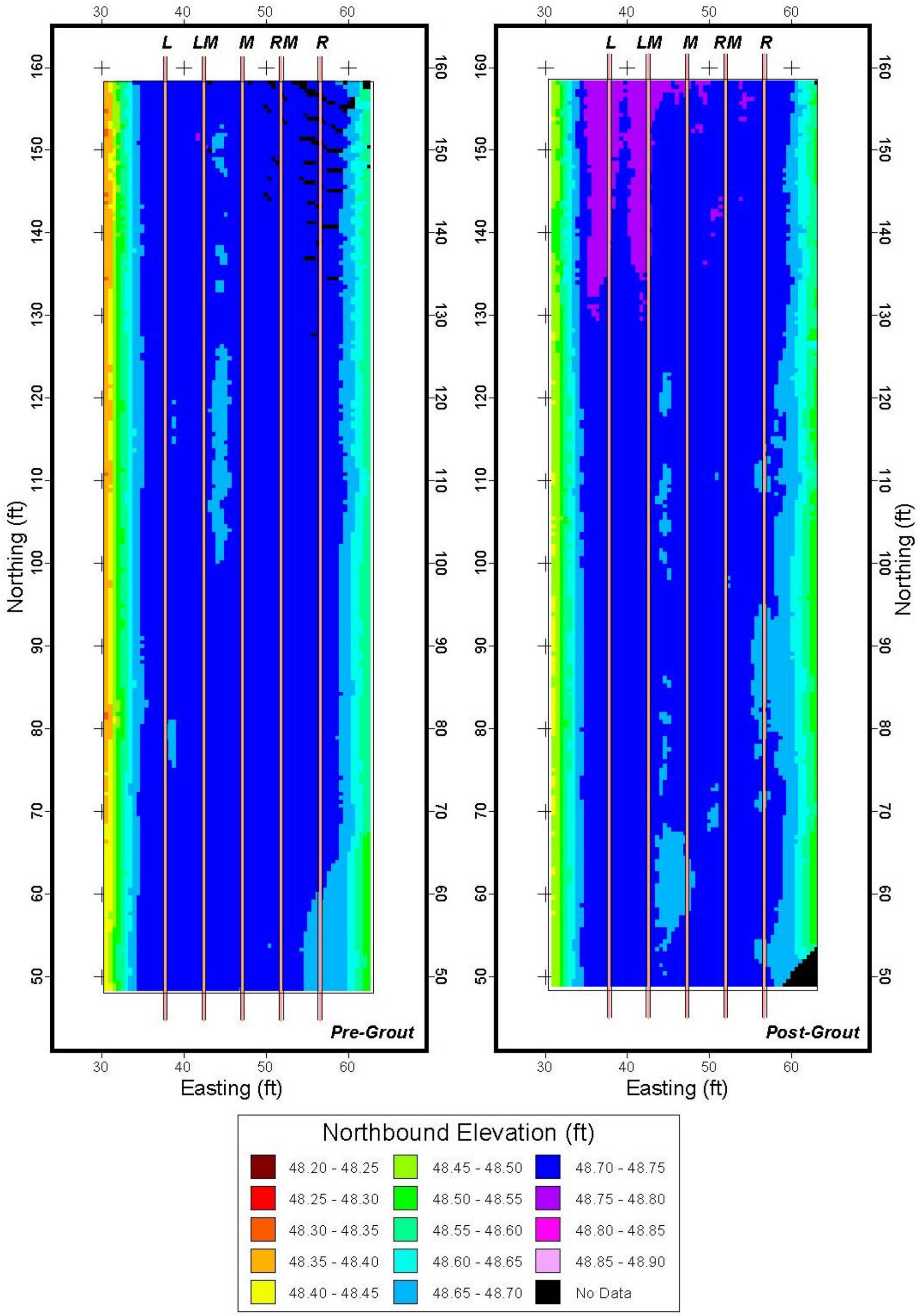

Figure 5. Pre- and post-grouting scanner elevations of the northbound lanes. Buried pipelines are located at approximately Northing Stations 70 and 130.

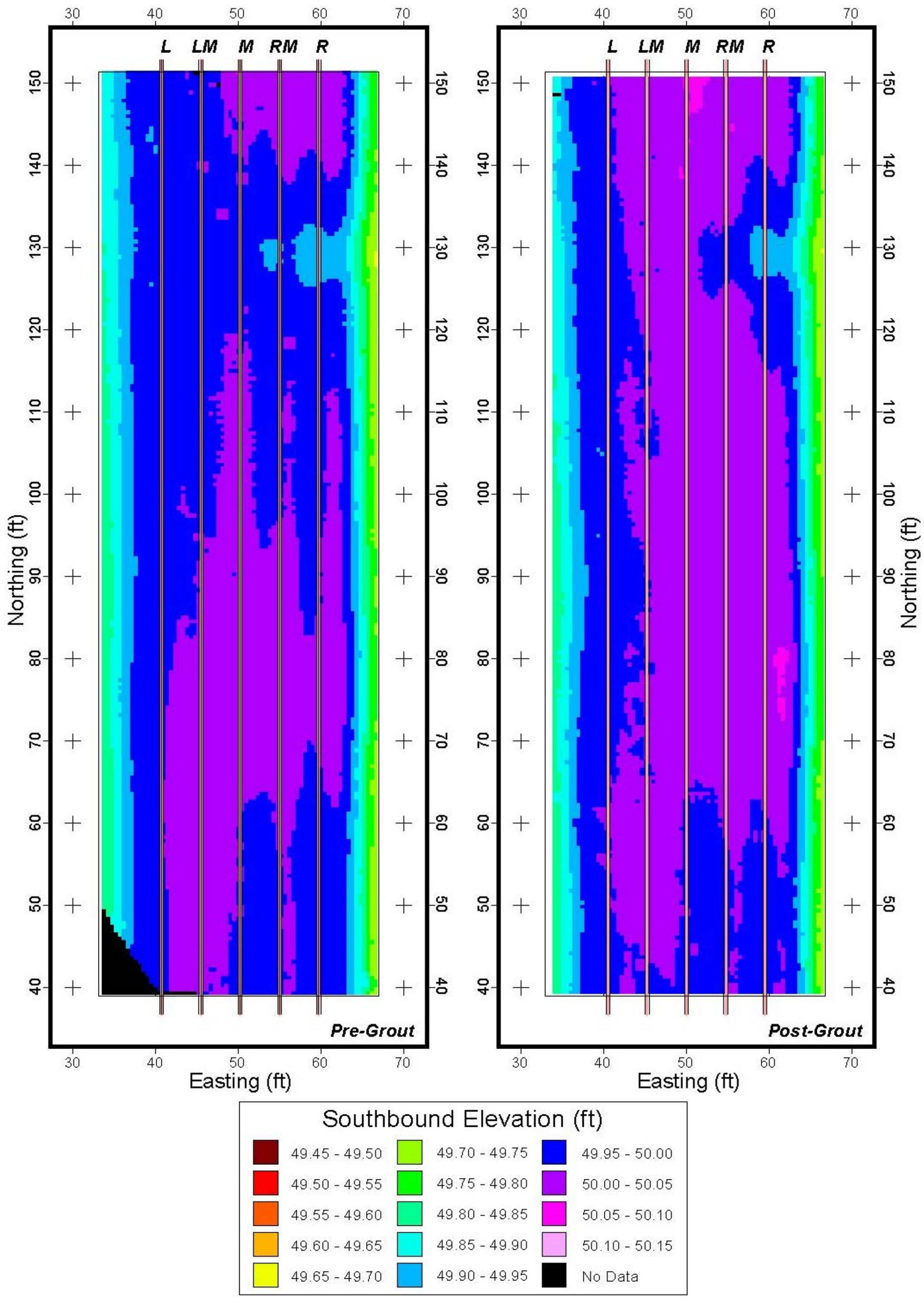

Figure 6. Pre- and post-grouting scanner elevations of the southbound lanes. Buried pipelines are located at approximately Northing Stations 70 and 130.

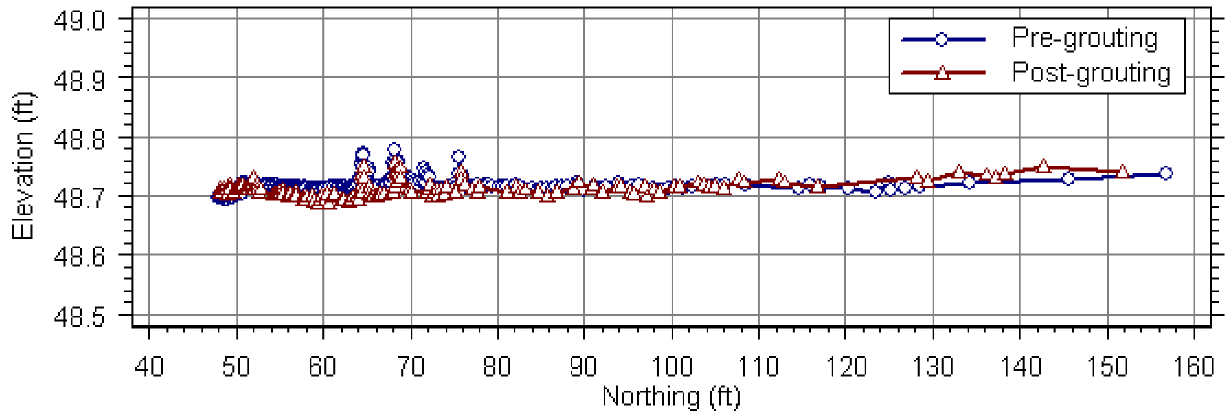

Figure 7. Pre- and post-grouting scanner elevations along middle profile on northbound lanes. Peaks between Northing Station 65 and 75 are raised lane-divider ‘dots’.

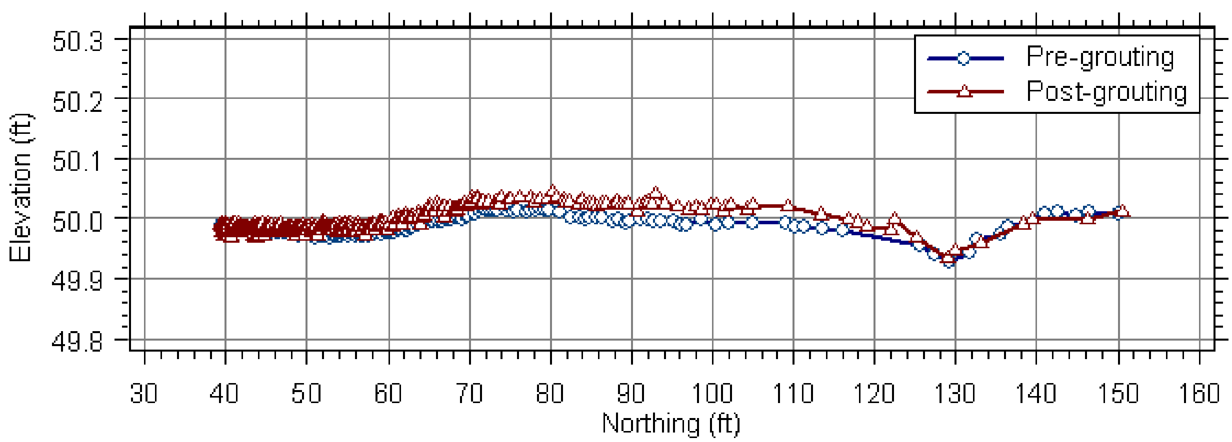

Figure 8. Pre- and post-grouting scanner elevations along right profile on southbound lanes.

“TOO LITTLE TOO LATE” OR
WHEN TO INCLUDE A GEOLOGIST IN HIGHWAY PROJECTS

By Albert F. Meijboom, PhD and Alan Barry Nelson, PG

INTRODUCTION

Geologists and engineers have co-existed and, in some cases, even spoken to each other during highway projects for many decades. However, even in the new millennium, geologists still face limited participation in the early planning phases of new highway projects. Too often, geologists are not involved in a project until after samples have been collected, rather than during route selection or even more importantly during the development of the project’s subsurface exploration and boring plan. A geologist brings a unique understanding to the project as part of the design team during the critical initial stages when changes are still allowable and often more economical than after surveying and drilling have been completed. Otherwise, a geologist may be forced to rely too heavily on outside resources and his faculties as a scientist to complete the tasks required of him.

However, due to today’s procurement rules, many large State/Federal funded highway projects are divided amongst several professional and technical firms acting as sub- contractors, each responsible for their own role of work with little initial discussion taking place between the participating sub-contractors. A recent project serves as an example. The owner was a State Division of Highways who, in conjunction with the road design engineer planned a geotechnical investigation along a 22,000-linear foot road corridor for the construction of a mountainous multi-lane expressway through the Appalachian plateau province. The project originally included some 160 borings. A work plan for the exploration drilling phase was established that determined the number, location, depth and type of borings that would serve as the data base for all geological interpretations and recommendations without the input from the geologist of record. Unfortunately, because of this method of procurement, ETPA was only retained as sub-contractor after the locations of the approximately 160 borings had been surveyed and access roads cut to the drill sites. Notwithstanding the late involvement, ETPA managed to provide detailed recommendations concerning the large cut benches along the road corridor and other pertinent information relating to the stability of the road’s subsurface.

EXPLORATION DILEMMA

Engineering Tectonics P.A. (ETPA) became involved in a large highway project once some 160 drilling locations had been laid out and drilling had started. As geotechnical sub-contractor, ETPA was asked to provide bench designs for both sides of the road cuts; however, there were several instances where we had borings on one side of the road, while another set of borings was further up the road rather than on the opposite side of the bench. A good bench design hinges on intimate knowledge of the strata that will be intersected by the road cut. The lithology determines the angle of the cut as well the width of the bench. Instead of providing the road design engineer with a simple “generic” bench design for areas where we lacked adequate core data, we felt that it was possible to obtain this detailed knowledge by geological correlation using depositional sequence analysis.

As the construction of this roadway involves large road cut areas, many borings were located far from the centerline of the road to cover the areas to be excavated. Unfortunately, this resulted in a paucity in the number of borings in the immediate vicinity of the roadway. While some borings were drilled only 400 feet apart, several were between 700 and 900 feet apart, which begins to stretch the reliability of structural and stratigraphic interpretations when engineering rock properties are being evaluated for slope stability. In addition, several borings were not drilled deep enough to provide overlap of geological strata with adjacent borings, which in a sedimentary environment, does not provide the geologist with a great confidence level. The ancient deltaic sedimentary environment identified in this area is typified by sudden influx of sandstone channels eroding older mudstone or silt deposits. In this case, moderately wide (100-foot) channels can be easily overlooked without a detailed knowledge of the paleo-environment, caused by the large distances between borings and their shallow depths which do not overlap.

THE SOLUTION

In order to overcome the limited information provided by the cores, ETPA augmented geological information by studying sedimentary rock outcrops in the area and through research into historical geological reports. ETPA geologists were able to reconstruct a local sedimentary model for the area that was then used to compile reliable vertical and lateral correlations across large areas.

Using our detailed core descriptions, we tediously recreated the paleo-environment through careful correlation of the strata. For instance, we established the spatial relationship of the strata between a single set of borings to recreate the paleo-environment relating to a specific site; this sedimentary model was then extrapolated in the direction of the next set of borings drilled further down the road on the other side, where a similar reconstruction of the stratigraphic sequence had been completed. By covering the road corridor with a multitude of geological fence diagrams, we were able to complete large distance extrapolations and connect several sets of correlations. In this manner, we subsequently obtained a better understanding of the spatial distribution of the various lithologies present along the road corridor, than had we merely relied on the snapshot information each single boring provided.

Notwithstanding the limited core information due to the scarcity of borings, ETPA was thus able to provide detailed recommendations for the large cut benches along the road corridor. ETPA also recommended a review of road sections where the proximity of old mine workings near the ground surface could cause significant road base instability.

PREFERRED PLANNING INVOLVEMENT OR A BETTER IDEA

While this methodology provided a sound analysis, a more traditional exploratory approach that was based on an understanding of the geological environment rather than compliance with procurement procedures, could have resulted in a higher degree of confidence in the recommendations and design data developed from the geological assignment.

Even though ETPA completed our assignment, had we been included in the initial phase of exploration planning, alternative methods such as geophysics and detailed geologic mapping using both remote sensing and field surveys could have been included in the scope of work, rather than relying so heavily on broadly spaced cores for the geological assessment. In turn, this would have maximized the amount of critical geologic information obtained and may have minimized the overall project costs.

Because the geologist is not involved in the exploration planning, we think there is a flaw in the process that puts the geologist in a position to make responsible geologic decisions based on data that were collected prior to his involvement. A similar analogy is a police force being asked to solve a crime, without being involved in the data collection at the crime scene and therefore do not know whether important data were overlooked.

SUGGESTED PROCEDURE

We believe that the following procedures should be implemented on future projects in a similar geological environment in order to maximize the information obtained for the least cost.

The geologist of record should be involved in developing the exploration plan from the start as soon as the route has been selected. In conjunction with the engineering design firm, the geologist will determine the optimum boring locations based on the regional geological model applicable to the area. Relative boring locations should not depend only on the sedimentology of the area, but should also take into account known paleocurrent directions in similar sedimentary terrains and sedimentary sequence analysis.

At each planned road cut, two borings should be located at the highest cut; these borings should be cored to an elevation beyond the lowest elevation of the road cut. These two cored borings can then be geophysically logged to provide typical “fingerprints” of the lithologies in the area. Additional open-hole borings can then be placed at a closer spacing than more costly cored borings would allow. All of the open hole borings should be geophysically logged. All cores will be analyzed, described in detail, and compared with the relevant geophysical logs. A trained geologist will then be able to analyze the geophysical logs from the open-hole borings and determine the succession of the lithologies in those borings. From our project, we found that a limited number of cored borings provide enough information with regards to the jointing nature of a typical lithology, when augmented by field observations in the area where joint patterns and directions can be measured at outcrops.

This approach allows for a reduction in expensive core drilling and yet provides better lateral and vertical lithological control of the strata necessary to provide reliable information for critical bench designs.

SUMMARY

In many cases, the work plan and scope of the field drilling phase of geological explorations and geotechnical investigations that provide highway engineers with critical structural and foundation design data are developed without input from the geologist or geotechnical engineer charged with evaluating the basic data without consideration of the local geologic framework. This occurs because of various procurement practices that do not consider it necessary for the drilling and field phase of the work to be conducted by the professional firm that will evaluate the field data. Often this is done to allow drilling contractors without a professional technical staff to "bid" on the drilling rather than using qualifications based selection to employ a full service professional geological/geotechnical consulting firm. This method of splitting the drilling and field data collection places a needless burden on the geo-professional and rarely saves money. Therefore, we believe it is time that this practice of separating these aspects of the subsurface investigation cease because it handcuffs the investigator to an investigatory plan which he has had no input and robs the project of the geo-professionals experience in gathering more data from a variety of exploratory methods for less cost to the project.

PRELIMINARY FINDINGS ON THE SEPTEMBER 16, 2004 DEBRIS FLOW AT PEEKS CREEK, MACON COUNTY, NORTH CAROLINA

LATHAM, Rebecca S.[1], WOOTEN, Richard M.[1], REID, Jeffrey C.[2]

ABSTRACT

Heavy rains from the remnants of Hurricane Ivan triggered the Peeks Creek debris flow in Macon County, North Carolina at about 10:10 p.m. on September 16, 2004. The debris flow began just below the top of Fishhawk Mountain at elevation 4,420 ft and traveled approximately 2.25 miles dropping 2,200 ft in elevation to the Cullasaja River. Five people were killed, fifteen homes were destroyed, and two people were seriously injured by the slope movement.

Since 1901, there have been fourteen recorded, landslide-producing storm or hurricane occurrences (including Hurricanes Frances and Ivan in 2004) in western North Carolina. Several of these have triggered landslides in Macon County including the Fishhawk Mountain area. In 1876, debris flows originated on both the northern and southern sides of Fishhawk Mountain. In 1995, Hurricane Opal triggered a debris flow in the Poplar Cove area of Macon County. Hurricanes Frances and Ivan triggered numerous slope movements in Macon County in September, 2004. Colluvial deposits exposed along Peeks Creek indicate prehistoric slope movements may have occurred in the same area as the recent major debris flow.

Remnants of Hurricane Frances produced rainfall totals up to 15 inches in portions of western North Carolina. Eight days later, remnants of Hurricane Ivan dropped 9 inches of rainfall across the region. These heavy rainfalls in combination with the thin (<6.5 ft), colluvial soil in sharp contact with the steeply dipping bedrock surface ($35 – 55^{o}$) created a setting conducive for slope failure. Subparallel striations on the bedrock surface in the initiation zone indicated that the initial movement may have been a debris slide that quickly mobilized into a debris flow.

Cross sections measured across the debris flow track provided information to calculate estimates of velocity and discharge. Velocity estimates ranged from 20.3 mi/h to 33.2 mi/h. Discharge approximations ranged from 20,800 cfs up to 45,000 cfs. Velocity and discharge values fluctuated over the length of the debris flow track due to changes in stream gradient and streamflow contribution from side channels.

Studies to analyze the cause of the Peeks Creek debris flow will continue. Preliminary GIS-based slope stability assessments indicate other areas in the watershed may be susceptible to debris flows. Soil testing will improve the soil parameters (such as cohesion, angle of internal friction, etc.) used in the slope stability computer models to further refine the delineation of areas of potential slope movement.

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INTRODUCTION

Macon county is located in southwestern North Carolina on the North Carolina-Georgia boundary. Franklin, the county seat, is located approximately eight miles northwest of the debris flow. The Peeks Creek community is situated in a mountain hollow along the banks of Peeks Creek near its confluence with the Cullasaja River (figure 1). A northwest to southeast trending chain of mountains ranging in elevation from 3,800 feet to 4,746 feet at the top of Fishhawk Mountain is on the southwest side of the community.

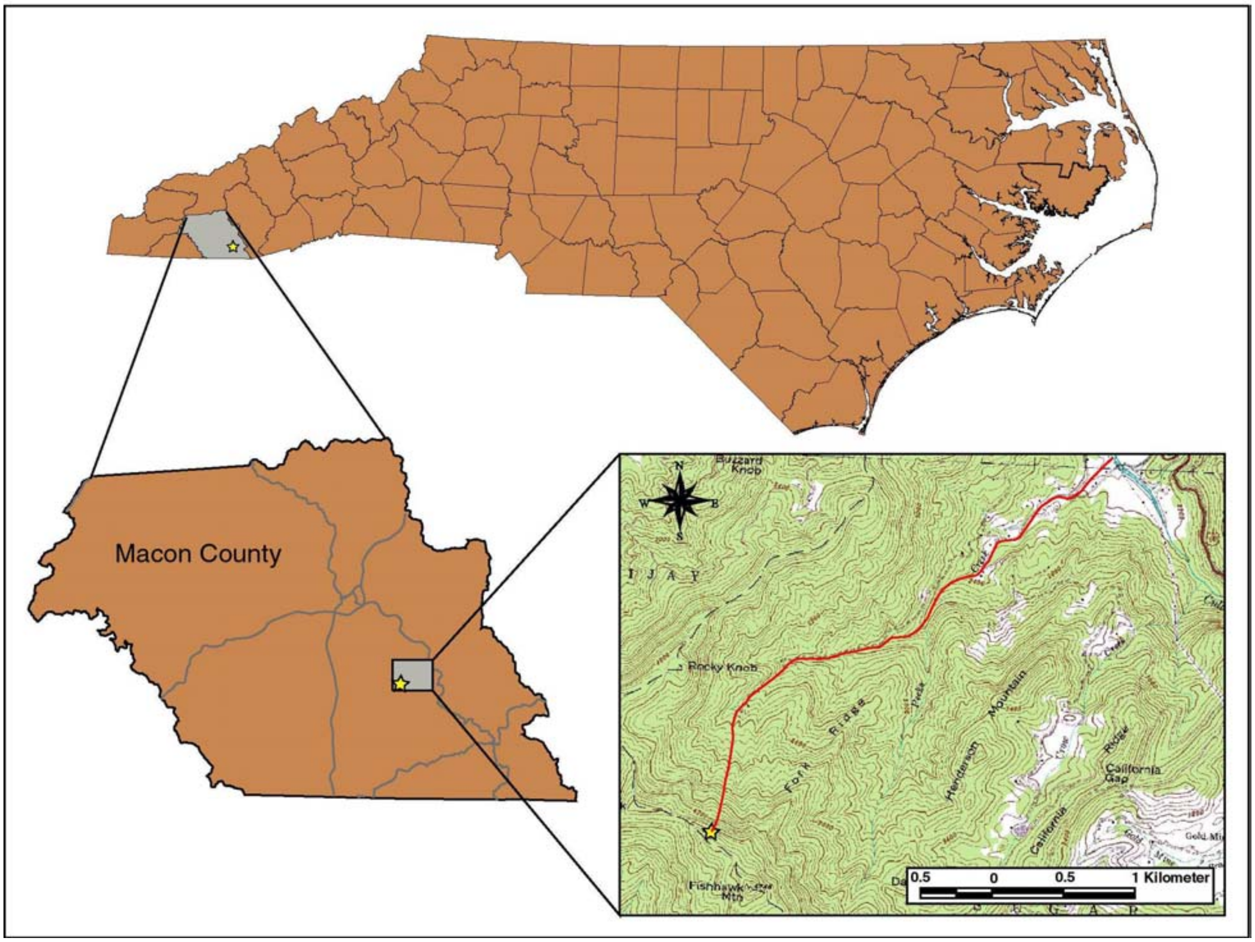


Figure 1. Map showing the location of Macon County and the Peeks Creek community. Red line on topographic map inset indicates the debris flow track. Track is approximately 2.25 miles long.

Back-to-back hurricanes struck western North Carolina in 1916 and 1940 and set off numerous slope movements. A similar situation occurred in September, 2004 when the remnants of Hurricanes Frances and Ivan tracked over the western portion of the state. These hurricanes produced over 30 inches of rain combined over parts of western North Carolina triggering over 90 slope movements. The Peeks Creek debris flow was triggered as the remnants of Hurricane

Ivan passed over western North Carolina about 10:10 p.m. on September 16, 2004. The debris flow killed several people and generated property losses of over $1.6 million (Cunningham, 2004). The debris flow began near the top of Fishhawk Mountain at an elevation of about 4,420 feet and traveled approximately 2.25 miles downslope to the confluence of Peeks Creek with Cullasaja River at elevation 2,220 feet.

The North Carolina Division of Emergency Management (NCDEM) asked the North Carolina Geological Survey (NCGS) to provide on-site technical assistance as part of Phase I of a cooperative multi-year, geohazards project. Local officials concerned with the safety of people involved in the rescue and recovery operations in the Peeks Creek community requested NCGS to provide general stability assessments in the drainage. Macon County Emergency Management (MCEM) later requested that the NCGS participate in a multi-agency task force to address what happened, safety concerns for residents remaining the Peeks Creek community, how to mitigate the damage, and how to determine where and when this might happen again in Macon County. NCGS staff began preliminary studies of this slope failure in an attempt to answer these questions.

To date, the NCGS has gathered information including historical data on slope movements in Macon County soil and rock data, precipitation data, and general characteristics of the debris flow/debris flow track. Initial velocity and discharge estimates have been calculated with a maximum velocity and discharge of 33.2 miles per hour (mph) and 45,000 cubic feet per second (cfs), respectively.

Studies have also begun to assess landslide susceptibility in the Peeks Creek watershed using Stability Index Mapping (SINMAP), a Geographic Information System (GIS)-based slope stability model (Pack, Tarboton, and Goodwin, 1998). Soil tests are being conducted in order to refine parameters used in the program as well as approximate a recurrence interval for slope movements.

HISTORICAL DEBRIS FLOW EVENTS

There are fourteen recorded, landslide-producing hurricanes or storms that have affected western North Carolina since 1901 (Scott, 1972; Neary and Swift, 1987; Clark, 1987; Pomeroy, 1991). Currently, there are seventeen slope movements in Macon County documented in the NCGS slope movement database. Figure 2 shows the locations and dates of some of these slope movements. Clingman (1877) reported several "waterspouts" descending upon Fishhawk Mountain on June 15, 1876. The one described in detail traveled down the southwest side of the mountain and, in description, resembles the September 2004 debris flow at Peeks Creek. He also tells of one on the northeast side of Fishhawk in the same watershed as the September 2004 debris flow. Any evidence of such a feature, however, is obscured by thick vegetation.

Local officials reported that residents of the Nickajack community just north of Peeks Creek had ancestors who described debris flow(s) in the area approximately 125 years ago. This would nearly correspond in time with those described by Clingman. A slope movement was also noted by Clark (1987) in the Burningtown area in northern Macon County. This event, described as a "waterspout," occurred on July 30, 1928.

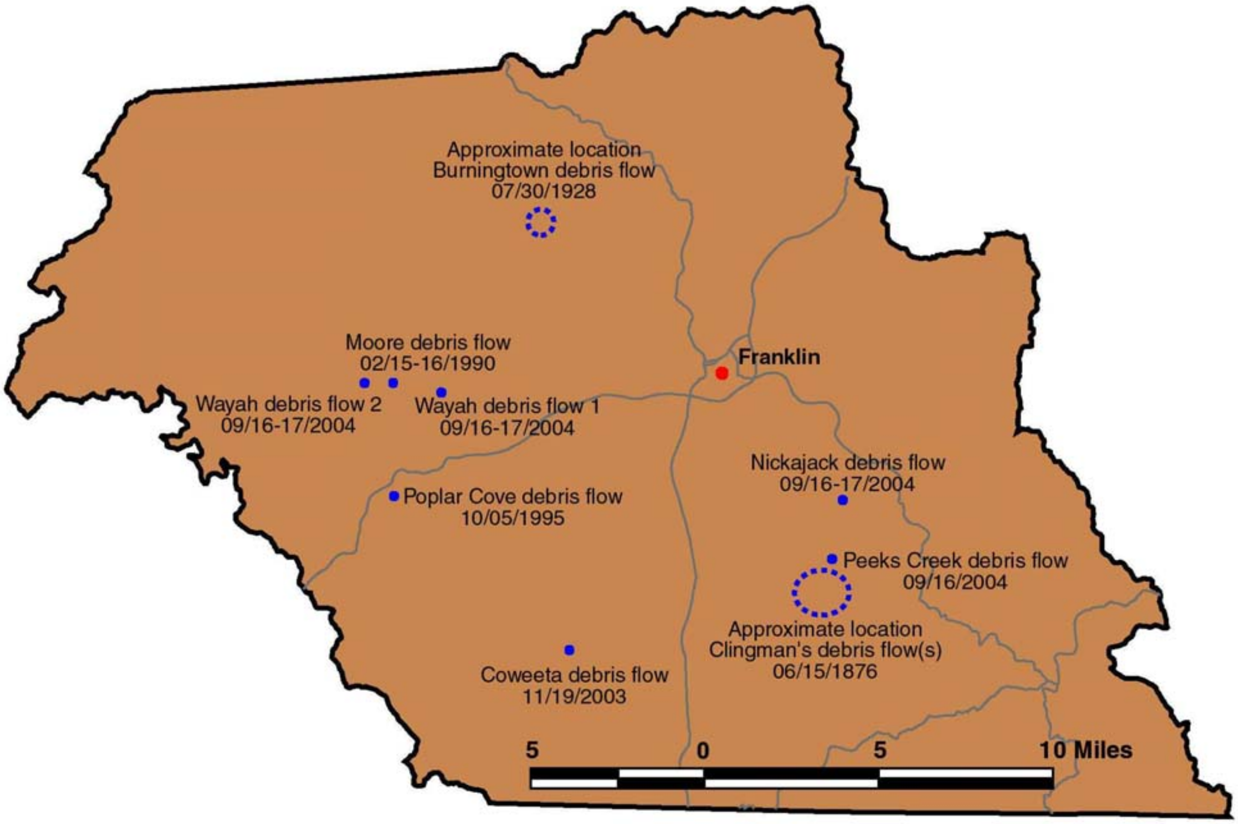


Figure 2. Map showing the locations of historic slope movements in Macon County. Franklin, the county seat, is shown by the red dot.

In more recent times, heavy rains compounded by high antecedent moisture conditions triggered a debris flow above Wayah Road in Macon County on the morning of February 16, 1990 (Moore debris flow). This 1,500-foot-long slope movement damaged an unoccupied home, deposited debris on Wayah Road, and threatened to cause sedimentation into Wayah Creek, a water source for the town of Franklin (USDA Forest Service, 1990). Heavy rains from Hurricane Opal triggered the Poplar Cove debris flow on October 5, 1995. This slope movement damaged multiple forest roads and deposited sediment in at least two creeks that feed into a water source for the town of Franklin (Wilkins, 1995). Two debris flows triggered by the remnants of Hurricane Ivan occurred above Wayah Creek (near the town of Franklin) less than a mile apart. Both of these debris flows originated on unmodified slopes, blocked the road with debris, and deposited sediment into Wayah Creek. Wayah debris flow 1 destroyed a barn at the toe of the slope. A small debris flow/blowout occurred in the Nickajack community near the reported location of the historical slope movement described above. This blowout sent minor amounts of sediment onto a private road and produced minimal damage.

GEOLOGY

The Peeks Creek debris flow initiated in the Middle to Late Proterozoic metagraywacke and minor biotite-muscovite schist of the Otto Formation as mapped by Lamb (2001). A tonalite dike cross cuts the metagraywacke in the source area of the debris flow. An exfoliation fracture defines the plane of failure in the headscarp, strikes 283-322^{o}, and dips 35-56^{o} northeast. The debris flow then traveled through the biotite-muscovite schist dominated portion of the Otto Formation as well as the biotite gneiss and metagraywacke of the Tallulah Falls Formation. Multiple pegmatites are exposed along the track as well as amphibolites and minor metaconglomerates. Rock outcrops in the debris flow channel are freshly exposed and have a visually fresh state as defined by the Unified Rock Classification System (Williamson, 1984). Foliation strikes northeast to southwest and dips 80-90^{o} southeast to 40-90^{o} northwest. Mesoscopic folds and ptygmatic veins are exposed along the whole length of the debris flow track.

Surficial deposits vary along the length of the debris flow track. The USDA Soil Survey Map of Macon County (1996) defines the soil in the headscarp as the Cleveland-Chestnut-Rock outcrop complex which is comprised of sandy loam to cobbly sandy loam approximately 17-36 inches thick. In the initiation zone, soils are generally less than three feet thick and are in sharp contact with the underlying bedrock. Four soil samples were collected from the headscarp for gradation, Atterberg, and triaxial tests. Tests indicate that these samples range from sandy silt to silt/organic silt. In the upper portion of the debris flow track, at least two generations of ancient debris flow deposits are exposed. These consist of crudely imbricated, weathered boulders in a silt matrix. In the lower third of the debris flow track, preexisting stream deposits are concentrated in the channel and colluvium is located on the side slopes.

PRECIPITATION

It is difficult to obtain accurate measurements of precipitation associated with storm events in the mountains of western North Carolina as storms are often isolated to areas without monitoring equipment or localized heavy rainfalls are restricted to very small regions. Weather stations are also often located in valleys or low lying areas (e.g., the forecast office for southwestern North Carolina is located at the National Weather Service office in Greer, South Carolina). Mountains can block radar signals originating in these locations thereby limiting their effectiveness and accuracy.

Elevation plays a role in precipitation amounts. Figure 3 shows a plot of precipitation versus time during the September hurricanes and rainfall measurements at various elevations throughout Macon County. The high elevation gauges recorded much higher precipitation amounts than the low elevation gauges, indicating that rainfall was higher at the initiation zone for the Peeks Creek debris flow (elevation 4,420 feet) than the lower elevation rainfall gauges recorded.

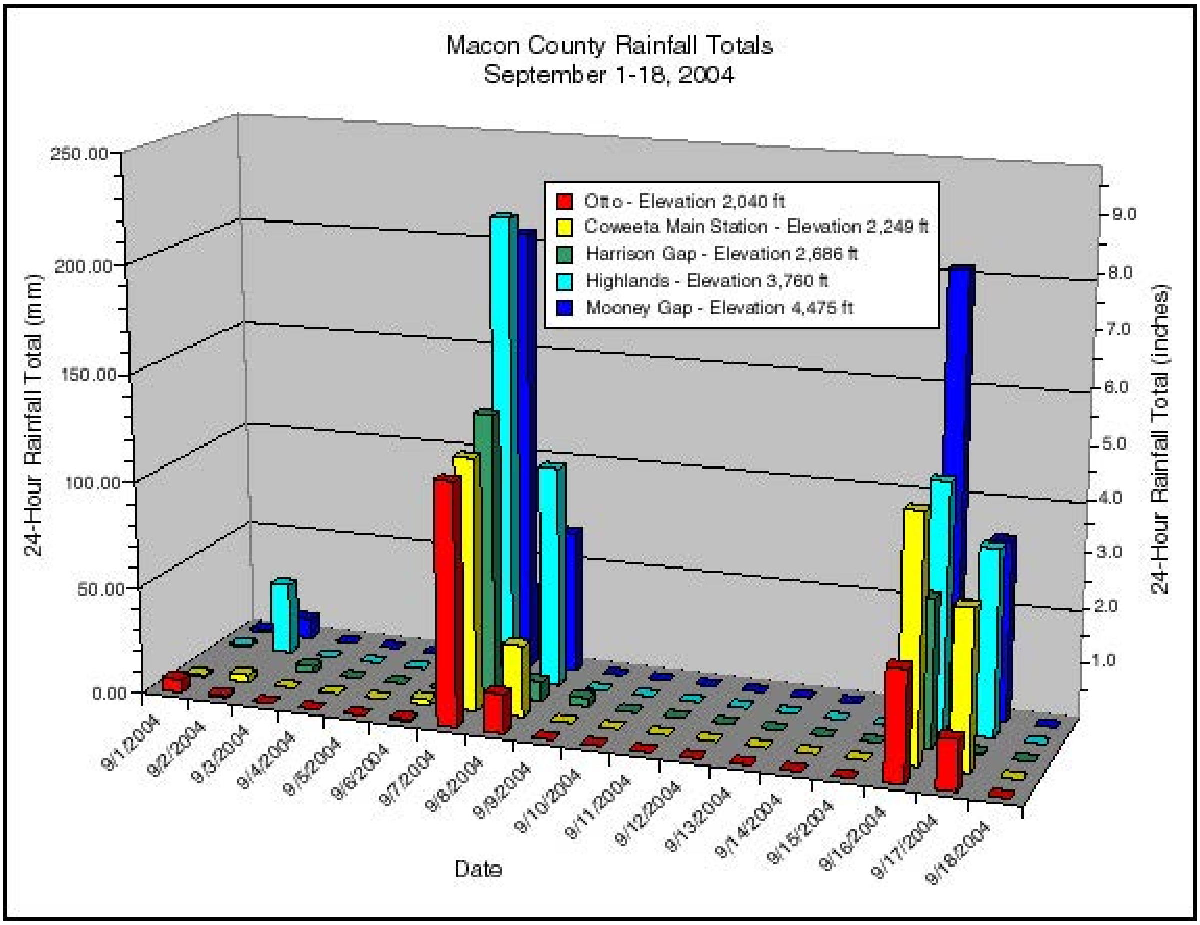


Figure 3. Graph showing rainfall totals collected from rain gauges at various elevations in Macon County over an eighteen day period. Elevation of rain gauge increases from front to back in chart. Note the much higher precipitation values recorded at the high elevation rain gauges. The debris flow began at elevation 4,420 feet near the elevation of the Mooney Gap rain gauge shown in dark blue. Rainfall data from the U.S. Forest Service Coweeta Hydrologic Laboratory and from the Integrated Flood Observation and Warning System (http://www.afws.net/).

The National Weather Service indicated that a powerful storm cell within the hurricane remnants passed over the Fishhawk Mountain area shortly before the debris flow. With a history of producing a tornado in Georgia, this storm crossed into Macon County and passed over Fishhawk Mountain between 9:40 p.m. and 9:50 p.m., approximately 25 minutes before the debris flow hit the Peeks Creek Community (Lamb, 2005).

THE SEPTEMBER 16, 2004 DEBRIS FLOW

The Peeks Creek debris flow most likely began as a debris slide. Parallel scratch marks caused by rock fragments in the colluvial soil moving over the bedrock surface indicate that movement was in one direction as opposed to the turbulent motion associated with a debris flow. The debris slide was approximately 75 feet wide, 350 feet long, and about 3 feet thick. It quickly began to mobilize into a debris flow as water was added to the system. Erratic marks were observed on bedrock in the debris flow track, and the material was acting like hyperconcentrated streamflow (indicated by the superelevation angle) at a cross section performed approximately 1700 feet below the headscarp. Figure 4 shows the initiation zone (also point 1 on figure 5).

Figure 4 (above). View from the headscarp looking down the debris flow track at the exposed bedrock surface where the debris slide initiated. Arrow points to patch of colluvium that was not removed by the initial slide.

Figure 5 (at right). Aerial view looking southwest of the Peeks Creek debris flow track. Numbered points refer to locations described in the text.

Near the top of the track, the debris flow cut through at least two generations of ancient debris flow deposits (point 2 on figure 5). These are identified by crudely imbricated, weathered boulders in an oxidized silt matrix. In this area the track is approximately 100 feet wide and 25 feet deep. Pending funding, carbon dating will be performed on several samples taken from these prehistoric debris flow deposits in order to approximate a recurrence interval for debris flows in this location.

Approximately one third of the way down to the Cullasaja River, the track gradient flattens into a relatively broad (approximately 250 feet wide) area where some deposition of material occurred (point 3 on figure 5). Imbricated boulders, some up to seven feet long, line the channel and have rerouted the flow of Peeks Creek into two channels.

Downstream from the broad, flat area is a 0.6-mile, steeper, more incised portion of the track that ends just upstream of the Peeks Creek community (point 4 on figure 5). The upper boundary of this section is near the confluence of the north fork of Peeks Creek with the main Peeks Creek channel. Also in this area are two side slope failures that originated from Fishhawk Mountain Road on the north side of the stream channel. The upstream failure appears to have been related to poor drainage along the road (point 5 on figure 5). Tension cracks and scarps continue to develop along this portion of the roadway. The downstream failure began as an embankment failure that quickly scoured the steep bedrock surface that leads down to the creek (point 6 on figure 5). It is possible that both these embankment failures created temporary debris dams along Peeks Creek before the main debris flow occurred, however any evidence for this was destroyed by the main debris flow.

Major damage to homes began below this section and continued along the last quarter of the track. The debris flow pushed several homes off their foundations, moving one house nearly 500 feet downstream where it collided with another home (figure 6) (Lewicki, 2004). A woman inside the house at the time survived the ordeal with no major injuries. Other homes were completely destroyed with, in one situation, a contorted vehicle the only thing left behind (figure 6). Five people were killed, and two others sustained serious injuries. Fifteen homes were destroyed, and most, if not all, of this property damage is not covered by insurance.

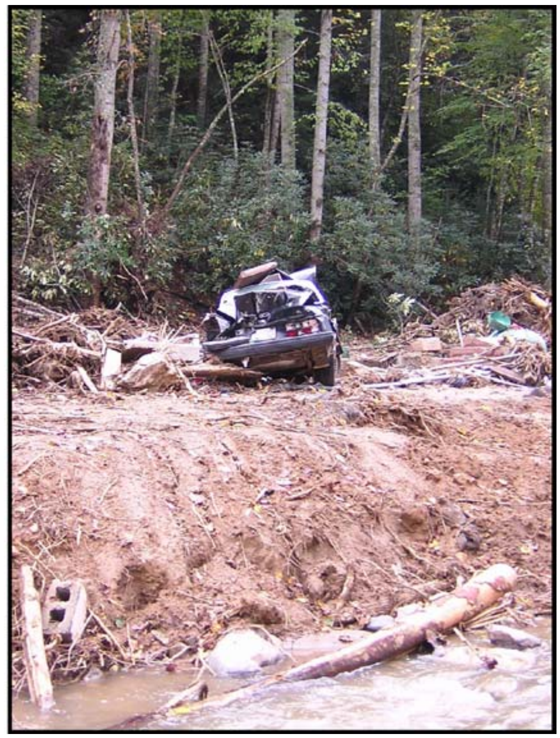
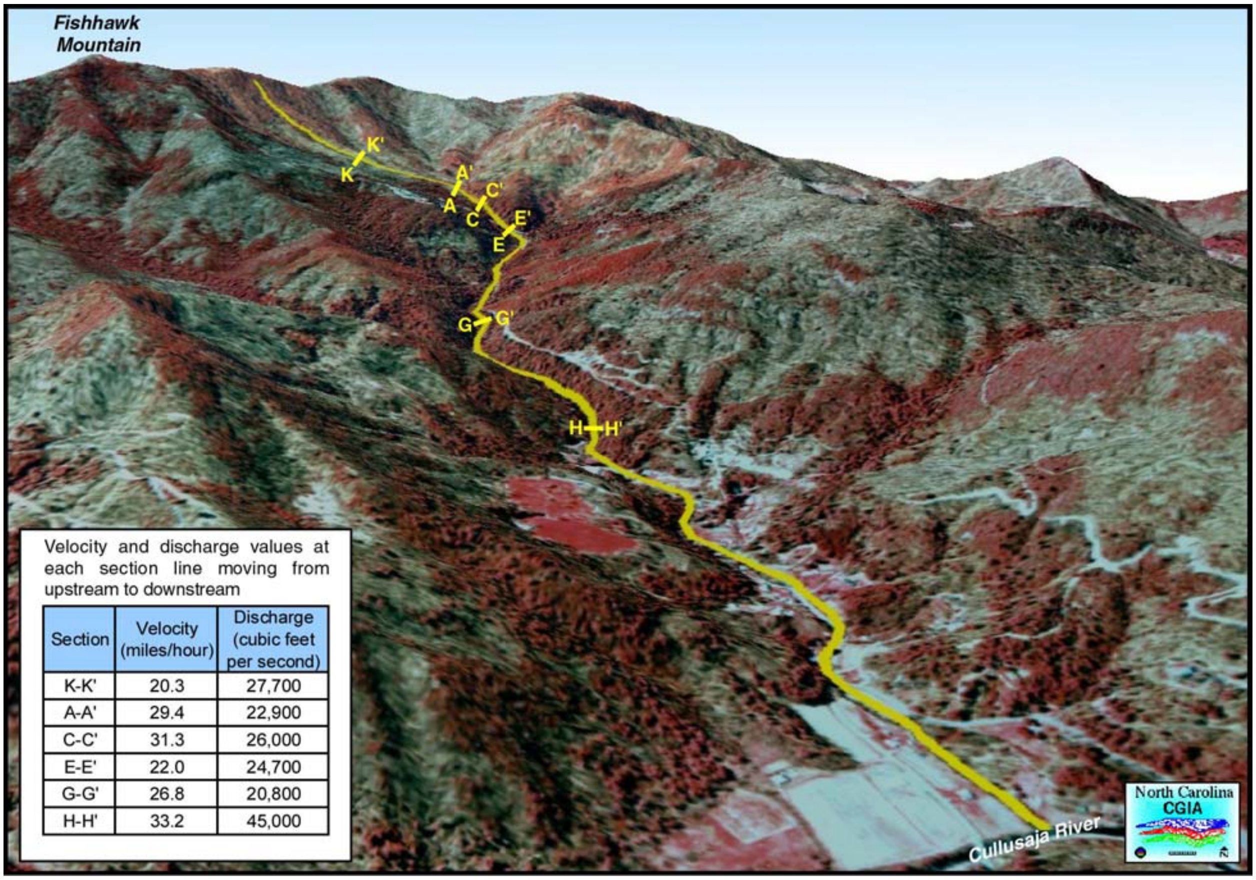


Figure 6. Photographs showing damage in the Peeks Creek community. Above, house indicated by arrow was moved 500 feet downstream by the debris flow. It stopped when it collided with the house on left. Right, contorted vehicle is all that remains at this location. Three people were killed here, and one was severely injured.

Just beyond the Peeks Creek community, Peeks Creek intersects the Cullasaja River. Near this location, deposits were limited to flood deposits from the river with little sediment deposited by the debris flow.

VELOCITY AND DISCHARGE CALCULATIONS

Estimates of velocity and discharge were calculated at six points along the debris flow track in order to gain a better understanding of why the debris flow as so destructive. According to Chen (1987), debris flows resemble hyperconcentrated stream flow and will bank as they travel around a curve in the channel. An estimate of velocity can be calculated given the radius of curvature of the stream channel, the superelevation angle of the material as it rounds the curve, the stream gradient, and acceleration due to gravity. If velocity is known, discharge can be calculated by multiplying the velocity times the cross-sectional area of the channel at that location (Fetter, 1994). Figure 7 shows the locations of the cross sections measured to calculate velocity and discharge and a table with the estimated values.



Section	Velocity (miles/hour)	Discharge (cubic feet per second)
K-K'	20.3	27,700
A-A'	29.4	22,900
C-C'	31.3	26,000
E-E'	22.0	24,700
G-G'	26.8	20,800
H-H'	33.2	45,000

Figure 7. 3-D oblique view of a digital elevation model with a 1998 color-infrared digital orthophoto quarter quadrangle (DOQQ) superimposed. Shown are the locations of six cross sections measured along the debris flow track to compute velocity and discharge estimates shown in inset table. Debris flow track is shown in yellow. Oblique view of superimposed DOQQ courtesy of North Carolina Center for Geographic Information and Analysis. All cross sections were performed by NCGS staff using 100-ft cloth tape, brunton compass, and clinometer.

Velocity and discharge values vary along the debris flow track from the initiation zone down to the Peeks Creek community (figure 8). The minimum velocity calculated is 20.3 mph at section K-K’, and the maximum velocity calculated is 33.2 mph at section H-H’. The minimum discharge calculated is 20,800 cfs at section G-G’, and the maximum discharge calculated is 45,000 cfs at section H-H’. In comparison, the Pigeon River caused extensive flood damage in the Canton area in Haywood County just west of Asheville during Hurricane Frances. Discharge on this river was 19,800 cfs at Canton on September 8, 2004 (USGS, 2004).

Fluctuations in velocity can be attributed to variations in the stream channel gradient as well as to contributions of flow from side channels. The debris flow will decelerate along a lower gradient reach and deposit more material. Contribution from side channels could increase the velocity by increasing the flow and reducing the viscosity of the material, depending on the volume of flow in side channels. The main debris flow may have temporarily dammed some side channels, limiting their influence on the velocity of the material. The velocity at section K-K' (20.3 mph) is probably lower due to the relatively smaller amount of water present and its proximity to the initiation zone (i.e., it is just beginning to accelerate). The velocity decreases at E-E' to 22.0 mph, probably from a change in stream gradient, as the channel gradient flattens just upstream from this section line (figure 9). It most likely continued to decrease as evidenced by deposition in this flatter portion of the channel until just upstream of section G-G' (velocity here is 26.8 mph) where the debris flow transitioned into a narrower portion of the channel.

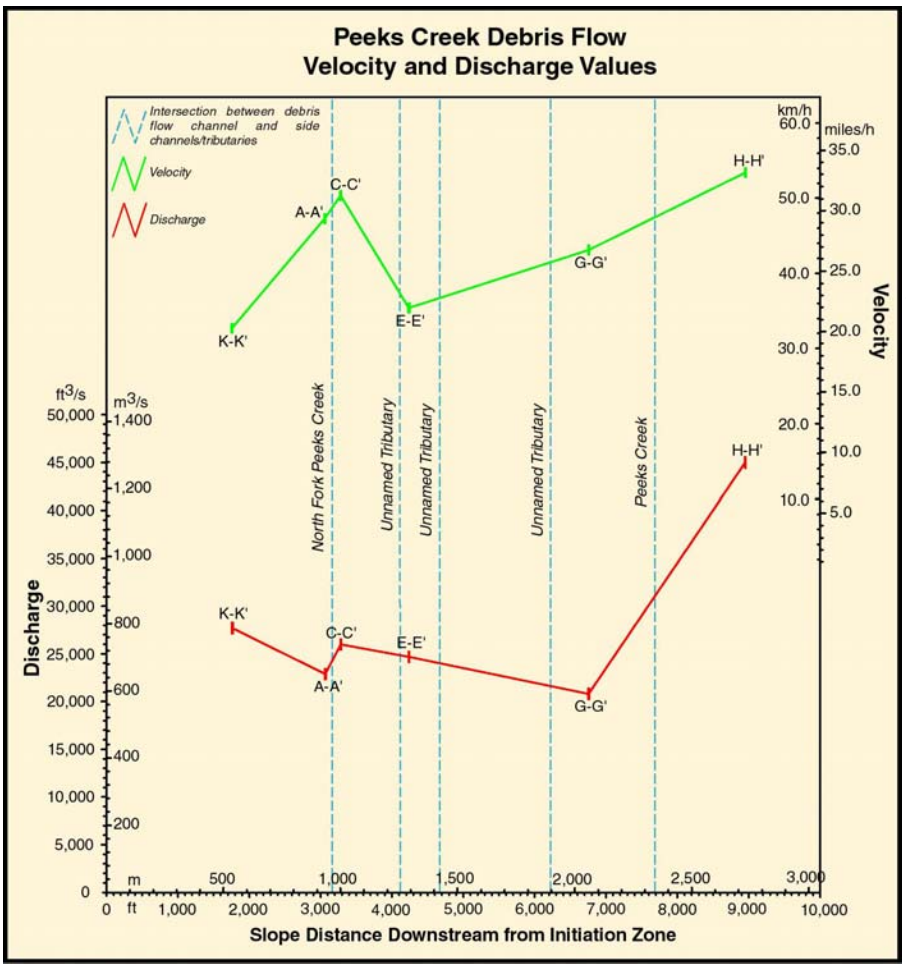


Figure 8. Graph showing a plot of velocity (shown at top in green) and discharge (shown at bottom in red) versus slope distance from the initiation zone. Locations of section lines are shown on each line. Blue dashed lines indicate the locations of side channels that may have contributed significant flow to the debris flow.

Discharge depends on the calculated cross sectional area of the debris flow and velocity. More scour will take place with higher velocity flow that can increase the cross sectional area. The depth to competent bedrock also dictates the amount of channel. Discharge is lowest at section A-A' (22,900 cfs) probably due to a flattening of the channel gradient (figure 9). Discharge increased at section C-C' (26,000 cfs) probably from the North Fork of Peeks Creek contribution. It continues to decrease downslope (minimum value of 20,800 cfs at section G-G') and then increases to a maximum of 45,000 cfs at section H-H' (figure 8). This is most likely due to significant contribution from the main channel of Peeks Creek, the steeper channel gradient, and steeper side slopes that produce greater amounts of runoff that increases the volume of water present.

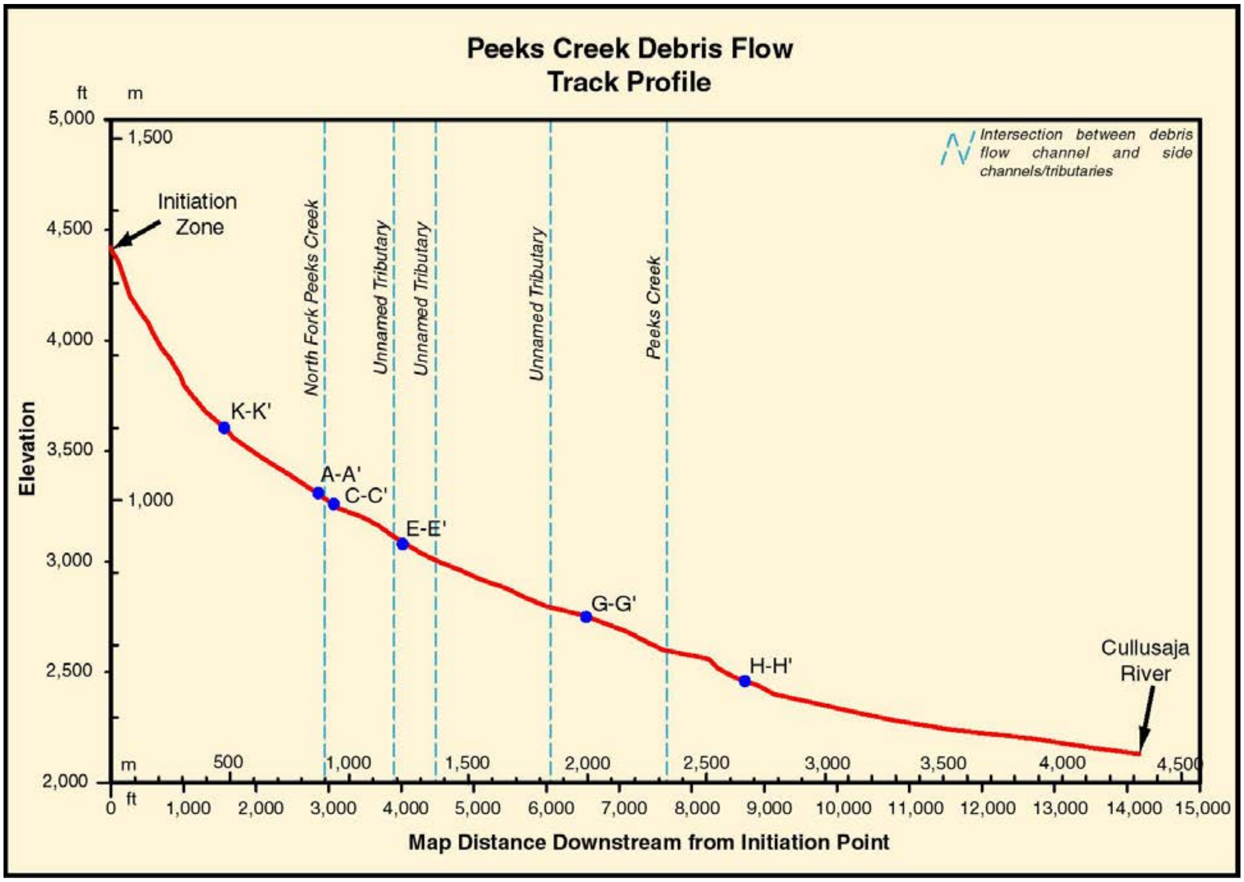


Figure 9. Graph of the debris flow track profile. Locations of section lines shown by blue dots. Blue dashed lines indicate the locations of side channels that may have contributed significant flow to the debris flow. This could account for the fluctuations in velocity and discharge estimates. Vertical exaggeration used to show subtle changes in the channels gradient.

CURRENT AND FUTURE STUDIES

Studies continue to further characterize and understand the Peeks Creek debris flow. Samples have been extracted from freshly exposed, ancient debris flow deposits. Pending funding, carbon dating will be performed on these samples in cooperation with James Madison University and Bucknell University to approximate a recurrence interval for debris flows along Peeks Creek. Preliminary GIS-based slope stability models were run for the Peeks Creek watershed to determine other areas susceptible to slope movements. SINMAP employs the infinite slope equation using a range of soil parameters including cohesion, soil density, angle of internal friction, transmissivity and recharge (Pack, Tarboton, and Goodwin, 1998). Laboratory analyses of soil samples including gradation, Atterberg Limits, and triaxial tests are in progress to refine SINMAP computer model input. Pending those results and the results of refined SINMAP runs, other stability models (e.g., LISA, Shalstab, etc.) may be employed to further assess the slope stability in the Peeks Creek watershed. Soil samples collected from the headscarp of the debris flow are being tested to refine these parameters. Once a reasonable model has been computed, areas more susceptible to slope movements will be field checked to determine if any past slope movements have occurred to verify the model.

CONCLUSIONS

Although why this particular slope failed at this particular time may never be known, much ground has been gained in understanding what happened and characterizing the nature of this debris flow. There has been a history of slope movements in Macon County and even in the Peeks Creek watershed. Heavy rains from the remnants of Hurricanes Ivan combined with high antecedent moisture conditions triggered the 2.25-mile debris flow that started a few hundred feet below the peak of Fishhawk Mountain. The debris flow exposed ancient debris flow deposits and in many places scoured the channel down to bedrock before entering the Peeks Creek community killing five people and destroying fifteen homes. Initial estimates of maximum velocity and discharge are 33.2 mph and 45,000 cfs, respectively. Studies will continue at Peeks Creek to further assess the potential for future catastrophic slope failures within the Peeks Creek watershed.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the following people: the Macon County Emergency Management and Sheriff's Department for their invaluable information, assistance and cooperation; the residents of the Peeks Creek community for their patience, cooperation and information during this difficult time in their lives; the North Carolina Center for Geographic Information and Analysis for graphics and GIS data and assistance; Bart Cattanach and Carl Merschat of the NCGS for their valuable knowledge and assistance in field work; the National Weather Service in Greer, South Carolina for rainfall data and information concerning hurricanes in western North Carolina; the United States Forest Service Coweeta Hydrologic Laboratory for rainfall data; the North Carolina Department of Transportation for conducting soil tests; the North Carolina Division of Emergency Management for continued funding to study geologic hazards in North Carolina.

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AN OVERVIEW OF THE NORTH CAROLINA GEOLOGICAL SURVEY'S GEOLOGIC HAZARDS PROGRAM – PHASE I

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ABSTRACT

The North Carolina Geological Survey (NCGS) has initiated a geologic hazards program funded through the North Carolina Division of Emergency Management (NCEM), with the long-term goal to develop a statewide geographic information system-based geologic hazards atlas. Phase I studies include statewide mapping of slope movements (landslides), abandoned mines, high hazard dams, sinkholes in Brunswick and New Hanover Counties, and compiling maps of earthquake epicenters in the region. Related studies include geologic and geologic hazards mapping along the North Carolina segment of the Blue Ridge Parkway, and cooperative studies with the North Carolina Aquifer Protection Section on arsenic in groundwater. The NCGS also assists other local, state, and federal agencies to help mitigate the impact of geologic hazards. Landslide and sinkhole information provided by the North Carolina Department of Transportation continues to contribute greatly to the geologic hazards program.

Geologic hazards occur in all geologic provinces in North Carolina, and include slope movements, subsidence above abandoned mines, limestone sinkholes, naturally occurring contaminants in groundwater (arsenic and uranium), acidic rock, earthquakes, high shrink-swell soil, and indirectly, high hazard dams. These hazards threaten public safety, transportation routes, and sustainable development statewide.

Damaging landslides occur frequently in the mountainous Blue Ridge Province and cost millions of dollars annually. Scores of people lost their lives from major flooding and debris flow events triggered by hurricanes that tracked across western North Carolina in 1916 and 1940. A debris slide-flow related to residential development on steep slopes destroyed a home and killed one occupant in Maggie Valley on December 11, 2003. Hurricanes Frances and Ivan in September 2004 triggered over 80 landslides in western North Carolina. Five fatalities occurred as a direct result of the Peeks Creek debris flow set off by Hurricane Ivan. Landslides triggered by Hurricanes Frances and Ivan damaged or destroyed over 25 homes. NCGS staff coordinated with NCEM during Hurricanes Frances and Ivan in compiling and relaying landslide information to the State Emergency Response Team. The NCGS web page (http://www.geology.enr.state.nc.us) contains information on landslides triggered by Hurricanes Frances and Ivan.

High hazard dams throughout the state continue to receive increased attention as a result of recent hurricane flooding. Subsidence and collapse above abandoned underground mines occurs mainly in the rapidly urbanizing, old gold mining districts of the Piedmont. Accurately locating the mine workings is difficult as most underground mining took place during the 19th and early 20th centuries, and reliable mine maps are rare. Arsenic in residual soil and groundwater is associated with gold and sulfide mineralization (acidic rock) in these areas. Uranium concentrations in groundwater >0.1 ppb occur mainly in igneous and metaigneous granitoids, and Triassic sedimentary rocks in the Piedmont. Sinkholes originate mainly in Tertiary

carbonate deposits in the Coastal Plain affecting rapidly developing coastal areas. Damaging earthquakes are rare in North Carolina, but previous major earthquakes in the New Madrid, Charleston, South Carolina, and Eastern Tennessee seismic zones would cause damage in North Carolina. Recurrence of major earthquakes in these zones may result in damage to structures in North Carolina and would certainly be felt.

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INTRODUCTION

Beginning in 2003 the North Carolina Geological Survey (NCGS), in cooperation with the North Carolina Division of Emergency Management (NCEM), began work on Phase I of a multi-year effort to develop a geographical information system (GIS)- based geohazards atlas for North Carolina. Once delivered to the NCEM the GIS data layers and other database information will used to help develop the State's hazard mitigation plan, and will be available to other state agencies and local governments for planning and hazard mitigation. Federal Geographic Data Committee (FGDC) compliant metadata accompanies all GIS data layers.

Phase I studies are complete and include GIS data layers with statewide information on landslides, abandoned and historic mines, high hazard dams, and earthquake epicenters, and locations of carbonate sinkholes in Brunswick and New Hanover counties. Close coordination with the North Carolina Division of Land Resources' Land Quality Section is required to produce and maintain currency of GIS data layers developed from the dams' database.

Flexibility in the Phase I program allowed the NCGS to respond to NCEM requests for technical assistance during and after the remnants of Hurricanes Frances and Ivan tracked over western North Carolina in September 2004. NCGS staff coordinated with NCEM during Hurricanes Frances and Ivan to compile and relay landslide information to the State Emergency Response Team. Additional work included technical assistance and reports on landslide hazards for Emergency Management officials in Watauga, Henderson, and Macon Counties.

Phase II studies, just now beginning, include continued work on maintaining currency of GIS data layers on landslides, abandoned and historic underground mines, high hazard dams, and earthquake epicenters, and expanded mapping of sinkholes in coastal counties along with field verification. Important new aspects of Phase II studies include development of: 1) GIS data layers showing the relative vulnerability of North Carolina's barrier island system to coastal erosion and flooding from hurricanes and storms; 2) GIS data layers that show the coincidence of geohazards with critical facilities, transportation networks, and associated infrastructure; and, 3) a geohazards section on the NCGS Internet site (http://www.geology.enr.state.nc.us) to include coastal hazards, landslides, sinkholes, abandoned mines, earthquakes, acid-producing rock, and geochemical hazards.

Naturally occurring contaminants in groundwater is a rapidly emerging issue, Elements of concern include arsenic and uranium that may pose a health hazard. The NCGS also cooperates with the North Carolina Aquifer Protection Section on the emerging issue of arsenic in groundwater. Future work may be necessary to address uranium in groundwater. Additional planned future work also includes mapping and documenting acid-producing rock and high shrink-swell soil.

LANDSLIDES (SLOPE MOVEMENTS)

Debris flows and other types of rapid landslides, such as debris slides and rockslides, occur annually in western North Carolina. Although landslides in the Piedmont and Coastal Plain are relatively infrequent, at least seven landslides, mainly on modified slopes underlain by Triassic sedimentary bedrock are documented in the Piedmont region. Known landslides in the Coastal Plain are restricted to steep bluffs along the Cape Fear River valley.

Along transportation corridors alone, costs related to landslides average in the range of $1-3 million annually, according to the North Carolina Department of Transportation (NCDOT). Direct costs associated with large landslides (e.g., the 1997 rockslide along I-40) can exceed $10 million. Indirect costs from the loss of business, tourism and transportation re-routing are extensive, but more difficult to estimate. Hazards and environmental impacts related to landslides include the threat to public safety, damage to roads and water supply facilities, and degradation of water quality from sedimentation. Figure 1 shows about 600 locations of slope movements and slope movement deposits currently in the North Carolina Geological Survey (NCGS) database.

Debris flows are a particularly dangerous type of landslide characterized by a mixture of soil, rock fragments, and water that moves rapidly down slope, usually without warning. Their high velocity and discharge makes them especially destructive. Studies of debris flows in North Carolina show that they can reach velocities of about 25-33 miles per hour (~37-48 feet per second), and can travel down slope over 2 miles (Wooten and Latham, 2004, and Latham and others, 2005a,b). These studies also show that computed estimates of discharge from debris flows can range from about 30,000 cubic feet per second (cfs) for a relatively small 2003 debris flow in Swain County, to about 45,000 cfs for the September 16, 2004 Peeks Creek debris flow in Macon County.

Widespread major flooding and debris flow events occurred in western North Carolina in 1901, 1916, 1940 and 1977 when hurricanes or tropical depressions tracked over the Blue Ridge Mountains, and dropped large amounts of rainfall on steep mountain slopes (figure 2). These storms can also contribute to catastrophic dam failures such as the Toxaway Lake dam failure in

1916 (Wooten, Carter and Merschat, 2003). Loss of life and significant property damage resulted from these events, even though the area was more sparsely populated at the time.

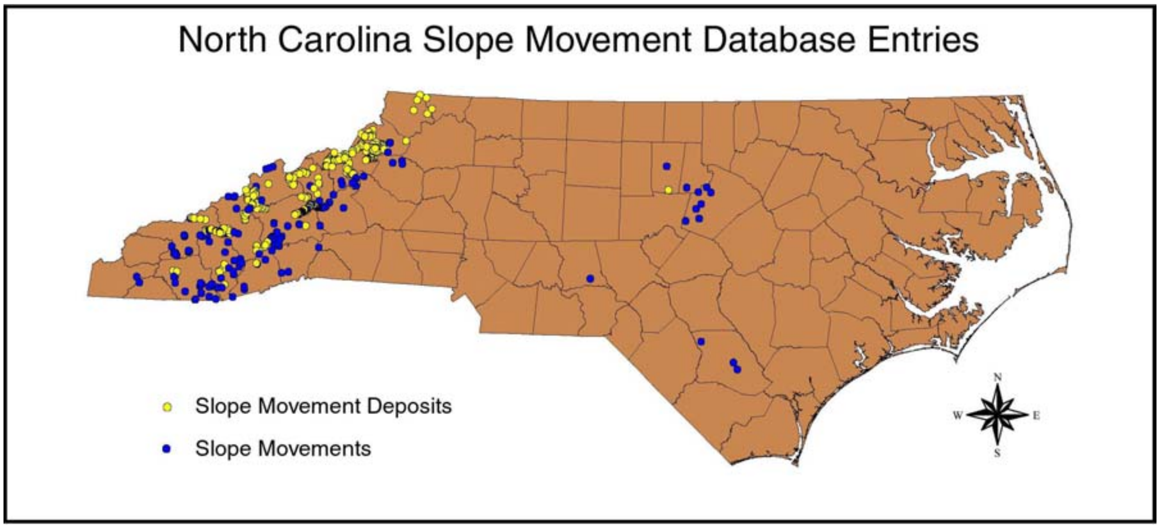


Figure 1. Map showing locations of approximately 400 slope movements and slope movement deposits in the NCGS database as of March 7, 2005. There are 90 entries for slope movements triggered by Hurricanes Frances and Ivan in September 2004; this number, however, is expected to increase.

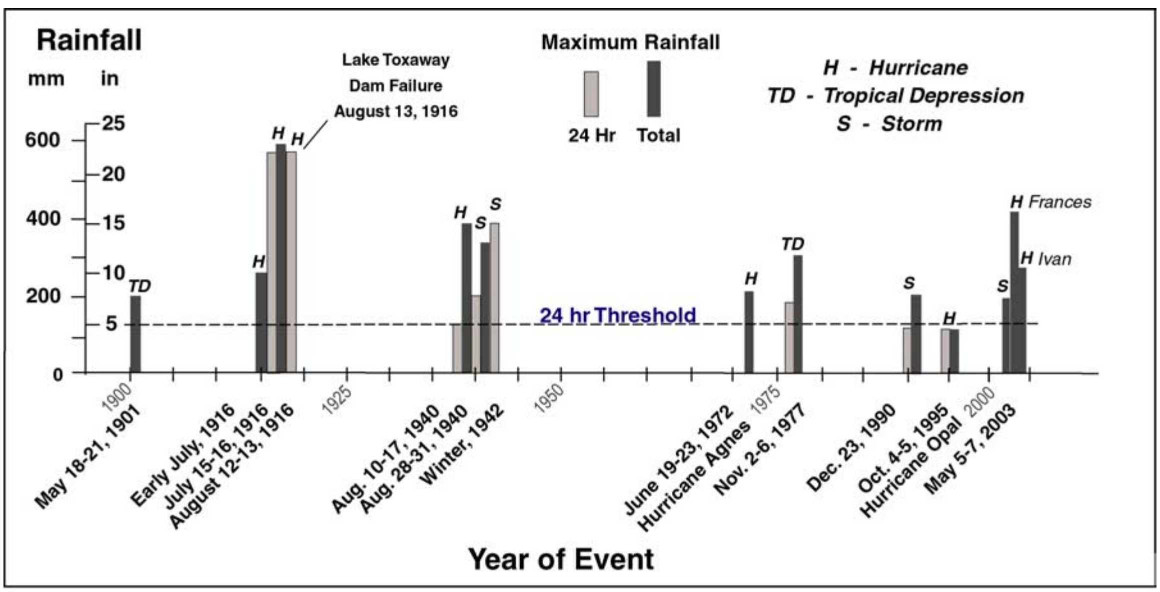


Figure 2. Chart showing rainfall associated with hurricanes and other storms that triggered landslides and flooding in western North Carolina. As a general rule, the 24-hour threshold line shows the minimum rainfall needed within a 24-hour period to trigger debris flows in the Southern Appalachians (Escher and Partic, 1982). The hurricanes of July 1916, August 1940, and Frances in Ivan in September 2004 set off hundreds of landslides causing loss of life and widespread damage in western North Carolina. Primary sources of information: U.S. Geological Survey (1949), Tennessee Valley Authority (1964), Scott (1972), Bailey and others (1975), Neary and Swift (1987); figure modified from Wooten, Carter and Merschat (2003).

Back-to-back storms in August 1940 triggered over one hundred slope movements in Watauga County, particularly in the Deep Gap area along the Blue Ridge Escarpment. The August 10-17, 1940 event affected most of the southeast and resulted in 26 deaths in North Carolina alone (Wieczorek and others, 2004). The sharp increase in mountainside development that has occurred since these earlier 20th century storms, places more people, homes, and infrastructure at risk from landslide hazards in western North Carolina.

Localized heavy rainfall not associated with tropical storms can also trigger landslides. A December 1990 debris flow in Swain County destroyed a house trailer and the chlorinator building for the Bryson City municipal water system (Wooten, 1998). Heavy rainfall that totaled from 6-15 inches over May 5-7, 2003 in Swain and Haywood Counties triggered flooding and over 25 landslides that resulted in costs over $2.5 million. One of the May 5-7, 2003 debris flows traveled into the Bryson City reservoir, less than 600 ft upstream from the run-out zone of the December 1990 debris flow event. Most, if not all of these landslides, were related to modified slopes associated with mountainside development (Wooten and Latham, 2004). This study also confirmed that steep slopes underlain by acid-producing sulfidic rock, common in western North Carolina, are prone to landslides, especially when the slopes are modified by human activity.

On December 11, 2003 a fatal debris flow occurred in Maggie Valley in Haywood County after less than two inches of rainfall during the 24-hour period preceding the embankment failure. Here, the debris slide-flow initiated in colluvial soil overlain by road fill in the transition zone from a debris fan source area and the debris fan deposit (figure 3). Legal action is pending over the death of one occupant. At issue is whether a water supply line buried in the road embankment was leaking and triggered the slope movement. Figure 3 shows the location of the fatal debris flow, and the extensive mountainside development in the Maggie Valley area on debris fan deposits. Development is now moving into the steep source areas.

The remnants of Hurricanes Frances and Ivan tracked across western North Carolina on September 6-8, and September 16-17, 2004 respectively. Five of the eleven deaths in western North Carolina attributed to Ivan resulted from the Peeks Creek debris flow in Macon County (Latham and others, 2005 this volume). In addition to flooding, the intense rainfall triggered at least 90 landslides documented as of March 2005 (this more recent number is revised upward from the number 80 given in the abstract). At least six major landslides occurred on the North Carolina segment of the Blue Ridge Parkway, and some segments of the Parkway remain closed. All told, these landslides caused fatalities, serious injury, substantial property damage, interrupted transportation corridors, and resulted in twenty-six homes destroyed or condemned. Most, if not all, of the private property damage from landslides is not covered by insurance.

As part of the Hurricane Recovery Act of 2005 (Senate Bill 7, http://www.ncga.state.nc.us/Sessions/2005/Bills/Senate/HTML/S7v6.html), the North Carolina General Assembly appropriated funding for the NCGS to begin developing county-based landslide hazard maps for western North Carolina. The NCGS plans to continue to maintain the slope movement-slope movement deposit database statewide, but concentrating in western North Carolina. Inventorying landslides and related deposits are important for several reasons: 1) to determine what geologic and other site conditions make certain areas more prone to landslides; 2) to help identify areas prone to landslides and determine mapping priorities - landslides are likely to reoccur in the same general areas; and, 3) to help calibrate GIS-based landslide computer models - landslide locations should correspond with areas predicted to be unstable.

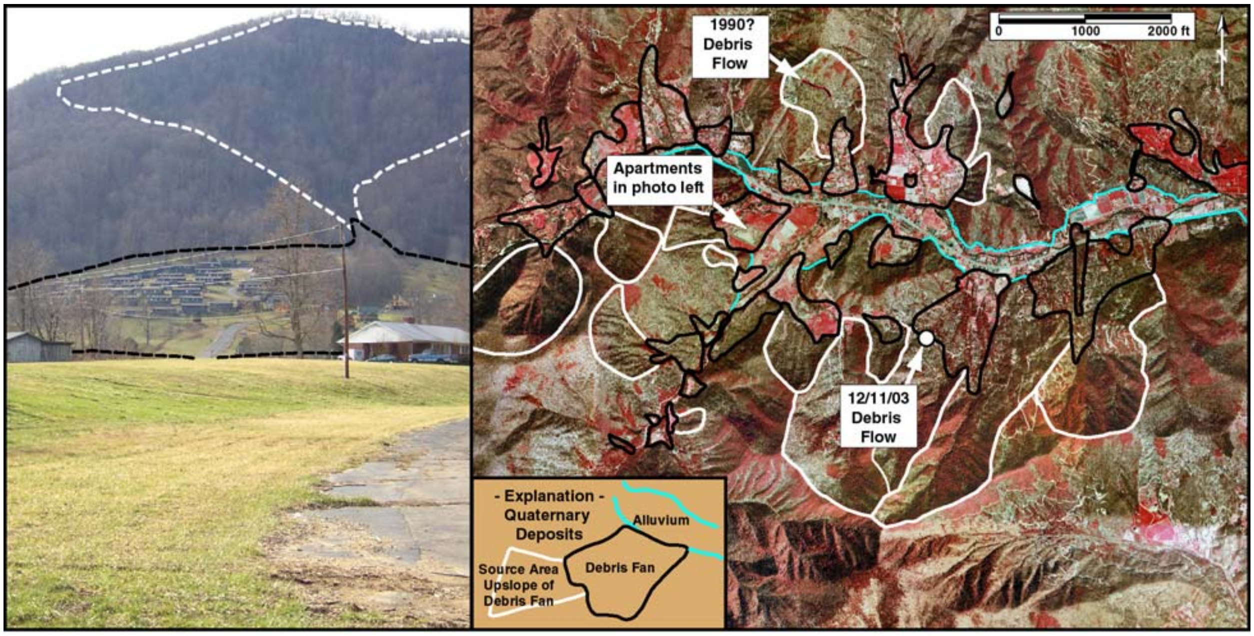


Figure 3. **Photo Left.** Apartment complex in Maggie Valley built on debris fan deposits (black outline) below a steep source area for the fan (white outline). Developments on the fans are at risk if future debris flows originate in the fan source areas. **Photo Right.** Reconnaissance map of some debris fan deposits and their source areas, and locations of recent debris flows near Maggie Valley. The December 11, 2003 debris flow killed one person and destroyed a home. A 1990(?) debris flow occurred on steep slopes in a fan source area and damaged roads. Map base is 1998 digital ortho quarter quads (DOQQ), Dellwood and Hazelwood 7.5-minute quadrangles. Sources of information: Hadley and Goldsmith (1963), Mills (1982), and unpublished reconnaissance mapping by NCGS, 2004.

ABANDONED AND HISTORIC MINES

Subsidence hazards from old mines has received heightened attention because of the rapid urban and suburban development within an old gold mining district of North Carolina (see Reid and Medina, 2000a-f), and by subsidence over abandoned underground workings at the Phoenix Gold Mine, Cabarrus County in 2000 (Wooten and Clark, 2000). Figure 4 shows the locations of the hundreds of known abandoned and historic mines in North Carolina that preceded the State Mining Act of 1971.

During the 2001-2002 timeframe a developer incurred additional expenses to mitigate hazards at a residential subdivision built over portions of the North State Mine near High Point, North Carolina. Additional costly investigations were required to locate remnants of an old underground tin mine during reconstruction of U.S. 321 near Lincolnton, North Carolina. In March 2005, previously unknown underground workings were encountered at the Gibbs Mine, near the Phoenix Mine in Cabarrus County. The NCGS received unconfirmed reports of an airshaft breach in Concord in May 2004.

The greatest potential for ground collapse and subsidence exists over abandoned underground mine workings such as tunnels and shafts that, in some cases, may extend for hundreds of feet underground. During Phase II studies we will prepare a thematic GIS map showing locations of those mines that have known subsurface workings. We will also begin compiling available underground mine maps into a digital format that is geospatially registered using global positioning satellite (GPS) coordinates for use in GIS applications. These mines include the Phoenix-Furniss Mines in Cabarrus County [gold], the Tungsten Queen Mine (Vance County) [tungsten], and the Carolina and Cumnock Coal Mines (Lee County) [coal].

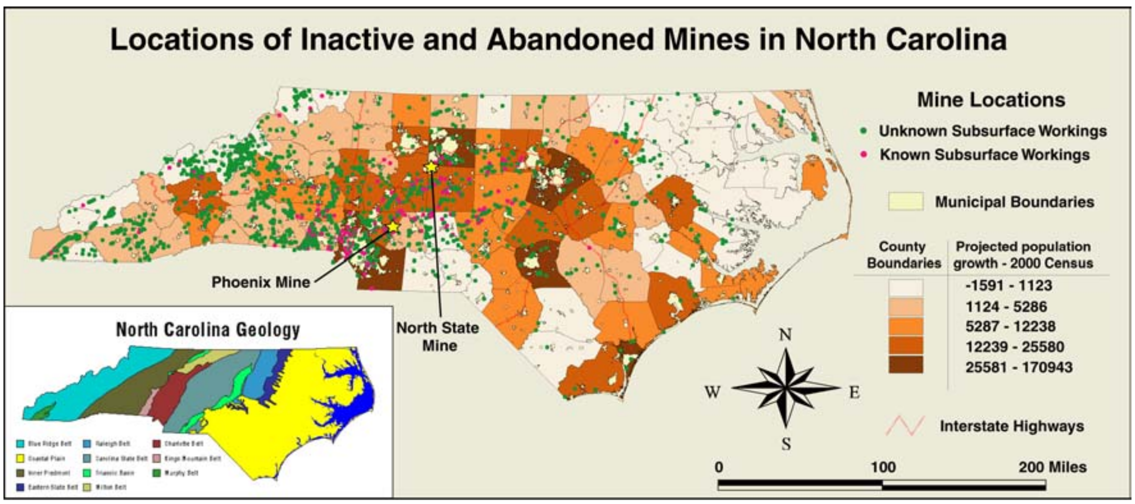


Figure 4. Map showing locations of inactive and abandoned mines from the Minerals Resource Data System for North Carolina (extracted from Mason and Arndt, 1996) along with projected population growth by county based on the 2000 census. Also shown are locations for the Phoenix and North State Mines referred to in the text. The old mining district roughly corresponds to the area containing mines with known subsurface workings. Ground subsidence and collapse, naturally occurring arsenic contamination in soil and groundwater, as well as acidic groundwater pose significant geologic hazards in this rapidly growing part of the south-central Piedmont.

SINKHOLES

Ground collapse and subsidence from limestone sinkholes is a hazard chiefly in coastal areas (mainly Brunswick, New Hanover, Pender, Onslow, Jones, Lenoir and Beaufort counties, figure 5). In addition to ground collapse and subsidence (e.g., in 2001 on I-40 near Rocky Point) sinkholes are also a factor in the potential rapid movement of contaminated groundwater, the dewatering of Boiling Springs Lake in Brunswick County, and mine dewatering. Sinkholes form naturally from the underground dissolution of limestone, but ground disturbing activity and changes in surface water and groundwater flow patterns can lead to the formation of new sinkholes.

Phase I studies mapped sinkholes at a reconnaissance level in Brunswick and New Hanover Counties using digital color-infrared aerial photography. Phase II studies will include limited ground verification of sinkholes mapped in Phase I, along with a terrain and lineament analysis in an attempt to interpret landforms that indicate structural or depositional geologic patterns that will help predict areas more susceptible to sinkhole formation**.** Newly available Light Detecting and Ranging (LIDAR) elevation mapping available through the North Carolina floodplain mapping program will also be used in the landform analysis. New mapping of sinkholes during Phase II will cover Pender and Onslow Counties. After Coastal Plain sinkhole mapping is complete, out-year studies are planned to map the less extensive karst terrain of the Blue Ridge and Piedmont.

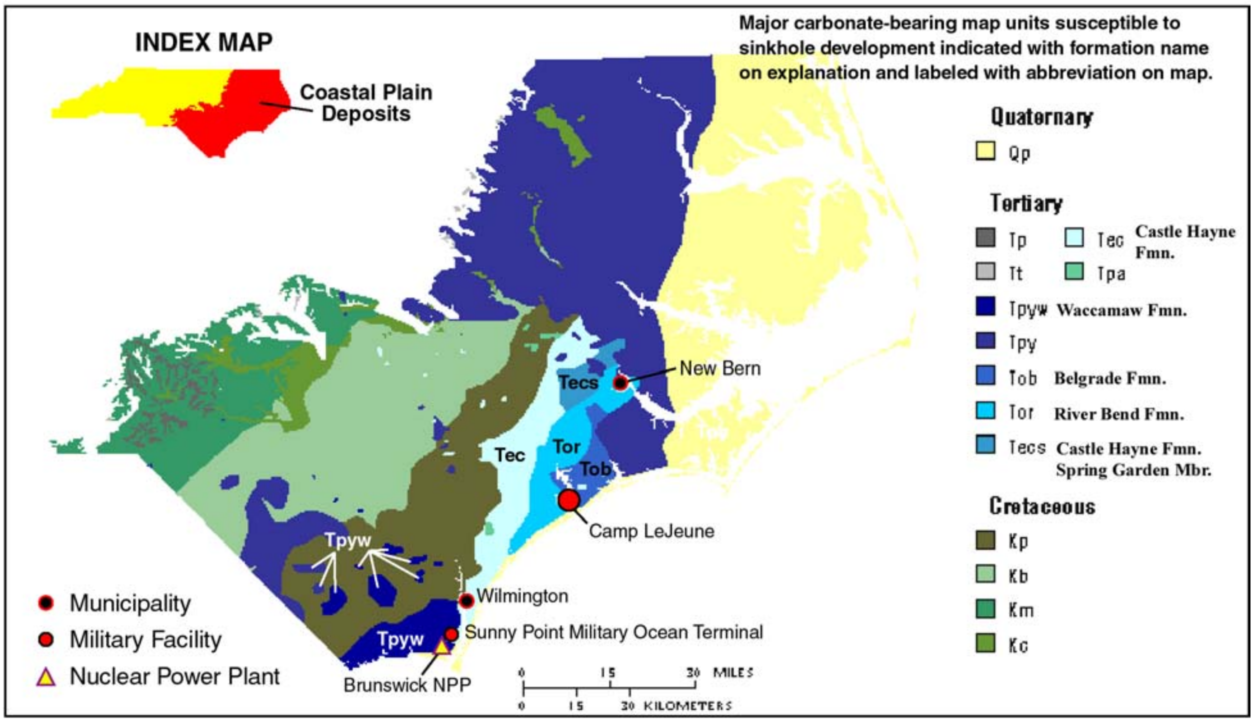


Figure 5. Generalized geologic map of the Coastal Plain showing selected municipalities and critical facilities coincident with major carbonate units susceptible to sinkhole development. In addition to hazards from subsidence and collapse, sinkholes and other karst-related features in carbonate rocks have a direct influence on surface and groundwater contaminant pathways. Carbonate rocks in this region are important aquifers for municipal, industrial, and private groundwater supplies.

HIGH HAZARD DAMS

High hazard dams throughout the state continue to receive increased attention as a result of recent hurricane flooding and the emphasis on homeland security. The purpose of the state's dam safety program is to prevent property damage, personal injury, and loss of life from the failure of dams. There are over 900 “High Hazard” dams in North Carolina, and many are located in densely populated areas. Dams are considered “High Hazard” if the Division of Land Resources, Department of Environment and Natural Resources, determines that failure of the dam could result in loss of human life or significant damage to the property below the dam. As these dams age, evidence that these structures are truly dynamic is realized through sometimes catastrophic failures.

Fatal and damaging dam failures in North Carolina include the following three examples: 1) although there was no loss of life, extensive property damage and a depressed local economy resulted from the Lake Toxaway dam failure in 1916 after three tropical weather systems tracked over the Blue Ridge Mountains during a six week period; 2) the 1976 Bearwallow Lake Dam failure in western North Carolina took the lives of four family members; and 3) Heavy rains during May 2003 triggered three dam failures near the Cumberland-Hoke county line (damages from just one of these failures, the Hope Mills dam, exceeded $6 million). Because of improved regulation and engineering, there were no major dam failures during Hurricanes Frances and Ivan.

Phase I deliverables include GIS coverages showing high, moderate, and low hazard dams in North Carolina. Phase II activities related to dam safety will primarily be to maintain currency of existing dams GIS data layers as the dams database is updated by the Land Quality Section (LQS). These data sets and GIS data layers are ‘law enforcement protected’ and have restricted access.

SEISMIC HAZARD (EARTHQUAKES)

Although damaging earthquakes centered in North Carolina are relatively rare, the state is at risk from known seismically active areas that include the Charleston, South Carolina, the Eastern Tennessee, and the New Madrid seismic zones (figure 6). The Eastern Tennessee seismic zone is the second most seismically active zone in the Eastern United States (after the New Madrid seismic zone) and extends into western North Carolina. A magnitude 5 or greater earthquake in this zone would not only cause structural damage in western North Carolina, but would likely generate landslides that could block major transportation routes (e.g., I-40 through Pigeon River Gorge). The earthquake GIS data layers will be updated annually. Phase II studies include publication of an earthquake epicenter map for North Carolina and adjacent southeastern states

ARSENIC

A collaborative investigation is underway to improve the understanding of naturally occurring arsenic contaminated groundwater in the Piedmont of North Carolina, between geologists of the NCGS and hydrogeologists of the Mooresville and Raleigh offices of the North Carolina Aquifer Protection Section, DENR (Reid and others, 2005). The current U.S. EPA drinking water standard for arsenic is 50 ppb and will decrease to 10 ppb in January 2006. Work by Pippen and others (2003) shows that, in general, the highest probability of arsenic concentrations exceeding the U.S. EPA drinking water standard corresponds with parts of the old gold mining district in the Slate Belt (figure 4). Additional information on concentrations of arsenic in stream sediment can be found in Reid (1991)

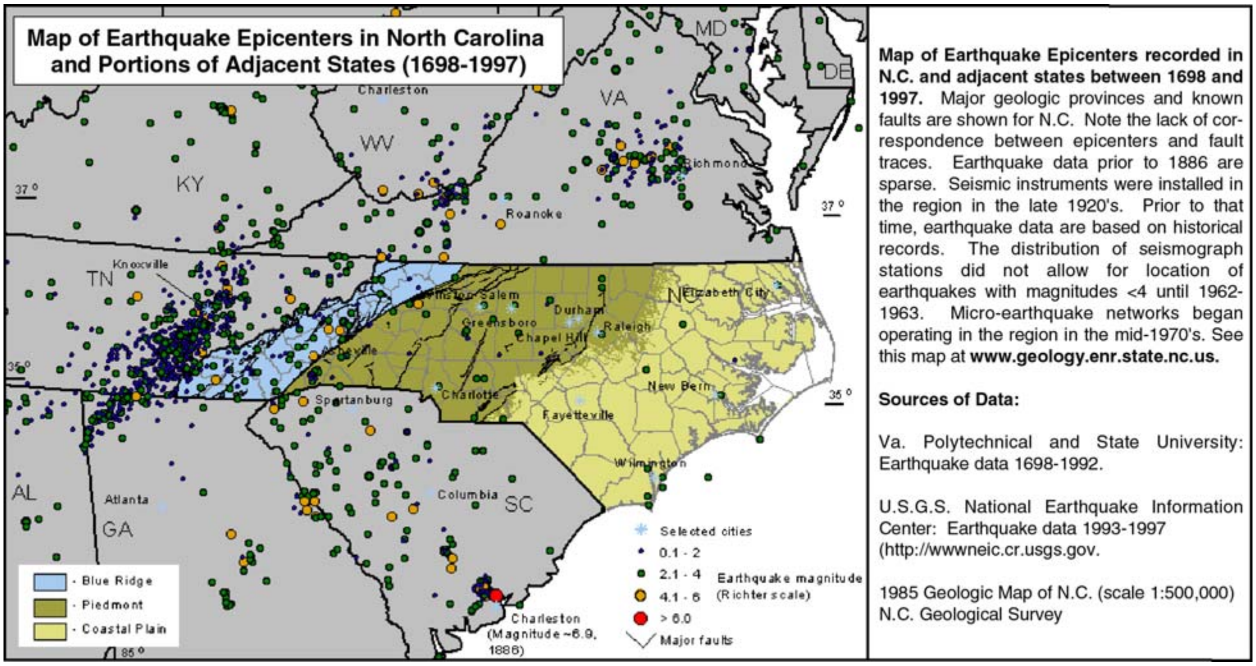


Figure 6. Map of earthquake epicenters in the southeast region excerpted from Geological Note 7 (Axon and Wooten, 1998).

Recent analysis of water samples collected from domestic supply wells across the North Carolina Piedmont revealed areas where the presence of arsenic contaminated groundwater is common (Pippin, personal communication 2003). Prior to the analysis, visual examination of archived geological mineral exploration cores from Davidson and Montgomery Counties, North Carolina, indicated the presence of sulfide minerals (Reid, personal communication, 2003). Davidson and Montgomery Counties are located in areas known to have elevated concentrations of naturally occurring arsenic.

Current field and laboratory studies involve laboratory analyses of these cores to determine if the respective sulfide minerals and their host rocks are a naturally occurring source of arsenic that has resulted in the contamination of local water supply wells. In addition, field-based geochemical studies of naturally occurring iron-manganese boulder coatings, and ceramic streak plate experiments, and other materials from water supply wells are being used to detect arsenic (figure 7).

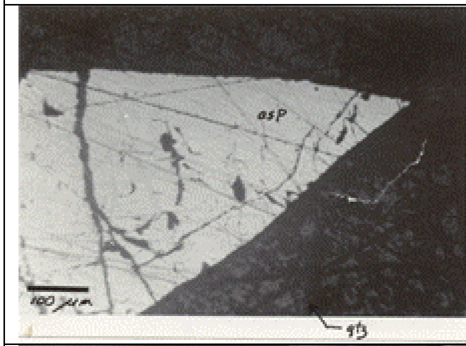
Sulfides have been observed in the examined cored rocks as vein deposits that were emplaced parallel to the regional foliation/bedding features or as later vein deposits that cut perpendicular to the regional foliation/bedding features and as disseminated minerals in the rock mass. Reflected light microscopy of the cores, has identified arsenopyrite and pyrite as the sulfide mineralogy of the cores. Other primary sulfide minerals have not been identified (figure 8).

Altered primary sulfide minerals have been observed throughout the core, often associated with manganese oxide 'blooms'. Secondary minerals that coat fracture walls in oxidized sections of the core bear significant arsenic concentrations, suggesting precipitation of dissolved constituents from the weathering zone observed in the studied core. A generally degraded primary framework of silicate minerals characterizes the weathering zone; many of the silicate minerals have been altered to clay. It is well understood that dissolved components of the weathering profile are commonly transported and precipitated on fracture walls during oxidation, thus creating the often observed iron and manganese hydroxide coatings found on fracture walls. It is also well known that arsenic has an affinity for iron and manganese oxyhydroxides and analysis of fracture coatings from the core bear this out (figures 7, 9).

As stated above, iron and manganese oxyhydroxide coated boulders and ceramic tiles (figure 10) have been collected for this study. Analysis of boulder coatings, collected from a stream draining a former gold mining area, indicates significant arsenic concentrations. Time integrated analyses of coatings on unglazed streak plates provide some insight into the flux of arsenic into the stream system and by extension to other discharge points in domestic water supply wells. Significant concentrations of arsenic have been detected on the plates over time. Similar results are expected when analyses of tiles deployed in an active supply well are recovered and analyzed chemically using an ICP. Chemical extraction was used to remove the iron-manganese oxyhydroxide coatings from rocks and streak plates for analyses by ICP. Subsequent analyses will use metal ratios for rocks to normalize data. A similar approach is planned for the streak plates along with normalization to surface area of the streak plates.

Our interim conclusion suggests that chemical weathering of the upper bedrock results in the dissolution of arsenic from sulfide bearing minerals, and depending on the groundwater chemistry, is precipitated onto fracture walls. The fraction that is not precipitated is then flushed from the groundwater system *via* discharge to surface waters where transport through a greater oxidation front (i.e. moving from a groundwater system to a surface water system) forces precipitation of more iron and manganese oxyhydroxides and as a result removes arsenic out of solution.

Figure 7. Unglazed ceramic streak plates deployed on a concrete block in a stream draining a gold prospect, Piedmont, North Carolina. A time-integrated field experiment to collect iron-manganese oxyhydroxide coatings with arsenic at this location. Broken natural rock (placed left corner of block) shows heavy natural iron-manganese coating on quartz rock.
Six streak plates were affixed to each block plates could be removed over time and their coatings geochemically analyzed to monitor rate of iron-manganese coating development. The blocks were set so that oxygenated water ran over the top of the blocks. Eh conditions are important in precipitation of such

	coatings (see figure 8).
	**Figure 8.** Arsenopyrite (white) and quartz (gray) – 100 micron scale. The reflected light photomicrographs are courtesy of Mr. Sam Phifer, Jr and are from his 1988 report, "Economic Geology of the Long Mine area, Gaston County, North Carolina." Phifer's report included a brief ore paragenesis study by Dr. Geoff Feiss, formerly of UNC-CH. The photomicrographs are from his study.
	**Figure 9.** Diamond drill cores from 'oxidized' zone display iron-manganese filled fractures, and a general breakdown of the framework silicate minerals (e.g., feldspar). Manganese 'blooms' are frequently observed. In contract, the 'reduced' rocks appear competent and sulfide minerals generally are retained.
	**Figure 10.** After nearly a year the ceramic plates display extensive coatings of iron-manganese oxyhydroxide coatings.

URANIUM

Paleozoic granitic rocks, Triassic sedimentary rocks, and some Cambrian-Late Proterozoic metasedimentary and metavolcanic rocks are typical sources for naturally occurring uranium contamination in groundwater. The map in figure 11, constructed from data collected during the NURE (National Uranium Resource Evaluation) program during the mid-1970's, shows uranium concentrations in groundwater >0.1 ppb are commonly associated with these rock types. The EPA drinking water standard for uranium is 30 micrograms/liter (ppb). The recent case of uranium in drinking water near Simpsonville, South Carolina, has brought attention to the possibility of similar occurrences in North Carolina.

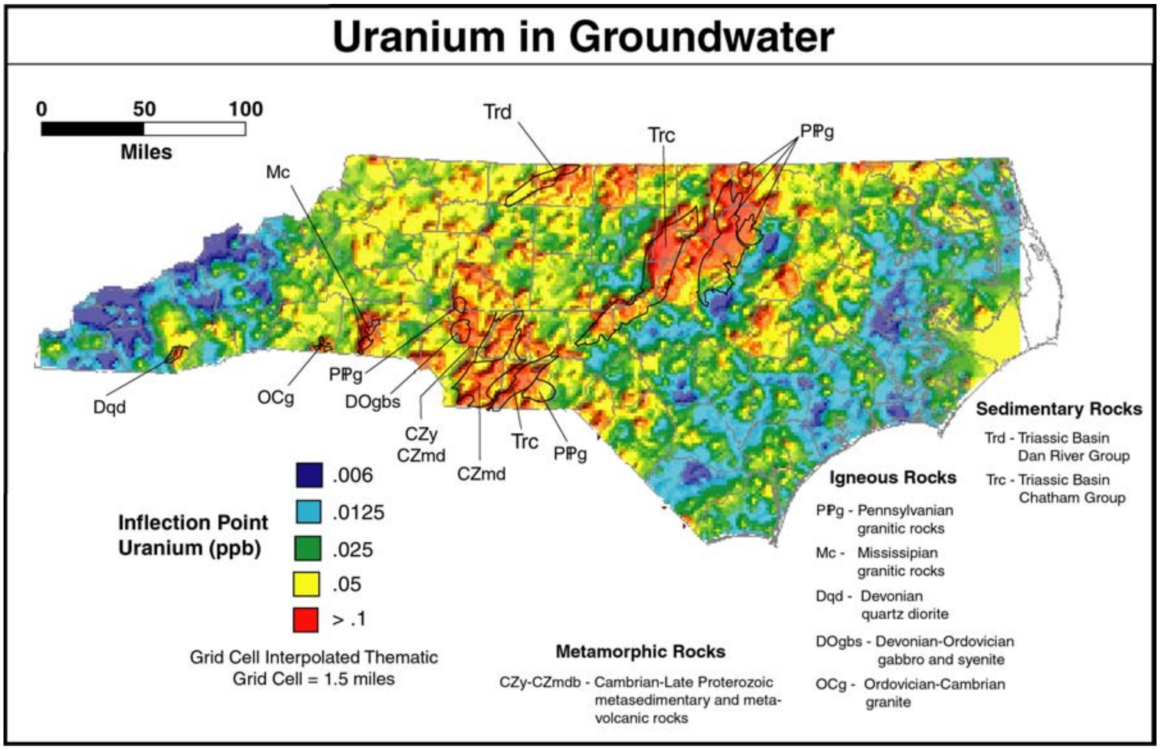


Figure 11. Map showing relative concentrations of uranium in groundwater and selected rock units that correspond with uranium concentrations in the range of .05-.1 ppb. Source of uranium data is the National Uranium Resource Evaluation Program (NURE) from Sargent and others (1982); concentration and distribution of uranium from Reid (1993), geologic data are from the North Carolina Geological Survey (1985); figure modified from Wooten and others (2003).

CONCLUSIONS

Geologic hazards occur in all geologic provinces in North Carolina, and include slope movements, subsidence above abandoned mines, limestone sinkholes, arsenic and uranium in ground water, acidic rock, earthquakes, high shrink-swell soil, and indirectly, high hazard dams. These hazards threaten public safety, transportation routes, and sustainable development statewide. Initial work is underway by the North Carolina Geological Survey to construct a GIS-based geologic hazards atlas for North Carolina. Funding through the North Carolina Division of Emergency Management and the Federal Emergency Management Administration is essential to continue work on this multi-year effort. Recent appropriations by the North Carolina General Assembly for landslide hazard mapping provides start-up funding for landslide hazard mapping in western North Carolina. Interagency cooperation between the North Carolina Division of Emergency Management and county Emergency Management organizations, North Carolina Division of Land Resources, North Carolina Division of Water Quality (Aquifer Protection Section), the North Carolina Department of Transportation and others is a vital to this program.

ACKNOWLEDGEMENTS

The authors would like to posthumously acknowledge Fritz Koch, a longtime member of the N.C. Department of Transportation Geotechnical Unit, and friend of the NCGS. Fritz first proposed the idea of creating of a geologic hazards atlas for North Carolina at a Raleigh meeting of the Carolina Section of the Association of Engineering Geologists in the early 1990's.

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Landslide Investigation and Mitigation Along US 160 Between Durango and Mancos Colorado using Lightweight Fill, Ground Anchors, and Rockery Buttresses.

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During the spring of 2004, five landslides developed along US 160 between Durango and Mancos, Colorado. The landslides became active after above average snowfall and rainfall in the area. Two of the five landslides appeared to have been unsuccessfully mitigated in the past and had re-activated. The other three landslides appeared to have become active recently. Yeh and Associates provided the geotechnical investigation, landslide evaluation, slide correction alternatives, and relative cost estimates for various mitigation systems. Yeh and Associates, Inc. also provided final mitigation design for all the landslides. Three of the landslide mitigations included the use of lightweight expanded polystyrene (EPS) fill replacement. The other two landslide mitigations included the use of ground anchor tiebacks and a rockery buttress.

Regional geologic units that underlie the landslide areas are composed of Cretaceous Age Mancos Shale that typically weathers near the surface to form sandy silts and clayey materials. The Mancos Shale and derivative materials are known for low shear strength and poor slope stability characteristics. The underlying subsurface materials typically consist of approximately 10 to 30 feet low to medium plastic clays that overlie weathered to unweathered shale/claystone bedrock.

The geometry and movement of the active landslides appeared to be controlled by a combination of factors including elevated groundwater levels, highly weathered bedrock surfaces, and inappropriately placed embankment materials. Typically the landslide geometries exhibited classic rotational and shallow planar failures.

Mitigation with lightweight fill (EPS) replacement was used on three of the landslides, since it appeared to be the most cost-effective and efficient mitigation method. The mitigation concept of replacing existing embankment fills with lightweight fill (EPS) is to reduce the driving forces that act on a slope profile to increase the overall global factor of safety. The density of the EPS is typically between 1 to 1.5 pounds per cubic foot, as compared to 100 to 110 pounds per cubic foot of existing embankment fill.

Ground anchor tiebacks and rockery buttresses were used at the other landslide sites where EPS fill replacement was not considered appropriate or functional. These mitigation concepts increase the resisting forces that act on a slope profile to increase the overall global factor of safety.

Ten Year Performance of a 400-foot High Rock Cut in Coal Measures Rocks

By

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A 400-foot rock cut was constructed in 1995 for US 460 near Grundy, Va., in massive sandstones, coal seams, and silty shale seams. The performance of the cut has been visually monitored over time. Rock durability and jointing, especially valley wall stress relief joints, were evaluated in the design of the cut by designing the cut to reduce the potential for undercutting of more durable units and using slopes so that joints with the highest probability of failure would be removed during construction. This was accomplished by using benches at stratigraphic breaks and varying the cut slope face based on the characteristics of the unit. Weathering and breakdown of the rock in the suspected low durability units was observed over time. Strength and slope durability index test data from the site and from the same formation in adjacent cuts are compared to actual weathering of the unit observed in the cut, and good correlation was observed. Joint measurement data and stereonet analyses are also compared against actual performance.

Blasting impacts on the in-place rock were also noted, such as cracking along presplit holes and overbreak. Some of these blast impacts may be related to the use of too much explosive due to the blasting agents flowing into valley wall stress relief joints intercepted by the blast holes or weathered conditions in the vicinity of the joints.

Overall, the performance of the slope has been very good, with only limited rockfalls landing on the benches and no rockfalls from the top of the cut reaching the drop zone. Many of the observed rockfalls appear to be related to thin zones of rock created by near vertical valley wall stress relief joints intersecting the cut face at shallow angles.

The change in the observed condition of the cut has been minimal over the last 6 years. Most of change occurred in first 4 years after the cut was constructed.

Geotechnical Challenges Associated with US-59; Lawrence, KS to I-35 near Ottawa, KS

Carrie Denesha, MS[1];
Robert Henthorne, PG[2];

Introduction

The Kansas Department of Transportation is currently working on a 21 mile new alignment for US-59 between Lawrence, KS and Ottawa, KS. The geotech portion of this project began while the design portion was still being completed. There are over 150 landowners on the project and 15 additional miles of frontage roads and side road improvements. Along with the logistical struggles, there are several geotechnical issues that we were presented with. The largest of which is a 60+ foot cut through a hill that is comprised of the upper part of the Lawrence Shale Formation.

The Lawrence Shale is Pennsylvanian in age and a member of the Douglas Group, which outcrops throughout the eastern quarter of Kansas. The Lawrence Shale Formation is a problematic thinly bedded sandstone and shale, with a history of landslides. The primary reason for landslide failure is within the fine scale grading within the laminations and in abrupt changes grain size which produce significant fine-scale inhomogeneities within the rocks (Archer, 1992). The original alignment chosen by the designers was going to attempt to notch out the west side of the hill, which not only was a set up for catastrophic failure, but also had placed the roadway over two large paleo-landslides. These landslides were easily seen on fine-scale topographic maps on early plans. The geotech section convinced the planners and designers to move the alignment to mount the hill at a direct angle, therefore reducing the chances of a total loss and avoiding the paleo-landslides, which posed a different set of problems.

Other complications of the Lawrence Formation on this project include the high percentage of kaolinite minerals, which poses an increased risk for slides once wet. The high clay content causes the Lawrence Shale Formation to have a high swelling percentage and will likely have unloading problems when several feet are removed. Archer 1992, stated that the upper portion of the Lawrence Formation was the "worst of all possible cases in the finely laminated shales; this includes kaolinitic clays which once moistened become very slick combined with thin sand layers that allow the clays to become readily saturated with groundwater and/or infiltration from rainwater or snowmelt. For these reasons the pinstriped to flaser bedded facies of shales are not suitable for normal grade slopes and are particularly unsuitable as fill materials." There are also several high angle slide planes in a variety of angles, with secondary mineralization which give more evidence for irregular instability within the finely bedded portion of the Formation.

There will still be several problems facing the geotech section of KDOT during the design phase, construction phase and ongoing maintenance; which may include over excavation of the problematic shales, lowering the angles of backslopes of the Lawrence Formation, retaining walls, soil nails, and auger cast piling.

Location

US-59 is located in the eastern quarter of Kansas connecting Atchison near the Missouri state line to Chetopa near the Oklahoma state line. US-59 is listed as a secondary arterial, but between the cities of Lawrence and Ottawa there is a much higher traffic load. The new construction will take place along a portion of US-59 which connects the growing city of Lawrence to I-35 which funnels traffic between Kansas two largest cities; Kansas City (and its suburbs) and Wichita (figure 1). This 21 mile project cuts thorough areas of rural homes and suburban farmland on rolling hills. The eastern quarter of Kansas receives 25-30 inches of rain per year with the majority falling in the spring months. There are also several freeze and thaw cycles in the northern part of the state.

The new four lane limited access alignment for US-59 will be 21 miles long and east of the existing two-lane road. The improvement to US-59 spans two counties, Douglas and Franklin, and is divided into two projects at the county line (though when 'project' is used in this paper it will refer to the two projects combined) There are over 150 landowners along this project and there is an additional fifteen miles of access roads and side road improvements. There are also over thirty span structures on this project which take US-59 over small streams, watershed lakes, county roads, US-56 and I-35.

Figure 1. Location of project outlined by black rectangle; US-59 from Lawrence to I-35. Major routes; 1-70, I-35, I-135 and I-335 are in bold

Geologic Setting

During the late Pennsylvanian Kansas was part of the Midcontinent basin, which consisted of the rapidly subsiding clastic dominated foreland basin of the Ouachita-Arbuckle orogenic belt (south-central Oklahoma) and a large carbonate-rich continental shelf to the north and west (Joeckel, 1994). The Groups encountered on this project include the Douglas and Shawnee as well as minor amounts of the Lansing at the very southern end of the project in Franklin County. Both the Douglas and Shawnee Groups were influenced by major fluvial-deltaic systems that drained the orogenic belt and were the main source of the clastic sediments. Both the Douglas and Shawnee groups thicken drastically and coarsen to the south in eastern Kansas closer to the source of the sediment (Joeckel, 1994). Subtle tectonism around the northern end of the Nemaha Uplift in southeastern Nebraska appears to have had a local influence on sedimentation and distribution as well as the Bourban Arch in southeastern Kansas. The large influx of sediment coupled with regressing and transgressing of the inland sea, resulted in several large cyclothems that posses paleosols, rippled and cross-bedded sandstones, coal, shale and marine limestones.

Geologic Descriptions

There are several geologic members that will be encountered on this project. All are Upper-Pennsylvanian series and of the Missourian and Virgilian Series (figure 2). The Stanton Limestone Formation are the lower most units encountered on the project are of the Lansing Group. These beds are near the surface at the southern end of the project near I-35 in Franklin County. Overlying the Stanton Limestone are the rocks of the Douglas Group which include the Stranger and Lawrence Shale Formations. These units are persistent along the majority of the 21 mile project. Overlying the Lawrence Shale Formation is the Oread Limestone of the Shawnee Group. The Oread Limestone is present on the hill tops in the northern half of the Douglas County portion of the project.

Lansing Group
Stanton Limestone Formation
<u>Stoner Limestone Member</u>

The Stoner Limestone Member was encountered in core drill soundings at the bridge sites for the US-59 over I-35 interchange. This unit is continuous in the subsurface of Franklin County. The Limestone is light-gray to white and fine grained. Individual beds range tend to be thick with thin shale partings. Several phyla are present including; algae, trilobites, bryozoans and brachiopods. The thickness of the Stoner Limestone Member in this area is approximately 18 feet.

<u>Rock Lake Shale Member</u>

The Rock Lake Shale Member is a thin limy brown shale, there is a thin coal seam and abundant fine pyrite crystals. The coal and pyrite are typical of a calm water oxygen-poor environment and a sea regression. The average thickness of the Rock Lake Shale is approximately five feet. This unit was found near I-35 and will only be encountered in minor cuts for side road improvements and for foundation elements for the US-59 over I-35 bridge.

<u>South Bend Limestone Member</u>

The South Bend Limestone Member is light gray sandy limestone with gradational contacts to both the Rock Lake and the Weston Shale. Fossils are locally abundant with fusulinids, crinoids, and bryzoa being common. In the upper portion of the South Bend chert nodules may be encountered. There are also dissolution vugs that are heavily oxidized where the occurrence of sand increases. The South Bend is approximately six feet thick with a wide range of lithologies. The South Bend Limestone is found along the southern mile of the project. Where found it has an eroded top and is weathered.

Douglas Group
Stranger Formation
<u>Weston Shale Member</u>

The Weston Shale is present along the new alignment for US-59 near the surface at the intersection of US-59 and I-35 north to the intersection with the Midland Railroad. The Weston is grayish blue to medium gray clayey shale. It weathers deeply to a yellow-brown to brown-tan. It lacks fossils but thin localized beds may be present. Iron concretions are common in the Weston Shale, they are typically pink-gray, weather yellow-brown or reddish brown and very hard. These concretions are elliptical, flattened parallel to bedding, and two to twelve inches in diameter occurring both in layers and as scattered concretions (O'Conner 1960). The Weston has an extremely high swelling potential with a high liquid limit. The total thickness was never reached, but was in excess of fifty feet. Problems are likely to occur with this unit when it is loaded.

<u>Iatan Limestone Member</u>

The Iatan is very inconsistent and may be mistaken for the Westphalia Limestone. If this unit is present along the project area, it only occurs in the central part of the Franklin County project.

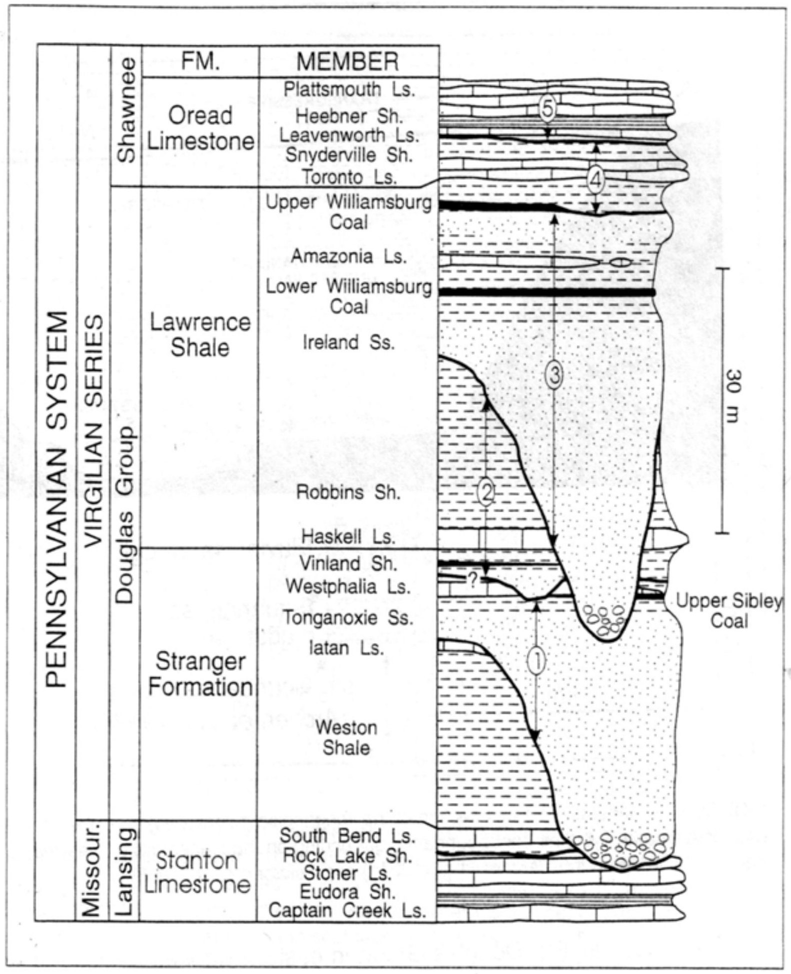


Figure 2. From Archer and Feldman, 1994. Stratigraphic representation of the geologic units found along the new alignment for US-59. The Stanton Limestone is found in the southern part of the project while the Lawrence and Stranger Formations dominate the majority of the near surface rocks and the Oread Limestone Formation is found in the northern part of Douglas County on hill tops.

Tonganoxie Sandstone Member

The Tonganoxie Sandstone Member may be present on this project, if so it is isolated in the north central part of the Franklin County project. The Tonganoxie is a non-marine sandstone that has cut a channel in the Weston Shale and locally the Stanton Limestone Formation.

Westphalia Limestone Member

Like the two members that underlie it, the Westphalia Limestone is discontinuous in Franklin County and may not be encountered. Where present the Westphalia Limestone is a very thin (0.5 to one feet), gray carbonaceous limestone.

Vinland Shale Member

The Vinland Shale Member is a thick gray to greenish gray clayey, calcareous, sandy shale and sandstone. Septarian concretions are present in the upper middle part of this unit. The Vinland has a faunal zone containing abundant mollusks in the upper portion of the Vinland Shale. The Vinland is present though much of the northern portion of Franklin County and is more than 30 feet thick.

Lawrence Formation

Haskell Limestone Member

In Franklin County the Haskell Limestone is a good marker bed to distinguish the Vinland Shale Member from the Robbins Shale Member. The Haskell is a very hard blue-gray fine-grained limestone. There is commonly a well developed oolitic facies near the top and bottom of the unit that contains algal remains, fusulinids and brachiopods. The Haskell is consistently five feet thick until it pinches out to the east near Stafford Road, here the Haskell is near the surface and exposed in the back slopes and ditches.

Robbins Shale Member

The Robbins Shale Member extends from the north central part of Franklin County into the extreme southern part of Douglas County along the new alignment for US-59. The Robbins Shale member is a gray to yellow-gray marine shale that is locally eroded and overlapped by the non-marine Ireland Sandstone. This sandy shale is very thick and the maximum thickness was not obtained. This unit weathers deeply and is typically overlain by ten or more feet of mantle.

Ireland Sandstone Member

The Ireland Sandstone Member is a locally developed sandstone channel that is extremely thick near the Douglas-Franklin County line. Rutan (1980) offered the informal name “Hole in the Rock Sandstone” to describe the sandstone that is found in the same stratigraphic position as the Ireland but in Douglas and Franklin counties where as the Ireland’s type locality is to the southwest in Woodson County. It has very unique bedding compared the Ireland, with festoon crossbedding and its erosional basal contact (Rutan, 1980). This sandstone is composed of monocrystalline quartz and minor amounts of feldspar and other accessory minerals. It is fine grained with mean grain size ranging from 3 to 1.7 phi and is very well sorted (Rutan, 1980). It is believed to be a result of deposition by a delta distributary due to its lateral association with shallow marine facies. In the lower part of this sandstone there is a thick sequence of sandstone conglomerate with sub-rounded sandstone and limestone cobbles. In the Upper portion of this sandstone it grades into a siltstone with linsen beds representative of a shallow tidal marine facies.

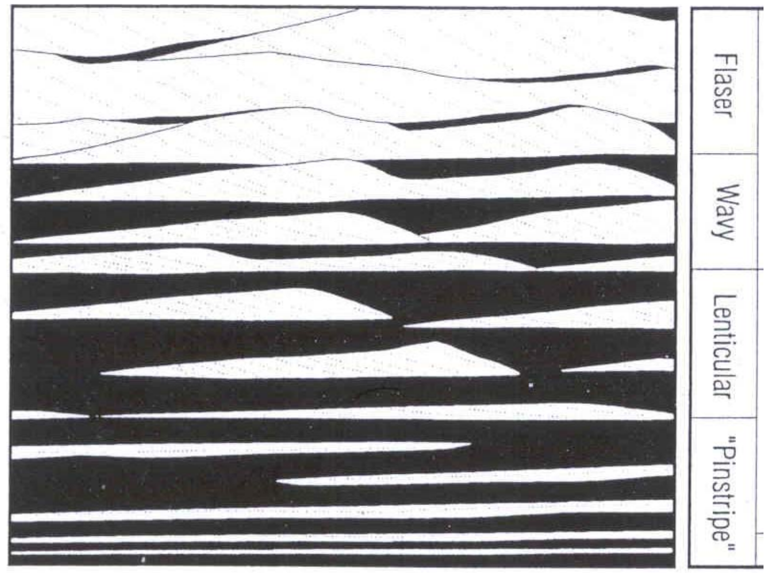


Figure 3 (from Archer and Feldman, 1994). Comparison of flaser, wavy, lenticular, and pinstripe bedding all common rhythmites associated with tidal environments where sedimentation alternates between bed-and suspended-load deposition.

As the Ireland Sandstone grades laterally to the north, toward the end of the project in Douglas County, into the Upper Lawrence Shale, it becomes a very sandy shale that is associated with an abundance of geotechnical problems. This portion of the Lawrence Formation is further from the main sources of sediments resulting in finer grained sediments. The bedding is very fine, alternating between thinning beds of dark fine grained sediments and thickening sand grained sediments. The bedding varies between pin stripe bedding, lenticular, wavy and flaser type bedding (figure 3). These types of bedding are indicative of tidal environments where sedimentation alternates between bed-and suspended-load deposition (Archer and Feldman 1994). It also exhibits soft sediment deformation that suggests an unstable environment at the time of deposition (figure 4).

Figure 4. Near vertical bedding within the upper Lawrence Shale, likely a product of soft sediment deformation. Notice the flaser bedding and wavy nature of the beds. Scale is in tenths of a foot.

This unit is extremely thick, over 150 feet, and will be exposed in very large backslopes as several (50+) feet will be excavated to reach the proposed grade. The deep excavations are likely to cause problems as the Lawrence Formation has a tendency to fail, resulting in landslides. Three paleo-landslides were observed near the new chosen alignment for US-59. The Shale slides to its preferred internal slope of 1:6. This unit will also cause problems when loaded due to its high liquid limit and tendency to fail internally near the zone of active weathering.

Amazonia Limestone Member

This unit is present in the middle part of the Douglas County portion of the project. It is a very hard dense white limestone with a high degree of variability in color and thickness. The Willimasburg Coal is found near the upper contact of this Member. The coal is very thin and is not laterally continuous like the Amazonia.

Shawnee Group
Oread Limestone Formation
Toronto Limestone Member

The Toronto is a dense gray-orange to white limestone. The upper four feet of the Toronto has abundant vertical joints with thin clay seams present. Fossils are also found in this portion. In the lower five feet the Toronto is very hard and light gray to white near the base. This portion lacks fossils. Near the surface the limestone weathers to a deep brown. It ranges from eight to nine feet thick in this area. It is found near the Pleasant Grove Hill (project map, figure 8, end of report) area along proposed US-59.

Snyderville Shale Member

The Snyderville Shale is gray-green and present only near Pleasant Grove Hill. It is limy and seems to lack internal structure. This unit is roughly eight feet thick.

Heebner Shale Member

The Heebner Shale is composed to four distinct units in ascending order; thin yellow-gray clay, black platey shale, blue-gray shale and a calcareous clay shale. Like all units in the Oread Limestone Formation the Heebner is found near the Pleasant Grove Hill area. It is approximately 7.5 feet thick.

Plattsmouth Limestone and Huemader Shale Members

The upper two most units found on the project are the Plattsmouth Limestone and the Huemader Shale Members. They are found near the surface in the Pleasant Grove Hill area along the new alignment for US-59. The Plattsmouth is a thick blue-gray limestone that is approximately 15 feet thick with abundant fossils. The overlying Huemader is a green-gray silty shale, the total thickness of which was not investigated in this project.

Design and Alignments

In the early stages of the design for the new US-59, several alignments were suggested. Over 10 alignment options were proposed and discussed amongst the road designers. Options were explored to the west of existing US-59, widening and lowering the grade of the existing highway, and several options to the east, with 300 foot and 1 mile options being the most favored. The list of 13 was narrowed to six with several paying attention to the area around Pleasant Grove Hill (figure 8). This area along the existing highway has had a tendency for slope failure and landslides. Any alignment chosen for this portion of the new roadway would have to traverse the Lawrence Formation. The Geotech Section main objective was to minimize the exposure to extremely poor geologic materials found in this formation.

Alignment 1

In September of 2003, the Geotech section of KDOT was presented with this project and a chosen alignment; a 21 mile new four lane alignment to replace US-59 in Franklin and Douglas Counties. At this time, all the Geotech section was given was a line on an aerial photograph. In addition to the 21 miles of new mainline, 15 additional miles of access road and side road improvements were planned. KDOT was also given a 30 day time period to complete the investigation, develop Cross-sections and Profiles and submit our recommendations to design.

During investigatory drilling along the centerline of the project the Geotech crew encountered the sandy shale of the Lawrence Formation. The kaolinite minerals found in the thin beds of the shale become extremely slick when wet and with repeated wetting and drying cycles are prone to be a leading factor in landslides. In investigatory cores near Pleasant Grove Hill thin and irregular bedding planes were noted in sandy shale, there were also several slip planes found with multiple slide planes that had polished slickensides (Figure 5). This chosen alignment was cutting away a steep hillside and exposing over 60 feet of Lawrence Shale in a side hill cut section. (Figure 6; Alignment 1). As the Geotech section became more involved it became apparent that alignment 1 was not the safest or financially sound option.

Figure 5. Slickensides found in the Lawrence Shale Formation on alignment near proposed grade on Pleasant Grove Hill.

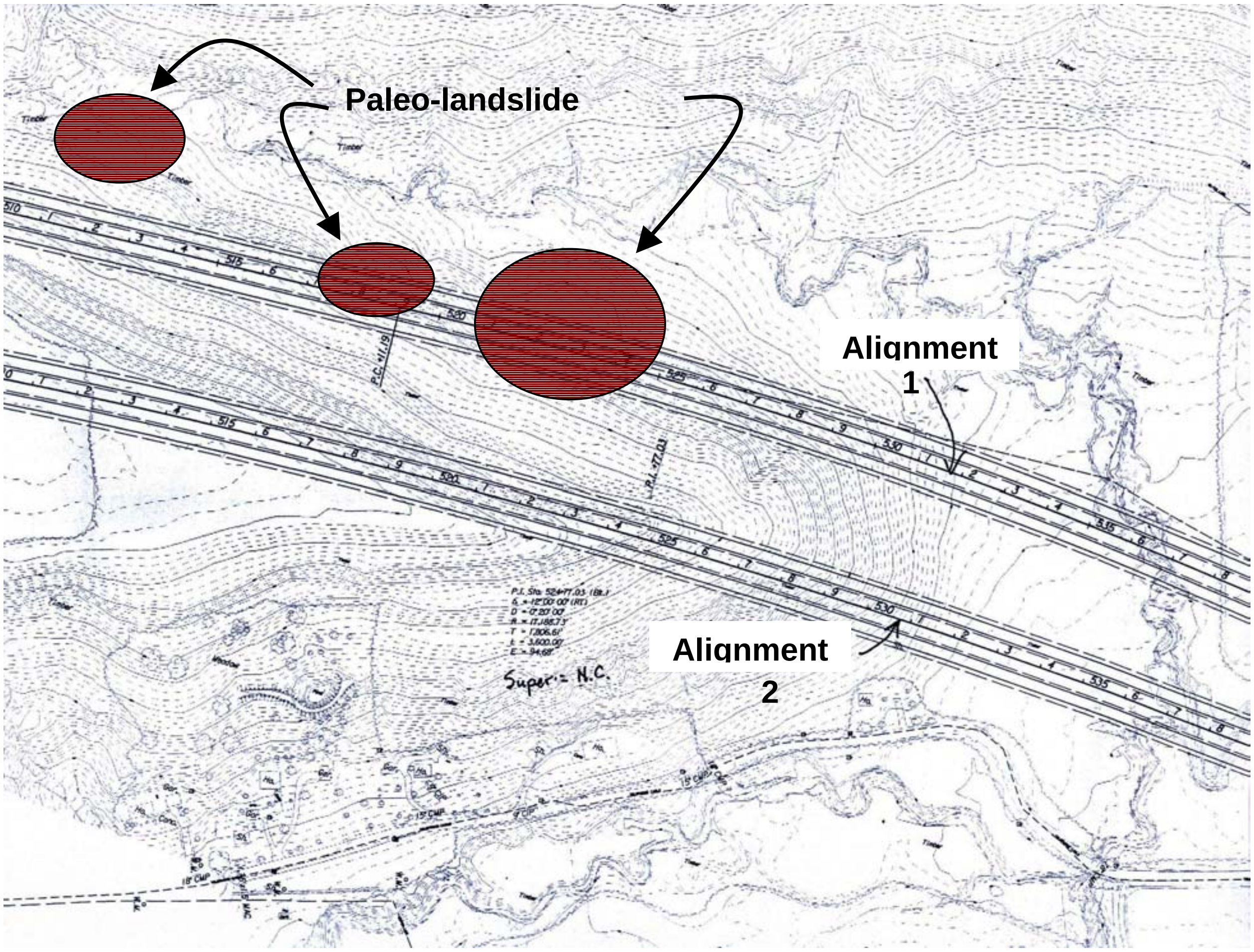
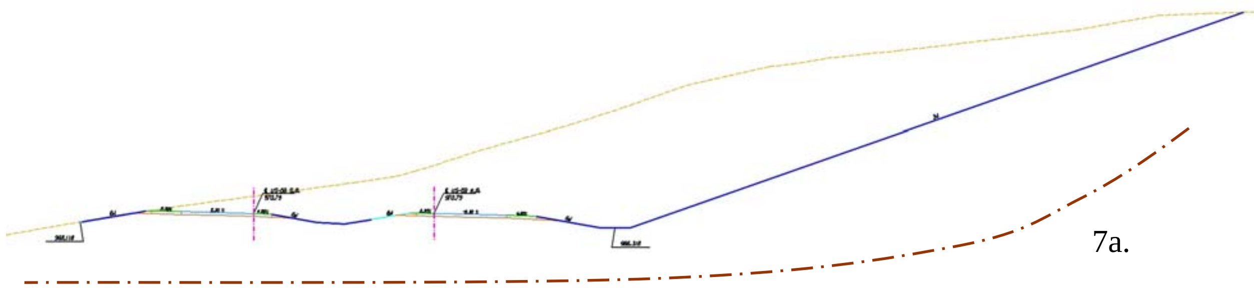


Figure 6. Alignment one was the original one chosen by road design. It poses several geotechnical challenges; large amounts of excavation along the side of the hillslope, exposing sixty or more feet of Lawrence Shale Formation, and crossing over three paleo-landslides highlighted in the figure above. Alignment two was proposed by the Geology section of KDOT to alleviate the problems caused by excavating the side of the hill by lowering the potential for catastrophic failure and avoiding the aforementioned landslides.

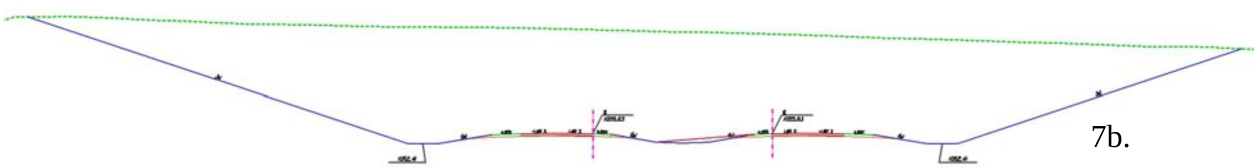
Alignment 1 posed numerous problems:

1. Exposing unstable shale to that depth will cause problems with unloading associated rebound.
2. Removing one side of this hill slope opens up the potential for catastrophic failure of the road way (Figure 7a).
3. Alignment 1 places the roadway over paleo-landslides developed in the Lawrence Shale Formation, further proof of its unstable nature.
4. Stabilization efforts for the backslope would be extremely costly with this alignment.

These potential problems and associated remediation efforts required to stabilize the new roadway were addressed with design. We had developed several methods of stabilization for the complicated problems on this section. This section of the alignment would have required subgrading approximately 5 feet of shale for 2500 feet, stabilizing the paleo-slides with rock anchors or other methods and approximately 187500 square feet of slope stabilization to prevent rotation failures in the backslope. The estimated cost of these stabilization efforts was in excess of 30 million dollars. The immediate reaction was that the Geotech Section should have forewarned the other departments about these problems. Our solution was to try and lessen the impact of traversing the Lawrence Shale by adjusting the alignment.



7a.



7b.

Figure 7a. and 7b. These cross-sections are examples of proposed road cuts on Pleasant Grove hill. 7a. shows the potential slide plane along which a catastrophic failure could take place when clays within the Lawrence Shale are wet and fail, which they have a proven history of in the area. 7b. is an example of the new alignment's road cuts, this will confine the Lawrence Shale within the backslopes lowering the potential for rotational catastrophic failure and total loss of the roadway.

Alignment 2

The Geotech section suggested Alignment 2 (figure 6). This alignment would eliminate the catastrophic rotational failure potential and removed the alignment from crossing the paleo-landslides. Alignment 2 has additional complications but they are ones that easier to deal with. Here KDOT will be excavating approximately twice as much material to construct the proposed gradeline. Rebound of the shale will require us to subgrade over 4 feet through the cut sections and approximately 44500 feet of slope stabilization will remain. The problems with maintaining the angle of the backslope is a concern with this alignment. All backslopes through the Lawrence Formation will be constructed on 3:1 or flatter configuration (figure 7b). The natural slope of the

Lawrence Formation is 6:1 so, some small failures can be expected in the backslopes. Hopefully these will be minor and be mainly a maintenance issue in the future. The expected increase in cost to the project with Alignment 2 is approximately 6.7 million dollars.

Conclusions

To date we have advanced over 1000 investigation borings and will likely require 500 additional borings to complete the Geotechnical investigation. We have a contract with Kansas State University to develop a photographic guide with descriptions of the problematic sections of the Lawrence Formation. This will aid our inspectors in recognizing material that should be wasted verses material that can be manipulated and utilized in the construction of embankments. The proposed cost of construction for this project is in excess of 60 million dollars. The re-alignment of US 59 has no alternative but to cross these geologically poor engineering materials, therefore it has become our task to minimize the costs and maximize the performance of the new roadway. At the completion of our investigation we will have exceeded our 30 day schedule by approximately 1 year.

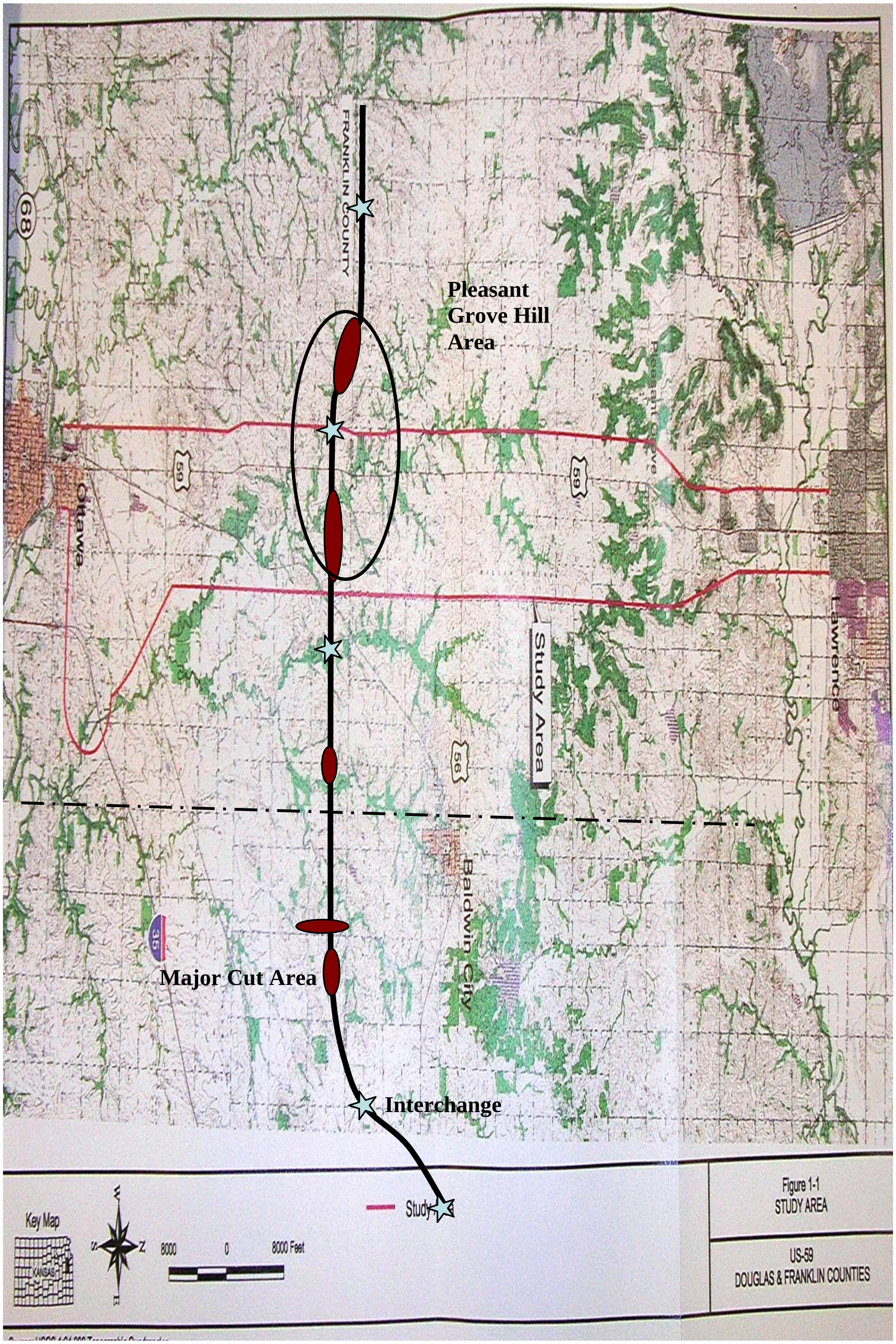


Figure 8. Map showing the study area for the entire project. Areas of signifigant cuts are highlighted by maroon ovals, stars represent an interchange.

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Geology and Landslide Activity, on Arizona SR 89A, in Jerome, Arizona

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Abstract

The outcrops, surrounding Jerome Arizona and underlying SR 89A, has been the subject of geological scrutiny ever since the late 1880s. Rich volcanogenic ore bodies, more than 1750 million years old, are found in the highly deformed Precambrian host rocks of the area. Crystalline basement rocks are unconformably overlain by flat-lying Paleozoic sediments and Tertiary basalt lava (550-250 and 15-10 million years old, respectively). Subsequent faulting has dropped the younger strata against Precambrian rocks along the Verde fault that passes through the upper part of Jerome, sub-parallel to portions of modern SR 89A. A former 1550 foot high fault scarp, generated about 8 million years ago along the western margin of the Verde graben, or rift valley, shed clay-rich fault gouge and weathered bedrock down slope onto the down-faulted Tertiary volcanic rocks on what was to become the site of Jerome. Millions of years of erosion of the fault scarp ultimately exposed the nearby Precambrian ore bodies and the debris from that erosional episode produced the layer of weathered colluvium that buried the unstable clay layer lying beneath the Jerome town site.

Portions of the town site and SR 89A, therefore, are constructed on a steep hillside on top of an unstable substrate that is subject to slope instability. Two other uninhabited sites along the trace of the Verde fault to the south of Jerome have also experienced similar scale landslides in geologically recent time. Other parts of the SR 89A alignment in the Jerome area were built across precarious bedrock outcrops and along the edge of deep canyons. These are prone to rock fall and retaining wall failures that could occur during high intensity rainfall events.

This paper will provide a brief review of the geologic conditions in the area and highlight some of the historic landslides still affecting on modern highway planning and construction. Additionally a recent project history will be summarized where these geologic conditions played a critical part in the analysis of a proposed rest area site.

Location

The project area is located in central Arizona, in Yavapai County on the northeastern slopes of the Black Hills Mountains. This is approximately 100 miles north of the state capital in Phoenix Figure 1.

Highway 89 A traverses the decent from Mingus Mountain via Deception Gulch through the community of Jerome and down the pediment slopes of the Verde Valley to the Town of Clarkdale. The elevation of the project is roughly at 5150 feet.

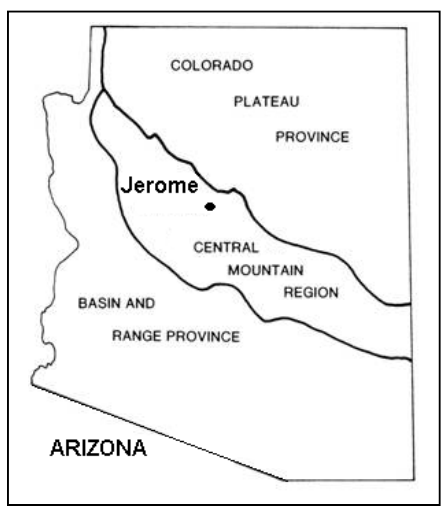


***Figure 1*. Project Location Map**
(From Nations and Stump 1981)

The area is physiographically assigned to a central mountainous region known as the Transition Zone, which is south of the Colorado Plateau and north of the Basin and Range Province. The area receives approximately 20 inches of precipitation per year.

This paper will concentrate on the conditions that exist near milepost 344 in the town of Jerome. This area is locally known as Clark, Main and Hull streets

Highway History

SR 89A (formerly SR79) was one of the earliest highway construction projects to receive federal aid funding in Arizona. Its completion led to a drastic reduction in the time it took to travel from Prescott (the county seat) to Jerome. A trip that once took 5 hours now takes 1 hour.

In 1919, Federal Aid Project No. 12, facilitated construction of the new state highway route 79, (now 89A) with a 20' wide roadway and 6 to 10% grades. The work on the two-mile segment just outside of Jerome was considered the most difficult roadway construction ever attempted by the state of Arizona. Its total cost was reported as $123,785.15, a rate of approximately $62,000.00 per mile. (An exceptional amount of money at the time.)

The route surveyors utilized pioneer trails previously constructed by the local government and mining interests. It is probably no coincidence that the present highway travels past many of the old mining structures and workings. The highway provided a direct down hill route to the smelter located near present day Cottonwood Arizona, avoiding the steep uphill trail over Cleopatra Mountain.

Construction through the town of Jerome dominantly consisted of integrating the existing local streets of Clark and Main into a manageable route through the residential and business district. (Figure 23) The town upgraded the gravel roadway surface to match that of the state highway system in 1920. In 1928 a special bond election facilitated paving all the main streets in Jerome.

In 1937 the highway was widened to 30 feet. (Figure 2) At this time stone retaining walls and concrete culvert extensions were constructed with the help of the Works Progress Administration (WPA). Additionally sidewalks and new retaining structures were added to existing gravity rock rubble retaining walls in the vicinity of downtown Jerome.

Figure 2. 1937 stone retaining wall construction (Clements 2003)

The present alignment, with only minor surface improvements and local detours, is essentially the same today.

However nine of the town’s retaining structures have had to be replaced or reinforced in the last 60 year. Others are being monitored because of deteriorating conditions.

Landslide activity in the Town Of Jerome

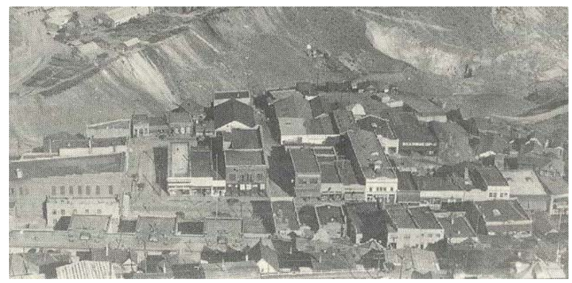
Initially slope stability issues were not a concern because the first rudimentary structures were few and lightly constructed. The local streets were constructed on a steeply inclined clayey gravelly soil and bedrock surface. The town's inhabitants utilized local soil and rock materials to construct terraced building lots, terrain contouring streets, rubble retaining walls and land fills on the steep slopes of Cleopatra Hill.

With the introduction of large scale mining activity the local population grew rapidly creating a great demand for buildings and transportation infrastructure. After three large fires destroyed significant portions of the community, the town incorporated itself in 1889 with a building code requiring brick and masonry construction. The town also initiated construction of a municipal water line and fire service.

This phenomenal growth is depicted in the two photographs displayed below contrasting the community in the 1880s and in the 1930s.

Figure 3. Jerome Main street circa 1880 (Young, 1964)

Figure 4. Jerome Main Street circa 1930 (Clements 2003)



The first signs of instability was reported in 1898 with subsequent events in 1903, 1912, 1913, 1914 and 1916 throughout the town site. Persistent damage to the town's water, sewer and fire fighting systems increased the saturation of the local soils. In 1913 instability in the area, which later became a major landslide, was reported in the vicinity of Clark, Hull and Main streets. In 1924 appreciable ground movement reoccurred in this area. Increasing horizontal and vertical movements continued until in 1937. At that time a 3-acre section of downtown Jerome had been deemed structurally unsound and many affected buildings had to be torn down. (See Figures 5, 6, & 8) Instability in the area continued well into the 1940s. (See Figure 9 & 10 for location of the landsides that occurred during this era.)

Figure 5. Main Street (from Young 1964)

Figure 6. Present day Main Street, exhibiting missing buildings in landslide affected area

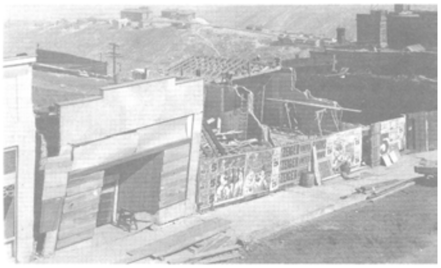


Figure 7. Exhibiting structural damage to building on Main Street circa 1937 (Now SR 89A) (Jerome Historical Society)

Figure 8. Exhibiting failed foundation on Hull Street. circa 1938 (Now SR 89A) (AZ State Archives)

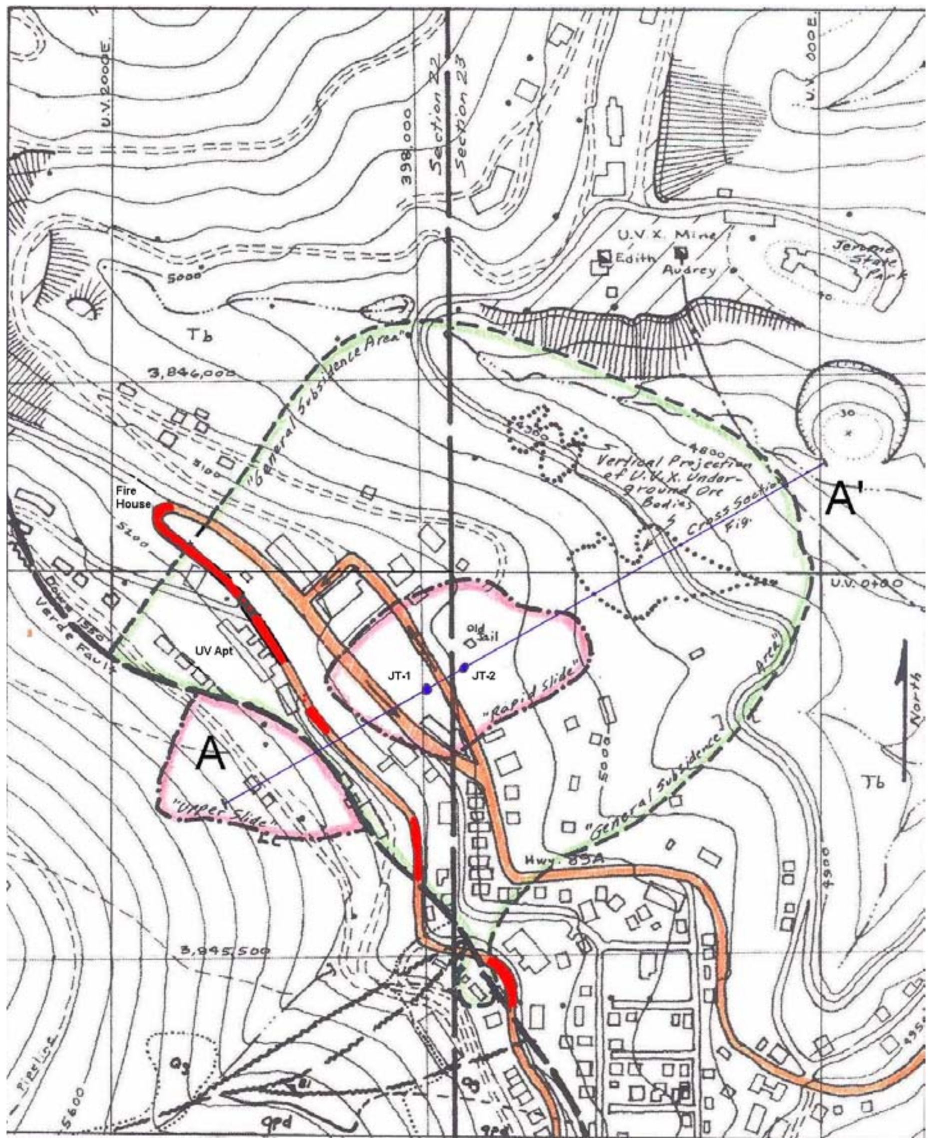


Figure 9. ***(From Lindberg 2002)*** This base map displays the outline of historic landslide areas of Jerome plus other pertinent information.

The location of the landslide cross section, (A – A'), passes through the exploratory borings JT1, JT2. See Figure 20.

The main strand of the Verde Fault, with a net drop of 1550 feet of the northeastern block, is shown with a long solid dashed line.

Outlines of the "Rapid Slide", "Upper Slide", and General Subsidence Area" are taken from, Kiersch, 1988.

The red outlined sections of SR 89 depict retaining walls that were replaced in recent years by ADOT

Figure 10. This picture foreground displays the 3 acres of downtown Jerome affected by the landslide

Figure 11. Fill and regrading of Main street was a common occurrence during the 1930's and 1940's

Considerable reconstruction of Main Street occurred throughout this era. In 1941 the state took over road maintenance that had grown beyond the towns capacity to repair. As late as 1948 the highway through the town of Jerome was regraded roughly every 6 months. Responsibility for the devastation of the business district focused on the local mining industry.

Brief Geology of SR 89A in the vicinity of the Jerome Landslide

There are four major groupings of lithologies that are present in the project area

Recent to Miocene Colluvium Deposits / with Landslide Debris

Investigations in the project area have identified a sequence of dense reddish brown sandy to clay-rich gravel beds deposited upon relatively solid bedrock outcrops of Tertiary and Precambrian lithologies. Fault gouge combined with chemical and mechanical weathering of bedrock materials in the vicinity of the Verde Fault Zone are presumed to have produced these high to moderately high plastic clay deposits that contain relic fragments of parent rock. Overlying this material is an assemblage of colluvium and a mixture of modern fill soils, local rocks, and mine waste and construction debris.

Figure 12. ***(right, above)*** sandy clay and gravel overlain by highway fill exposed in retaining wall cut in the vicinity of the UV Apartments on Clark Street. (SR89A)
Figure 13. ***(right, below)*** Landslide disturbed soils and fill in the vicinity of sliding jail, down slope side of landslide mass

Tertiary Volcanic and Sedimentary Deposits

Figure 14. Hickey Basalt Outcrop

Hickey Basalt of Miocene age has been displaced downward along the Verde fault zone and forms the dominant rock type that underlies the Jerome town site. In the upper part of town basalt bedrock lies below surface colluvium within the highway right of way, from the Jerome fire hall (at the switchback where Main and Clark Streets join) to a point 1300 feet to the south.

For the next 800 feet the centerline of SR89A intermittently intercepts the plane of the Verde fault. Hickey Basalt lies downhill to the east and Precambrian bedrock is exposed to the west. The Hickey formation is composed of fine to coarse-grained olivine basalt lava flows that contain a few intercalated conglomerate beds. Beneath the lava flows, well below the surficial colluvium deposits, is a pre-Hickey conglomerate of Miocene age that contains well-cemented Precambrian and Tertiary rock clasts. Except for several feet of surficial weathering, and thin layers of tuffaceous material, the Hickey Basalt lava flows are generally quite competent.

Paleozoic Sedimentary Deposits

Only two Paleozoic age rock formations are encountered in highway road cuts. To the west of the bold Precambrian rock exposures in Deception Gulch and Hull Canyon, SR 89A crosses down-faulted Devonian Martin Dolomite. The Martin is a relatively competent, tan colored, stratified sedimentary formation containing conspicuous blocky fractures. Further uphill on the flank of Mingus Mountain the route crosses into conformably overlying Mississippian Redwall Limestone, a relatively pure calcium carbonate formation, locally quarried for the manufacture of cement. The Redwall is typically very competent and grey in color but it locally displays karst solution cavities and cemented collapse breccias with an iron oxide pigmentation. Continuing uphill, (towards Prescott), the road crosses an unconformity and passes into Miocene Hickey Basalt. These same two Paleozoic formations are exposed in highway road cuts down slope from Jerome in down-faulted blocks below the Verde and Bessie faults. Neither of these Paleozoic sedimentary formations is exposed in the immediate Jerome town site area.

Precambrian Rock Types

There are two different Precambrian volcanic lithologies exposed on the hillside of the Verde fault immediately above Jerome. Relatively massive Lower Cleopatra Rhyolite crops out to the west of the fault plane and forms the massive outcrops of Cleopatra Hill above town. This extrusive lava flow has a granitoid texture with conspicuous quartz phenocrysts set in a groundmass of sericite (hydrothermally altered feldspar) and silica. About 1300 feet to the south of the Jerome fire hall (located at the sharp switchback where Main and Clark Streets come together) SR 89A enters Deception Rhyolite along the uphill side of the Verde fault. This rock has a similar composition to the Lower Cleopatra but it lacks the quartz phenocrysts. Both of the Precambrian rhyolites display a brownish weathering patina but actually are a pale green in color when exposed in fresh outcrops. High angle reverse faulting on the ancestral Verde fault during the Laramide Uplift (~75 million years ago) and more recent normal offsets along the reactivated Verde fault (~8 million years ago) generated clay-rich fault gouge formed from these rhyolite rocks. During the latter phase of faulting weathered rock and clay fault gouge from these rocks contributed to the surficial instability of the Jerome town site.

After leaving Jerome and passing westward through precipitous Deception Gulch and Deception Rhyolite outcrops, the road re-enters Lower Cleopatra Rhyolite where the more subdued landscape of Hull Canyon is exposed. Beyond the newly constructed vista point the road continues through Lower Cleopatra Rhyolite and traverses across unaltered Upper Cleopatra Rhyolite, Upper Sequence rhyolite flows and breccias, and finally into submarine-emplaced Grapevine Gulch turbidite debris flows before reaching the Warrior fault plane. Beyond that the road encounters Paleozoic strata

Geology of the Jerome Landslide Area

The area where the Jerome surficial landslide has occurred has undergone dramatic geologic changes over the past 75 million years. The general “head” of the historic slide area as shown in Figure 9, lies at the edge of the Verde Fault plane along the base of Cleopatra Hill. Precambrian age Cleopatra Rhyolite is exposed for more than 800 feet in elevation directly above the plane of the Verde Fault that passes through the upper part of the Jerome town site. Tertiary age Hickey Basalt, dated at 10-15 million years old, has been dropped approximately 1550 feet against Precambrian basement rocks along the Verde Fault. Most of the town of Jerome is situated on top of surface colluvium and Hickey Basalt bedrock.

The Verde fault plane has an attitude of about –60 degrees to the northeast and lies well below the landslide area. The most recent period of faulting took place approximately 8 million years ago. The ancestral phase of the Verde fault however, experienced a period of high angle reverse motion during the Laramide Uplift that occurred ~75 million years ago. During that time the northeast block of the fault plane was raised several hundred feet higher than the southwestern side. The ancestral Verde fault plane that was reactivated ~8 million years ago when it experienced a normal drop to the east-northeast. As a result, the rocks within the Verde fault zone have been severely crushed and subjected to deep weathering over the past 75 million years. Underground mine exposures and drill logs, show that flexures in the Verde fault zone can vary from a few inches to thirty or more feet wide locally. Figures 15 & 16 shows the present day result of this prolonged period of activity.

Figure 15. (right) Schematic cross-section through Jerome area graben fault.

Panel A shows an early phase of fault offset and subsequent erosion. The Verde fault forms the southwestern margin of the Verde graben. Much of the Upper left portion of the fault scarp has been eroded away during this early phase.
Panel B shows conditions following renewed graben development and rapid displacement of the Verde fault that displaces the various strata to their present day elevations. The wide slab of Crushed Cleopatra Rhyolite and fault gouge exposed along the fault scarp erodes quickly and the material is re-deposited at the base of the slope on the top of the Tertiary Hickey Basalt. This material, composed of Precambrian clasts of rhyolite and fault gouge, forms the unstable colluvium under the central part of Jerome.

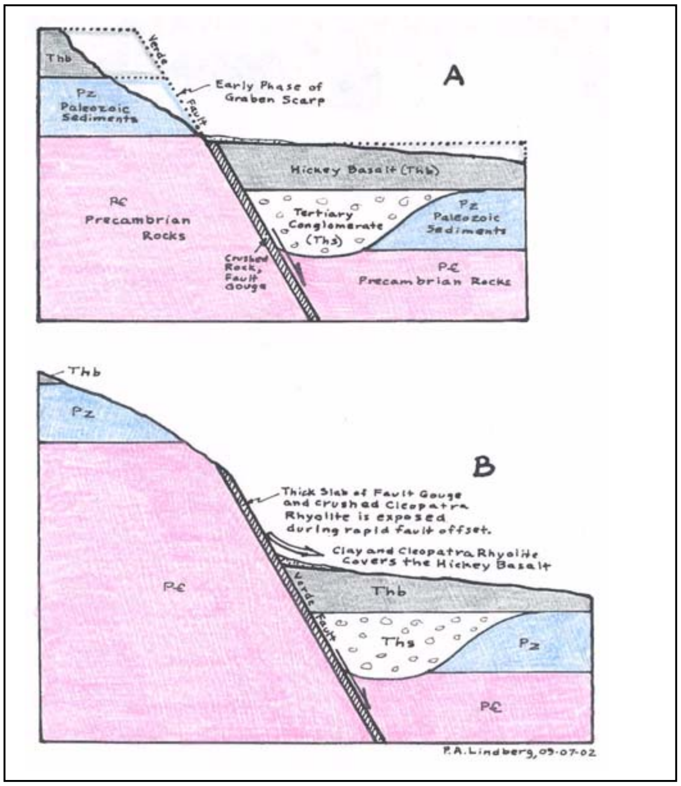
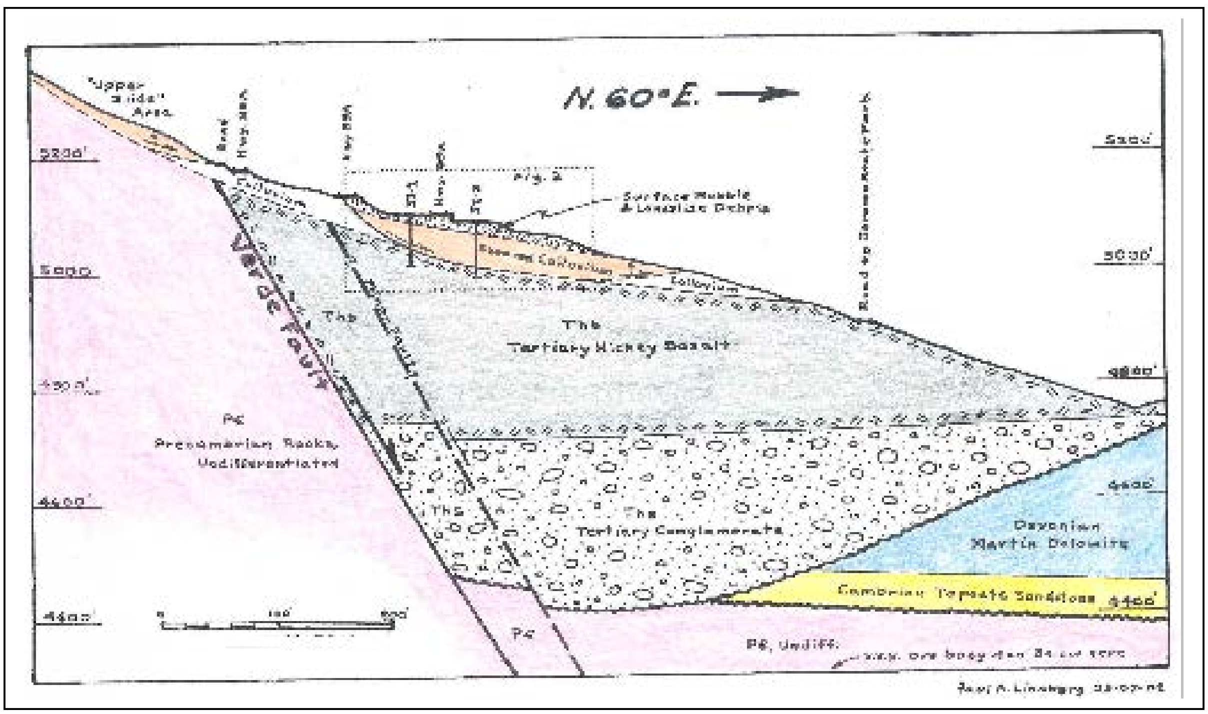


Figure 16. (below) Cross-section through the Jerome Landslide area looking northwesterly. Within this figure the dotted outline displays the position of project drill holes and the area of investigation. The current study strongly suggests that the entire landslide and subsidence area lies above the down dropped Hickey Basalt and is located within a thick colluvium that covers this part of the Jerome hillside.



The thin-skinned landslide deposits appear to have affected only surficial clay-rich and unconsolidated surficial colluvium deposits that lie above solid bedrock. Presently there is no observed field evidence for movement of underlying bedrock, or for recent displacement along the Verde fault, within the project limits. As such, the unconsolidated material upon which the central part of Jerome was constructed is underlain by unstable colluvium that is vulnerable to heavy precipitation events and down slope creep. The unconsolidated material in the immediate landslide area may have originally been up to 100 feet deep but to the north and south of the landslide area this thickness is much less. It is probable that the landslide occurred directly over the deepest part of a bedrock gully that has been filled with the thickest amount of clay-rich colluvium. This has exacerbated the potential for very local landslide movement. The source of clay is believed to have been derived from fault gouge and decomposed, hydrothermally-altered, sericite-rich Lower Cleopatra Rhyolite that was once exposed up slope on the scarp face of the Verde fault plane directly above Jerome. The Verde graben formed during the Basin and Range period of extension about 8 million years ago.

Geotechnical Investigation of the Jerome Landslide Site

In 2002 the town of Jerome partnered with ADOT's Roadside Development Section to re-develop the site as a rest area and parking facility for the tourists who visit the area. Since the area had had a long history of instability it was felt that a sub surface investigation was in order to try to understand the mechanisms that led to the 1930's era landslide. To accompany the study a detailed look into the records of the town was undertaken to find collaborating information, which described the landslide in the past.

ADOT through its on-call consultant, Terracon, conducted a field investigation at the site in which the subsurface soil, bedrock, and groundwater conditions were explored. An abstract of historical documents were compiled and a summary of potential causes of landslide activity was ascertained. Additionally a slope stability analysis of the present site conditions was conducted.

Surface Conditions

The site of the proposed rest area is an existing gravel parking lot for Lower Park in Jerome. There is an existing landscaped slope from Main Street down to the west side of the parking lot. Although the head scarp for the landslide was located further west near the middle of Main Street, the slope generally depicts the extent of the head scarp for the 1936 landslide.

The existing site conditions of the area in and around the 1936 landslide generally consisted of: the buildings west of Main Street, Main Street, the slope from Main Street down to Lower Park, Lower Park and associated parking lot, a portion of Hull Avenue, the newly constructed dry block retaining wall, the Sliding Jail Park and associated parking lot and basketball court, recently placed fill below the Sliding Jail Park and parking lot, and the existing ground surface down slope of the recently placed fill.

The majority of the buildings along the west side of Main Street show evidence of cracks that have been repaired at sometime in the past. Concrete retaining walls at various locations behind these buildings show cracking at the face of the wall. Otherwise, the buildings and retaining wall structures appear to be in relatively good condition considering their age.

The ground surface down slope of the recently placed fill is hummocky and irregular in appearance near what appears to be the central portion of the old landslide. The undulations are on the order of 1 to 5 feet in height. Vegetation in this area is thick with grasses, cattails, bushes, and trees. The ground surface was soft in places indicating groundwater apparently comes to the surface in these areas. Along the flanks of the landslide the ground surface appears to have been bermed.

Soil and Bedrock Conditions at the Site

The subsurface soils encountered in the test borings generally consisted of clayey gravel fill materials and colluvium soils overlying the Hickey Basalt. The upper clayey gravel fill soils varied in depth from 16 to 28 feet below the existing surface. The upper part of the fill material contained distinctive evidence of construction debris consisting of smelter slag, broken concrete and Cleopatra Rhyolite. The colluvium soils generally extend to depths of 65 to 85 feet in (or varied in thickness from 50 to 55 feet). The borings were terminated at depths of 16 to 25 feet into the Hickey basalt.

Field Test Results

Standard Penetration Tests for boring No. JT1 indicates the fill materials are generally poorly compacted. The underlying soils from 16 to 24.5 feet varied from loose to medium dense in relative density. Remolded in-situ soils were also observed in this zone. The fat clay soils below a depth of 24.5 feet are generally hard in consistency

Standard Penetration Tests for boring No. JT2 indicates the fill materials are generally well compacted, however, the penetration tests appear to represent the presence of gravel and cobbles, and not the surrounding matrix. The fat clay soils at a depth of 35 feet are generally soft to medium stiff in consistency. The clay and sandy soils generally increase in consistency and relative density below a depth of 40 feet.

Laboratory testing indicated the moisture content of the subsurface soils varied from 6 to 23 percent. The moisture of the upper 25 feet of materials in the test borings is generally low. The moisture content of the materials below a depth of 25 feet is relatively high, ranging between 15 and 25 percent. The clay fraction ranges from 10 to 50 percent. The clay soils generally have very high plasticity indices ranging up to 56.

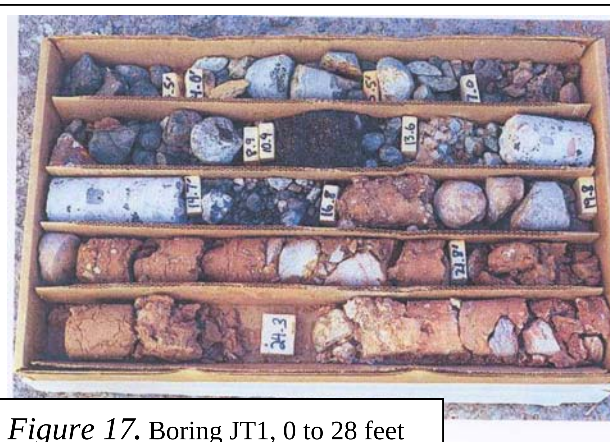


Figure 17. Boring JT1, 0 to 28 feet

Figure 18. Boring JT1, 28 to 40 feet

Results of Investigation

Based on an analysis of the historical conditions, site geology and the geotechnical investigation, seven conditions could have contributed to the landslide of 1936.

1. Low shear strength soils in the near surface for the development of failure planes at shallow depths
2. Shallow groundwater concentration, caused by heavy rainfall events, leaking water utilities, and surface water accumulation near the head scarp.
3. Assimilated seismic events from large-scale blasting activity (over an extended number of years) may have contributed to keeping the landslide area in a state of creep.
4. The magnitude 4.5 seismic event of 1931 that also would contribute to creep.
5. Movement along the Verde Fault from blasting or the seismic event and a subsequent potential for change in the groundwater regime due to offset of the Verde Fault.
6. Over steepening of some slopes during building construction, leading to minor movement during the time when the ground relaxed into an active lateral pressure state. This may have led to broken water pipelines in these areas.
7. Reconstructing the local streets and backfilling the head scarp increasing surcharge to the already creeping mass.

Strength Parameters

All the strength parameters for geotechnical analyses could not be established by direct laboratory testing because of poor sample quality. For purposes of the engineering analyses, published correlations were used to estimate strength parameters based on available laboratory test results.

1 **Residual Cohesion:** The values used for the residual cohesion of the soils along the estimated failure plane of the landslide were approximated using a correlation based on moisture content and index properties (Fang, 1991). The anticipated variability of residual cohesion of the soil value was approximated using the variability of the liquid limit test data. For this analyses, an average residual cohesion value of 100 psf was used. The variability was approximated using a standard deviation of 25 psf.

2 **Effective Residual Friction Angle:** Six different correlations were used to estimate the effective residual friction angle of the colluvium soils. Data from all of the samples from Boring Nos. JT1 and JT2 were included in the data set.

The average values and values at two standard deviations away from the average were used with the correlation charts to estimate the range of effective residual friction angles expected based on the laboratory data from the two borings.

For this analysis, an effective residual friction angle of 16 degrees was used together with a standard deviation value of (4) degrees.

Slope Stability Analyses

Stability analyses were performed using the computer program SLOPE/W version 5.11 developed by GEO-SLOPE International, Ltd. SLOPE/W utilizes algorithms to solve the Morgenstern-Price limit equilibrium method of slices. This method satisfies force equilibrium in both the horizontal and vertical planes and also satisfies moment equilibrium. Direction of the resultant inter-slice forces is determined using an arbitrary function. The percentage of the function, λ, required to satisfy moment and force equilibrium is computed with a rapid solver.

For purposes of the stability evaluations, a cross-section (A - A') through the landslide area and the portions of the slope above and below the site of the rest area was developed. A specific failure plane was analyzed based on data obtained from the historical review and our field exploration. The failure plane slopes steeply to the east near the middle of Main Street, becomes planar for approximately 400 feet and slopes slightly up to the ground surface where it day-lights at the surface. The failure plane is parallel with the ground surface and is located at a depth of 25 to 35 feet. The location of the cross section is shown on Figure 20, with a few modifications to the ground surface to represent present day topography across the landslide area.

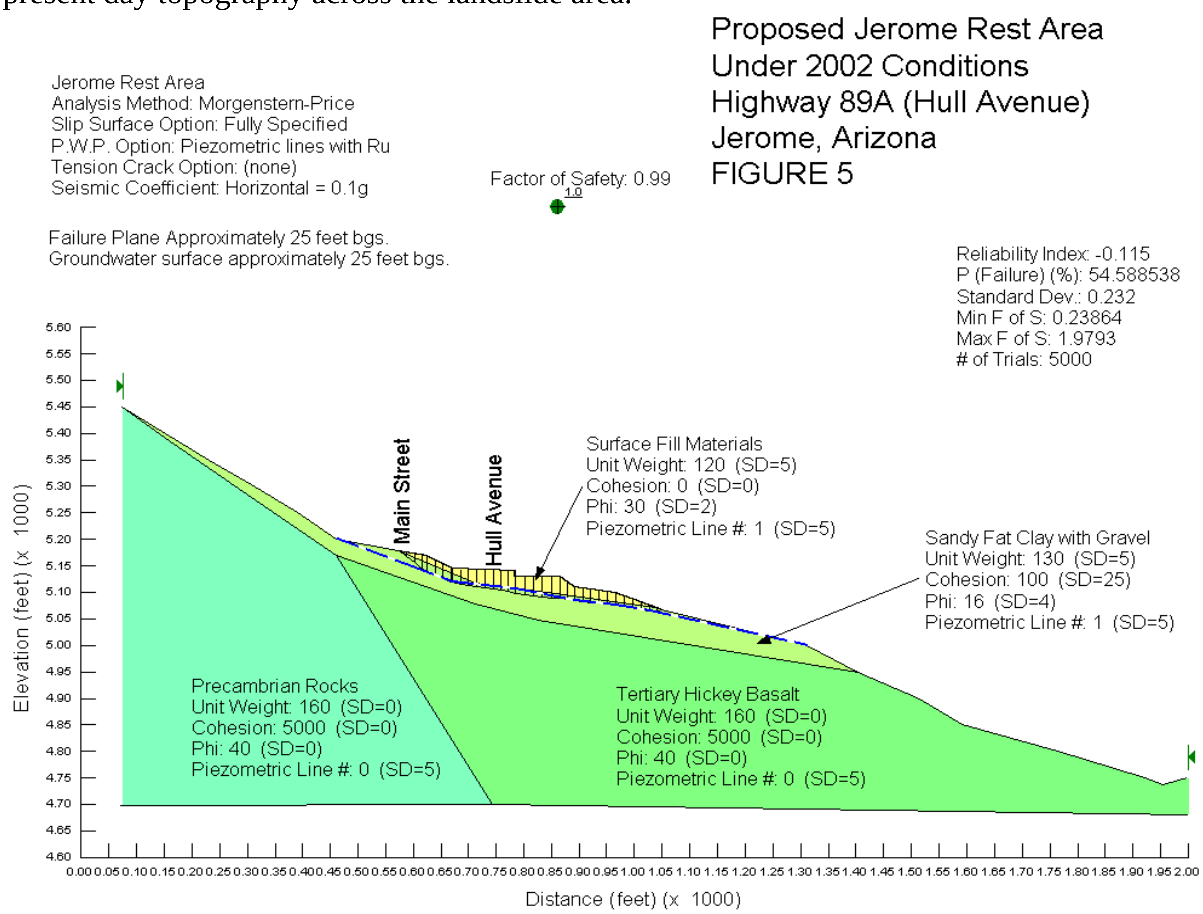


FIGURE 20, Slope Stability cross section; typical.

Slope stability models were analyzed for 1936 conditions and 2002 conditions. The conditions included in the analyses are outlined below:

Parameter	1936	2002
Geometry	as shown on cross section	same as 1936
Failure Surface	specified	same as 1936
Depth to Groundwater	10 feet	25 feet
Friction Angle	16°	16°
Cohesion	0 psf	100 psf
Seismic Coefficient	none	varied

The seismic coefficient was varied under the 2002 conditions to ascertain the sensitivity of the analyses to the seismic coefficient. The cohesion under the 1936 conditions was reduced to zero considering the slide was in motion and the failure surface slightly adjusted until a factor of safety of 1.0 was achieved. The proximity of the factor of safety to 1.0 indicates the correlated residual friction angle, groundwater surface and failure surface assumptions are relatively close to those conditions that continued landslide movement.

Results of the stability analyses for each case and the corresponding calculated factors of safety are summarized in the following table.

Summary of Stability Analyses		
Condition Analyzed	**Seismic Coefficient**	**Factor of Safety**
1936	0.00g	1.0
2002	0.10g	1.0
	0.02g	1.3
	0.00g	1.5

Based upon this analyses, the slope is stable under 2002 conditions provided there are no strong ground motion forces added to the slope and based on current groundwater conditions. In addition, the results of the stability analyses indicate the safety factor is sensitive to the magnitude of the seismic coefficient.

Risk Analysis

The notion of risk is an important aspect of any geotechnical exploration. The primary reason for this is that investigative and analytical methods used to develop geotechnical conclusions and recommendations do not comprise an exact science. The analytical tools are generally empirical and must be tempered by engineering judgment and experience. The solutions or recommendations presented in any geotechnical study should not be considered risk-free and more importantly, are not a guarantee that the proposed structure will perform satisfactorily. What the engineering recommendations do constitute is the geotechnical engineers' best estimate of those measures that are necessary to make the structure perform satisfactorily based on usually limited subsurface information. The purpose of the following paragraphs is to discuss the concept of risk so the owner, who must ultimately decide what is an acceptable risk, can better apply the finding of this study.

As previously outlined, the most critical geotechnical consequence of this study is considered to be slope stability of the landslide area. The stability of a portion of this slope is expressed as a factor of safety. It is important to note the concept of factor of safety is a derived value and not an intrinsic property of the slope. The accuracy with which the factor of safety for a given slope can be determined, is based on a number of factors the most significant of which are listed below:

- Variability of surface conditions
- Variability and type of subsurface conditions
- Validity of the analytical method
- Validity of simplifying assumptions
- Intensity of study
- Certainty of the design loading conditions occurring.

Depending on how well the above factors can be assessed determines what minimum factor of safety would be required to have a reasonable degree of confidence that a failure will not occur. It is the geotechnical engineers' responsibility to assess these conditions and advise the owner as to a minimum acceptable factor of safety.

Theoretically, a factor of safety of 1.0 indicates that a slope is on the verge of instability. Therefore, any lower factor of safety should result in failure and any higher factor of safety should theoretically represent a safe slope. However, due to the uncertainties associated with any geotechnical investigation and the factors discussed in the preceding paragraph, all slopes, even those with factors of safety greater than 1.0, have some potential for failure. The higher the computed factor of safety is for a given slope, the lower its probability of failure. Approaches have been developed to relate computed factor of safety to probability of failure. This approach is called a probabilistic analysis and a limited risk analysis was performed for this study.

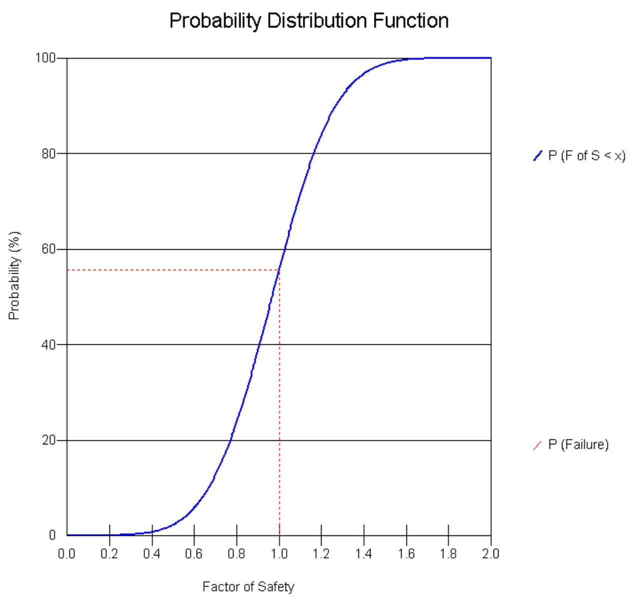


FIGURE 21: Probability Distribution graph graph for seismic coefficient of 0.1g.

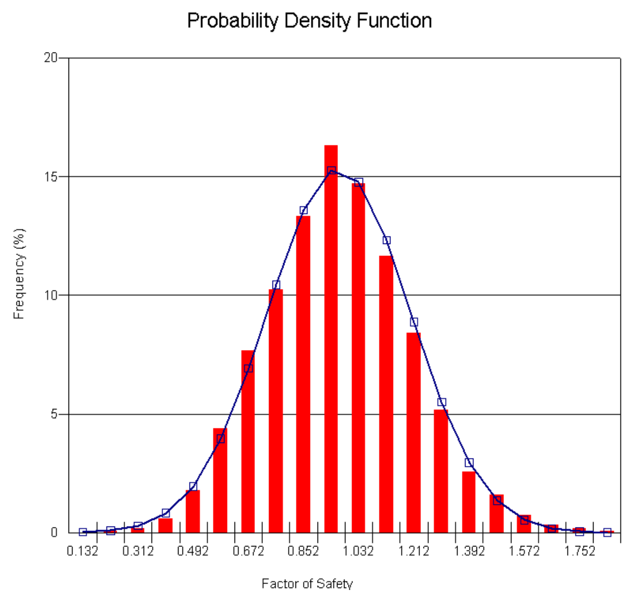


FIGURE 22: Probability Density for seismic coefficient of 0.1g.

The list of parameters that were varied included unit weight, cohesion, friction angle, groundwater elevation, and seismic coefficient.

Conclusions

The results show there is generally a one in seven (15%) chance of slope instability under 2002 conditions when the seismic coefficient is 0.02g. When the seismic coefficient is 0.10g the probability is generally one in two (55%).

The risk of future landslide movement at the site is particularly sensitive to the seismic coefficient used in the slope stability analysis. Though the other parameters when varied do effect the slope stability, their effect is relatively small. When considering future development on this historic landslide, the forecasting of seismic or assimilated seismic events will be the most crucial parameter to acquire accurately.

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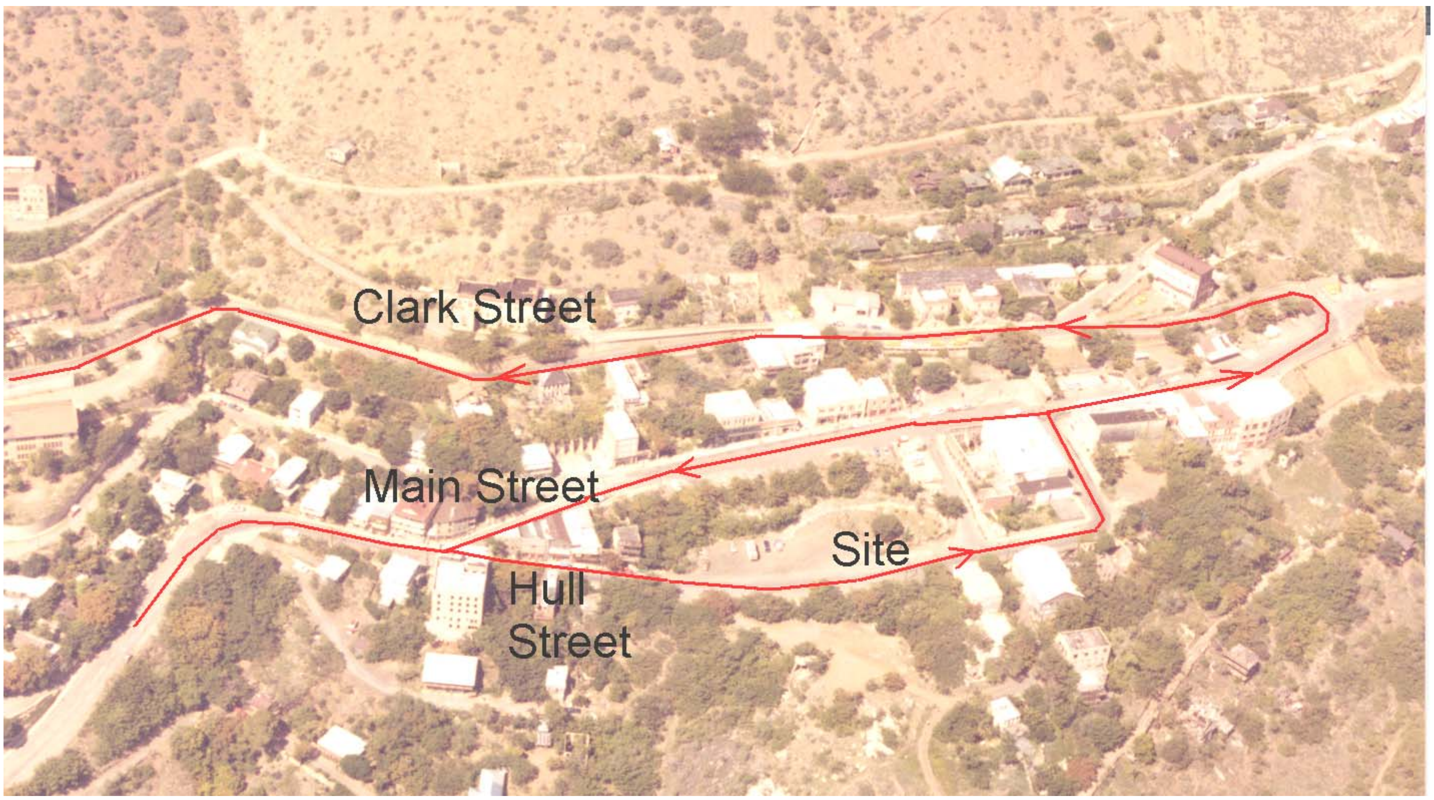

Figure 23. Aerial Photograph of the town of Jerome (1988) displaying the routing of SR 89A through down town. The site of the historic 1937 landslide and the proposed rest area is located between the one-way streets of Main and Hull.

Geophysics and Site Characterization: K-18 over the Kansas River

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Kansas DOT

Many state highway departments are attempting to incorporate geophysical methods into their preliminary geotechnical investigations. The cost of drilling, combined with budget and staff cutbacks, has prompted many highway geologists to try geophysics in order to save money. In addition, the Federal Highway Administration actively promotes the use of geophysics for the same reason.

In 2001, the FHWA reviewed the geotechnical policies of the Kansas Department of Transportation and made recommendations about how to improve efficiency. One of these recommendations was to use geophysics in preliminary investigations, to supplement the drilling program. Up until then, KDOT had limited experience with geophysics, and most of that was simply in response to a problem with a particular section of roadway.

At that time, the geologists at KDOT began looking for a place to experiment with different geophysical methods. We wanted to find a characteristic bridge project, preferably one with good as-built elevations so that we could easily compare the geophysical results with known information. We also wanted to find a location where drilling would be restricted in some way. The most troublesome drilling situation in Kansas involves shallow, broad rivers. These streams are too deep to cross with track-mounted drills, and too shallow to use a floating platform. In addition, a very detailed geological profile is needed at such a crossing. Scour has to be evaluated, and drilled shaft design must be considered. Heavy loading of isolated piers requires that the geologist learn a great deal about strata in the middle of the channel. It is this situation that KDOT Geology concentrated its interest in the use of modern geophysics.

In 2004, the Abilene Regional Geology Office received plans for a proposed bridge over the Kansas River on K-18 between Manhattan and Junction City. The bridge is to be built alongside the existing structure, which was constructed recently enough so that detailed geology information is available. The overbank portion of the bridge site was easily accessible with drilling equipment, and core holes were advanced on both banks. The riverbed itself, however, is nearly 800 feet wide, sits 10 feet below the bank, and is braided with sand bars. In addition, Milford Lake, the largest reservoir in the state, is about 12 miles upstream, and fluctuations in the water flow are common. Drilling for information at piers 2, 3, and 4 of the new bridge was out of the question.

The project location is in the heart of the picturesque Flint Hills, which are composed of alternating layers of limestone and shales—classic lower Permian cyclothem stratigraphy. Kansas River alluvium typically consists of quartz sand with lenses of clay and silt. We felt that the contrast of material properties at the bedrock/alluvium contact—limestone and hard shale underlying sand—might lend itself to good quantitative geophysics data that would pinpoint the top-of-bedrock elevation across the channel.

The next step was deciding which method to use. Seismic refraction and resistivity were the two methods that we believed could give the information we needed. Geophysicists at the Kansas Geological Survey were consulted for their advice; the consensus was that both resistivity and refraction had their advantages and limitations in the setting that we proposed. We finally decided to try both methods and learn for ourselves.

KDOT has a standing contract for drilling services with a consulting firm that also performs geophysical investigations. A proposal from this company was submitted and accepted. KDOT would pay $33,000 for resistivity and seismic refraction surveys and their interpretations.

In early December, 2004, data was collected along the centerline of the proposed bridge. Two geophysicists and a technician were sent by the consultant, and KDOT geologists helped. The water level was agreeably low, and a boat was needed for only the main channel of the river, along the south bank. This channel was about 80 feet wide and 4 to 5 feet deep, with a swift current. Other channels were minor and could be waded.

The seismic survey used a 24-channel Seistronix RAS-24, with geophones set at 10-foot intervals. The energy source was a technician or KDOT geologist, via a sledgehammer and a small striking plate. The seismic refraction survey covered 950 feet.

The resistivity survey used a SuperSting-R8 meter, manufactured by Advanced Geosciences, Inc. This resistivity meter uses 56 electrodes, and the electrode spacing was also 10 feet. “Marsh phones” were used underwater; otherwise the survey used aluminum rods to transmit and receive electrical signals.

Placing geophones in the main channel for the seismic refraction survey proved exciting. A small aluminum boat, held by a rope strung across the river, held workers. A PVC tube was used to seat the geophones in the sand beneath the rushing water. This was very time-consuming and only somewhat successful—a few phones never did couple with the bottom correctly. An initial spread across the channel gave marginal results. The source was then moved to the other bank, and a back-shot was attempted. Just as the crew was ready for the back-shot, a sheet of ice floated down the channel and tore loose all remaining geophones from the bottom of the channel.

The resistivity survey also had its snags. The original meter that was rented by the consulting firm would not initialize, and a replacement had to be shipped overnight from AGI. Once the second resistivity meter was in use, the survey went well. All together, three-and-a-half days were spent on data acquisition.

In the subsequent report, our consultant expressed little confidence with the refraction results. Background noise was blamed for the poor data—wind, road noise from the nearby bridge, and vehicle activity at the Fort Riley military base. Despite data stacking, this noise was reportedly still troublesome. The resistivity data was much better, and the geophysicists were optimistic about the results. The report contained interpreted top-of-bedrock elevations for both surveys.

It is our experience that the alluvium/bedrock contact beneath even the largest rivers in eastern Kansas is generally planar. Occasional scour holes are found, but the rock is usually too resistant to give dramatic weathering differences across a site. At the K-18 crossing, drill holes on opposite banks (separated by about 800 feet) showed that the contact varied by less than 3 feet. We expected that at least one of the geophysics methods would clearly show this contact between such different materials as sand and hard Permian bedrock.

We were quite disappointed. The interpretations for the bedrock contact of both the resistivity and refraction data showed irregular lines with unrealistic high and low points. In the riverbed itself, the refraction data yielded differences of up to 16 feet; the resistivity results varied by up to 20 feet. The two methods diverged by as much as 25 feet toward the north end of the channel area. Additionally, both methods seemed to reflect the topography, as the interpreted bedrock contact followed the surface elevation up both banks and across sand bars between channels. In short, the results were useless for our work.

At this point, we once again consulted with geophysicists at the Kansas Geological Survey to help us figure out what had happened. Did we make a mistake in assuming that these methods would work at this location? Was there something irregular about the equipment or approach used to gather the data? Was the interpretation questionable?

According to KGS scientists, the computer programs used to interpret the data may be responsible for the shaky results. There are a handful of different programs available to help geophysicists handle refraction and resistivity surveys, and each has its own biases. Interpretation of geophysical data is the most important step of a successful survey, and the one which an outsider (such as the author) is least likely to understand. As a result, we have now requested the raw data from the consultant, and will forward it to geophysicists at the KGS for review.

We remain hopeful that either seismic refraction or resistivity can be useful to us at KDOT. As for the K-18 bridge, our design had to go forward without any information from the riverbed area. In March, we submitted a drilled shaft design for the piers based on as-built information from the present bridge and from core holes on each side of the river. During construction, we will supplement our information with additional core holes, once the contractor has built the needed pads in the river. At that point, we can adjust our design somewhat, if necessary.

GEOPHYSICAL METHODS FOR SITE CHARACTERIZATION OF OFFSHORE HIGHWAY STRUCTURES

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Evaluating surface and subsurface conditions along proposed or existing highways, bridges or causeways that span or are located adjacent to waterways is often done with a limited amount of data. Subsurface information for these projects is primarily acquired using traditional methods, such as boreholes, test pits or divers, which tend to be expensive and limited in the area of coverage. A number of geophysical methods, including seismic reflection and refraction, sidescan sonar, and ground penetrating radar are extremely cost-effective and obtain significantly more data than can be acquired using traditional intrusive methods. These geophysical methods provide continuous profiles of the subsurface stratigraphy, which can be invaluable for selecting the best location for geotechnical boreholes and map potential geohazards such as submarine slides and filled or open scour holes. Offshore investigations conducted for two proposed highway bridges, the Lake Washington floating bridge and the Knik Arm Bridge (Cook Inlet, AK), and one existing bridge (Potomac River) used a combination of geophysical methods to map the water depth, detect and map debris (sunken vessels), map the thickness of unconsolidated sediment, determine the depth to bedrock and identify potential scour holes. The results of the geophysical investigations will be used to select locations for obtaining geotechnical borings and to design the anchor systems for the floating bridge, to extrapolate geologic information across the deepwater crossing of Knik Arm and to establish the depth of scour around the bridge foundations on the 14th St. East Bridge. Transportation departments we have worked with are realizing the importance of offshore geophysical methods for (i) site characterization, (ii) reduce the uncertainty of geotechnical conditions thus avoiding designs that may be overly conservative and thus more expensive and (iii) potentially reduce changed condition claims.

INTRODUCTION

Subsurface information for highway projects are primarily acquired using traditional intrusive methods such as boreholes, test pits or with divers offshore. Geophysics, the study of the Earth and its physical and material properties, can provide a cost-effective, non-intrusive approach for initial site reconnaissance and/or detailed investigation (8). The use of surface and borehole geophysical methods for highway projects is well documented and a number of publications are available describing these methods (1, 2, and 6). What's not readily available are technical information and case histories on the use of geophysical methods offshore for bridges, highways, or causeways that span or are located adjacent to waterways (9).

Geophysical methods based on acoustics, including seismic reflection and refraction, sidescan sonar, or electromagnetic such as ground penetrating radar, provide a cost-effective means to investigate and map surface and subsurface sediment and other geologic features offshore (7). This paper introduces several of these methods and presents three case histories that demonstrate how the use of offshore geophysics provided invaluable information that if acquired by other methods, would be prohibitively expensive.

SOME CONSIDERATIONS FOR CONDUCTING OFFSHORE GEOPHYSICAL SURVEYS

The primary considerations when planning an offshore geophysical investigation are selecting the survey vessel and geophysical instrumentation (Figure 1 and 2). The selection of these two parameters depends on a number of factors including the location of the study area (ocean, river, lakes, etc.) project objectives, how the data is to be used, size of the survey area, expected subsurface geology and possible operational limitations (tides, currents, obstacles, shallow water vessel traffic, etc.).

The use of acoustical (sound) methods is the most common means of performing offshore surveys. The following is a brief discussion of the offshore instruments followed by a discussion of three offshore projects that used two or more of these methods.

Navigation

Most offshore surveys use the global positioning system (GPS) for determining the position of the survey vessel and the location of each data point. The GPS navigation system obtains position information from satellites with an accuracy of approximately +/- 20 feet. However, if needed the accuracy can be improved to +/- 2 to 3 feet using differential GPS, which obtains correction from a U.S. Coast Guard beacon, a receiver placed on a shore monument or from a commercial satellite system that broadcasts corrections.

Bathymetry or Water Depth-Precision Echosounder

A survey grade precision echosounder is used to obtain very accurate measurement of the water depth (Figure 2). The data is obtained with a high frequency transducer (100 to 200 kHz typical) that resolves very small changes in the relief of the bottom (+/- .03 feet). For bathymetric surveys of large areas or complex bottom conditions bathymetric data is often acquired with a multibeam system which can provide depth measurements along a swath that is equal to 3 to 10 times the depth of water.

Surficial Features-Sidescan Sonar

Sidescan sonar produces a plan view image of the bottom to either side of the survey vessel trackline (Figure 2). As the survey vessel moves along the track line the sonar transmits a fan-shaped acoustical signal that "paints" a narrow image of the bottom for each transmit/receive pulse. The full width of this image, which is controlled by the operator, typically ranges from 150 to 500 feet. The resulting image can be used to map lateral variations in soil type (sand, gravel, mud etc.), identify exposed bedrock and geologic hazards such as landslides, depressions and surface faults, detect and map the location of cultural artifacts, pipelines and cables, sunken vessels and discarded debris.

Subsurface Features-Reflection Methods

Subsurface reflection methods are used to image features, including sediment layers, bedrock, discrete objects such as pipeline, boulders etc, that are below the bottom. Subsurface reflection data can be obtained using acoustic sources or ground penetrating radar which transmits an electromagnetic signal (Figure 3). The choice of instrument or method (acoustic or EM) depends on the geologic characteristics of the bottom and the expected depth of penetration. GPR (40 to 100 MHz) and high frequency acoustic systems (3 to 10 KHz) can be expected to penetrate 5 to 30 feet of sediment and lower frequency acoustic systems (400 Hz to 1 KHz) can achieve 50 to hundreds of feet of penetration. The reflected energy is received at the water surface by a hydrophone or in the case of GPR and subbottom profilers by the transducer that transmitted the signal. The received signal is converted to an electrical signal and printed in real-time on a graphic recorder that displays a continuous cross-section image of the subsurface along the survey trackline. The data can also be stored digitally for later processing, analysis and presentation.

Velocity of Sediment-Seismic Refraction

Seismic refraction is used to determine the compressional velocity of the material on and beneath the bottom. The data is also used to model the depth to sedimentary horizons, provide qualitative classification of material and identify and map the top of bedrock. The refraction data can be obtained by using a towed hydrophone array and continually discharging the energy source while the boat is underway. The refraction data is acquired and displayed on a conventional land seismograph.

CASE STUDIES OF OFFSHORE GEOPHYSICAL SURVEYS

Three examples of typical offshore investigations conducted for highway related projects are now discussed. The three studies were performed in entirely different environments and required different considerations with regards to survey vessel, instrumentation, and operations for achieving the project objective.

Lake Washington SR 520 Floating Bridge

Lake Washington is a 25-mile-long, 4-mile-wide, deep-water, glacial lake located immediately east of Seattle, Washington. Two bridges cross the lake (I-90 and State Route Highway 520 corridors) and because of the water depth,-over 150 feet- these bridges float on pontoons that are anchored to the lake floor. It is estimated that the remaining useful life of the floating portion of the SR-520 Bridge is 20 years. Therefore, the facility must be replaced before 2020. The Washington State Department of Transportation is evaluating several alternatives including a 4-lane and 6-lane bridge running parallel to the existing bridge (Figure 4).

Project Objectives

The objective of the geophysical survey was to obtain information that would be used to assist in selecting a final location for the new bridge and anchor systems (6). The specific information requested included accurate water depth, classification and thickness of the lake floor sediment, and identification of debris on the lake floor. This information was critical for designing the anchoring system, for selecting locations for geotechnical boring where the bridge will be pile supported and to identify potential geohazards or conditions that might impact or delay construction.

Survey Operations

The investigation was performed using differential global positioning system (DGPS), a precision echosounder and swath bathymetric system, side scan sonar, subbottom profiler, and high resolution seismic reflection system. The geophysical data were collected on a series of parallel transects (6) that ran east-west along the proposed corridor at a interval of 75 to 100 feet

Summary of Survey Results

The results of the investigation were presented on a series of AutoCAD maps showing bathymetric contours, surficial features including lateral distribution of sediment and location and description of cultural artifacts, thickness of fine-grained sediment and a plan and profile of the interpreted surface and subsurface geology along the centerline.

The bathymetric contours were relatively regular (parallel with no rapid changes) with the exception of a localized feature at the base of the slope on the west end. The contours suggest the presence of a submarine slide. This feature was very evident on the side scan sonar data.

Surficial features, identified with the side scan sonar, were the apparent submarine slide and three large sunken vessels located within the center line of the proposed corridor (Figure 4). The sidescan data also provided information for identifying and mapping the lateral distribution of sediment (silt, sand, cobbles, and boulders), and aquatic plants. The sunken vessels were of considerable interest and two additional studies, first with a remote operating video system and secondly with deep-water divers, were requested to determine if they were of historical significance or if they contained potential contaminants.

The interpreted subbottom profiler (SBP) and seismic reflection data suggest that the main part of the lake, in water depths from 180 to 200 feet, is covered with at least 150 feet of post-glacial sediment.

This includes approximately 40 feet of fine-grained lacustrine sediment underlain by up to 100 feet of glacial outwash sediment. The glacial/lacustrine sediment thins to approximately 70 feet on the east end of the survey area. The shallow water and slope areas are interpreted to consist of over-consolidated glacial sediment with some localized surface deposits of fine-grained silt and mud. There was no evidence of subsurface faults or submarine slope failures on the reflection data with the exception of the area previously discussed. The subsurface data are being used to select locations for geotechnical borings.

Knik Arm Bridge Crossing-Cook Inlet Alaska

Cook Inlet extends north and then east from the Gulf of Alaska. It divides into two arms: Knik Arm on the north and Turnagain Arm on the south, with Anchorage on the peninsula between the two. The Knik Arm Bridge and Toll authority is studying the feasibility and cost for constructing a bridge across Knik Arm providing easy access from the City of Anchorage to the east shore of Knik Arm (Figure 5). During early stages of this study a number of offshore borings were obtained to provide geotechnical and geologic data on the crossing. The borings were done from a jackup barge that was unable to work in the deeper water along the center of Knik Arm and was also unable to drill to bedrock. The Alaska Department of Transportation subsequently requested a geophysical investigation to obtain this information.

Special Considerations and Operation Difficulties

Cook Inlet and its two arms have extreme tidal range (30 to 35 feet) producing large tidal currents and at low tide large areas of the two arms are exposed mud flats. Because of the currents and shallow water it was necessary to use a survey vessel with considerable power, enough space to carry the primary geophysical equipment and backup equipment, and have a shallow draft to work in the nearshore area.

The large tidal currents restricted the survey time to approximately two to three hours around relatively slack water conditions.

Survey Operations

The geophysical data were acquired with a precision echosounder, high-resolution reflection system (sparker) and deep penetration reflection system (airgun). The two acoustic systems, with different output characteristics, provided information on the upper 200 feet (sparker) and deeper information with less resolution to a depth of 800 feet (airgun).

The survey consisted of running a series of parallel transects across the inlet spaced at an interval of approximately 500 feet. A second set of transect was run perpendicular to these spaced at an interval of 2000 feet. Navigation was provided with a differential GPS. The entire operation required under three days..

Summary of Results

The bathymetric data, which were acquired on wide line spacing, were used to produce a reconnaissance level contour map. The water depth within the survey area ranged from –5 feet to -90 feet referenced to mean lower low water. At the base of both slopes are two relatively deep (85 feet) depressions. It is not known if these depressions are related to the geology, hydrodynamics and scour or a combination of both.

The data were interpreted using seismic facies analysis. This method identifies various reflection patterns (uniform horizontal reflectors, discontinuous reflectors, chaotic reflectors, etc.) on the graphic records and assumes they are characteristic of a particular lithology or depositional environment. For example, a geophysical facies on the record composed of continuous, thin layers suggests fine-grained sediment deposited in a low-energy environment. Sediments deposited in a high-energy environment usually produce very strong reflections (dark layers on the records) and the reflectors are often discontinuous. The availability of borehole data (data was available from 11 boreholes) makes the interpretation process considerably easier and more reliable.

Information on subsurface features, seismic facies, and stratigraphic boundaries from both data sets (sparker and airgun) were integrated and used to produce interpreted profiles or cross-sections for each of the survey transect (Figure 5).

In the shallow water areas the near surface deposits are interpreted to be predominantly sand and sandy gravel and where there was limited subsurface penetration the deposits are very dense, and contain cobbles. In the deep-water central area the seafloor is mantled with recent marine deposits of loose sediment that typically ranged in thickness from five to 40 feet. Underlying the surficial deposit is approximately 100 feet of dense to very dense silty sand that is underlain by hard, gravelly clay. This sediment is deposited on what are interpreted to be sand and gravel and marine or glacial marine deposits. There was no evidence on the reflection data to suggest the presence of bedrock at a depth less than 900 feet.

14TH STREET BRIDGE

The 14th Street Bridge carries I-395 and US-1 across the Potomac River, joining Arlington, Virginia to the District of Columbia. The original bridge, built in 1906, was replaced by the Rochambeau Memorial Bridge in 1950, which carries the northbound traffic, and the George Mason Memorial Bridge in 1962, which carries the southbound traffic. The Rochambeau Bridge is a single bascule span structure consisting of 15 piers. Significant cracking has occurred in both the stone and the mortar on several of the piers and is believed that these failures may be in part due to the development of scour holes around the piers.

Project Objective

The objective of the geophysical investigation was to attempt to image the river bottom and find evidence of open or filled scour holes or depressions along the bridge piers or bents. To achieve this objective several subsurface geophysical methods, including subbottom profiling (3.5 kHz), seismic reflection profiling (1 kHz) and ground penetrating radar (100 MHz) were evaluated on-site to determine which instrument would be the most effective. The seismic reflection system provided the best penetration and resolution and therefore was selected for the study.

Survey Operations

The survey profiles were run adjacent to the piers. Since the overhead bridge structure interfered with reception of the satellite signals it was not possible to use DGPS for navigation. However, good horizontal control was achieved by using chalk to mark stationing on the piers at 10 feet intervals. As the boat profiled along each pier a mark was placed on the data at the stationing and at the start and end of each bent or pile structure.

Summary of Results

The seismic reflection data indicated the presence of several shallow, subsurface reflectors and open and filled depressions along several of the piers (Figure 6). The depressions often extended 10 to 30 feet downstream of the pier. The maximum depth below the riverbed of the filled depressions was approximately 5 to 6 feet. Rip rap or other hard bottom material prevented seismic reflection from detecting subbottom layers near several of the piers. The deepest depressions were found near piers 10 and 11, which are located near the west side of the bridge

SUMMARY OF OFFSHORE GEOPHYSICAL INVESTIGATION METHODS

Offshore geophysical methods provide a rapid and cost-effective means for obtaining surface and subsurface data. When planning an offshore geophysical investigation it is extremely important to clearly define the objective of the study and the desired deliverables and to obtain all available data regarding site conditions. All three parameters may have a significant effect on whether the investigation is successful. Geophysical data from terrestrial, borehole and offshore investigation may be able to provide quantitative and definitive answers and therefore achieve the objective of the study. However, their real value is often in helping to plan a cost-effective site investigation that includes traditional geologic, geotechnical and engineering methods.

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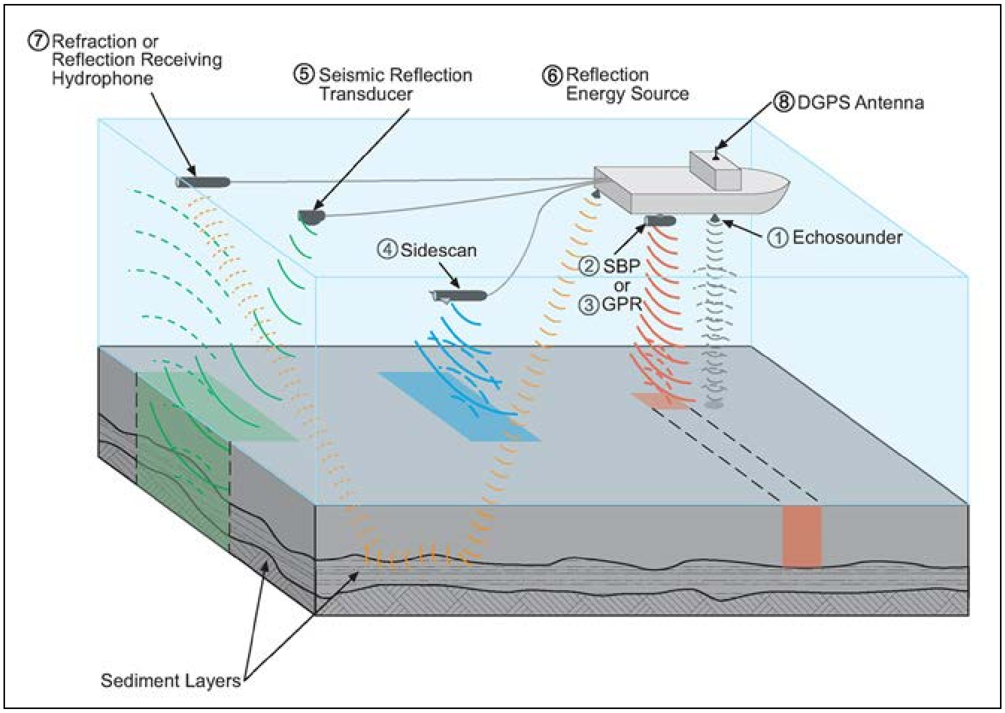
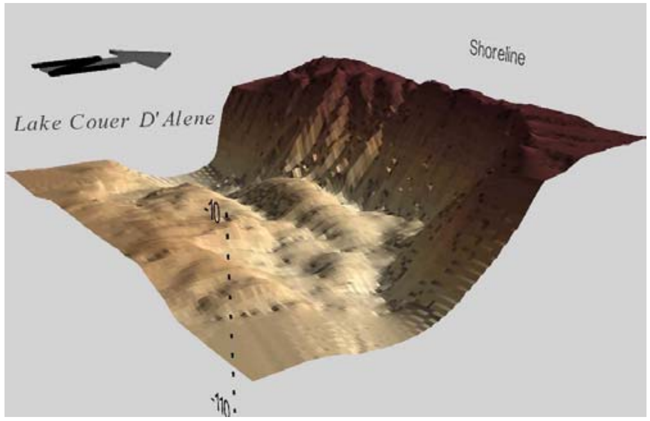


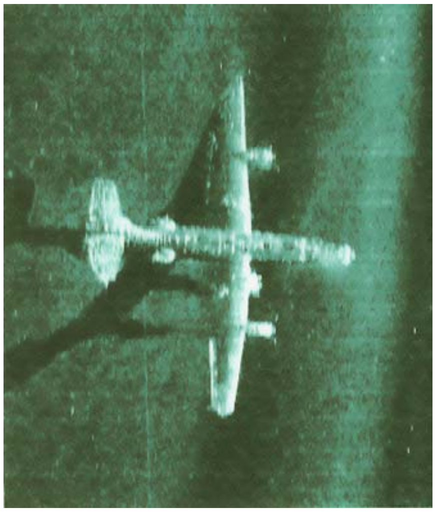
No.	System	Range or subsurface depth	Application
1	Echosounder	1 ft to 1,000 ft	Water depth, construct bathymetric map of bottom.
2	Subbottom Profiler	1 to 50 feet below bottom	Map thickness of fine-grained sediment
3	Ground penetrating radar	1 to 30 feet below bottom	Map thickness of coarse-grained sediment and gas charged sediment
4	Sidescan sonar	20 to 500 foot swath	Map distribution of sediment, debris, biological habitat, aquatic plants, and cultural artifacts.
5	Seismic reflection	5 to 500 feet below bottom	Determine depth to bedrock, map thickness of medium to coarse-grained sediment
6	Refraction energy source	10 to 300 feet below bottom	Determine compressional velocity, model subsurface geologic structure
7	Receiving array		Used with reflection or refraction energy sources
8	GPS	Worldwide	Global positioning system for navigation

Figure 1. Table of geophysical instruments commonly used for offshore investigations.

Small boat operation for offshore survey.

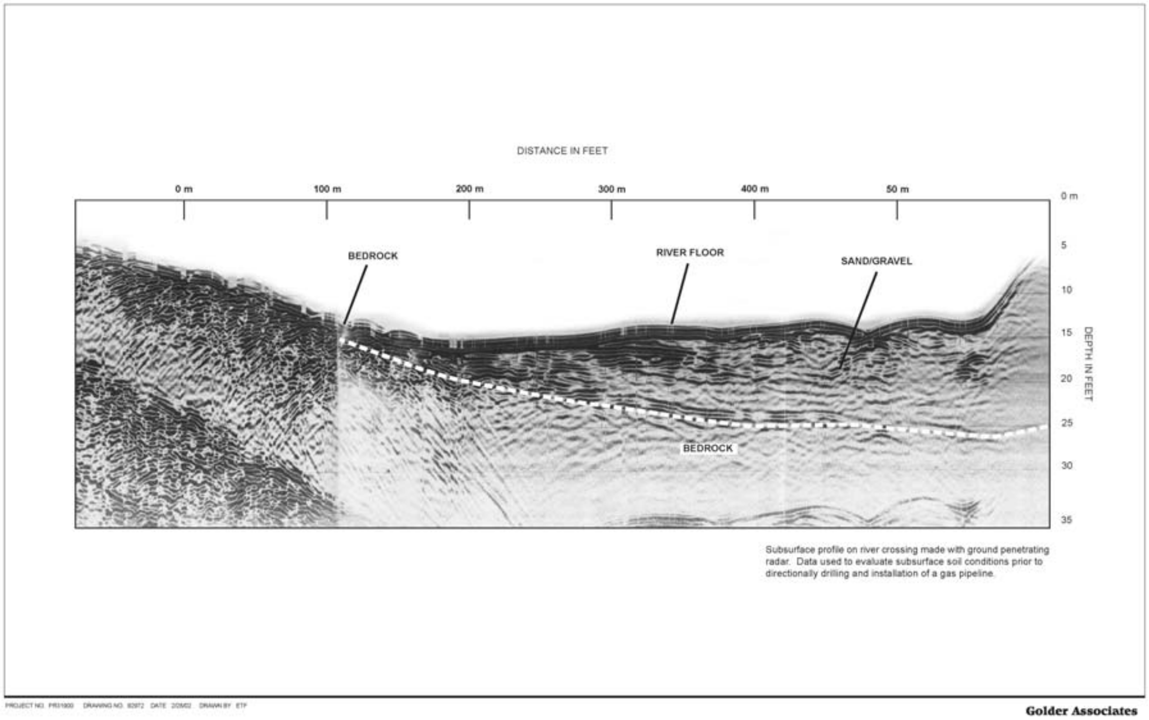


Bathymetric data presented as a isometric view



Sidescan sonar image of sunken plane in 150 feet of water

Figure 2. Examples of offshore geophysical operations and data.



GPR data across a river to evaluate subsurface for a directional drill project.

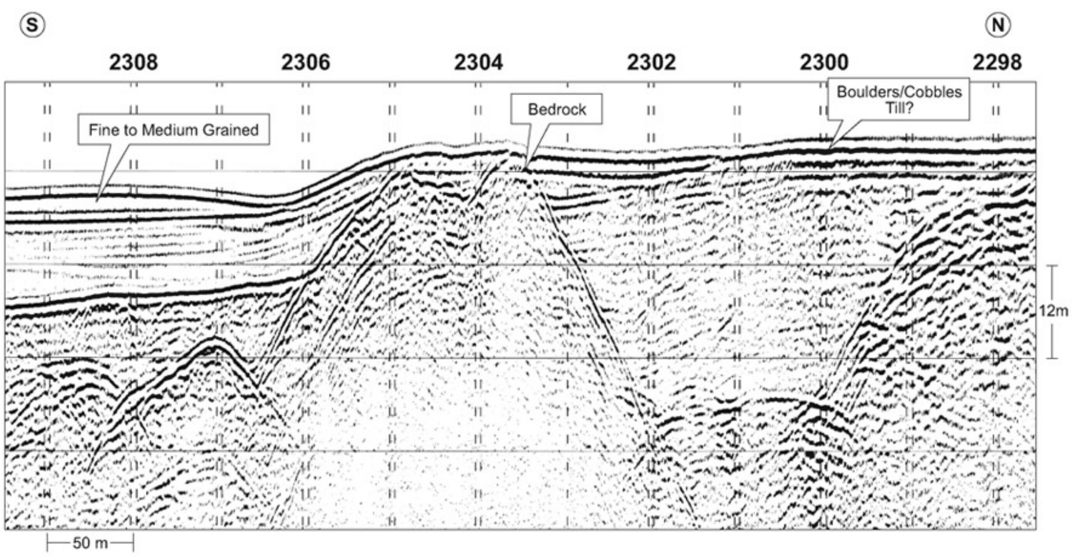
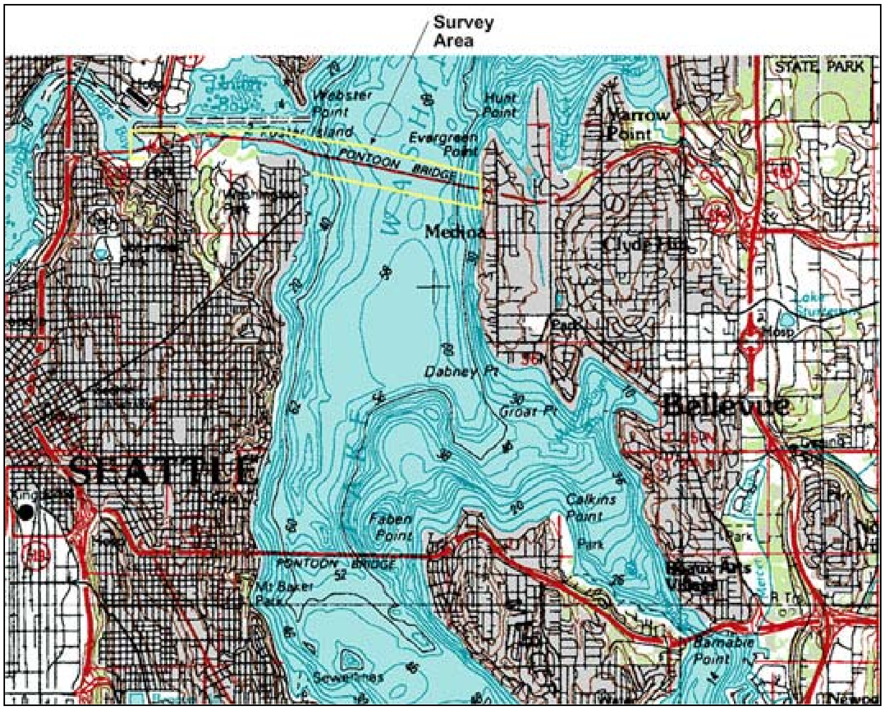


Figure 3. Upper: Reflection data across a river obtained with GPR. Lower: Reflection data obtained with a high-resolution seismic reflection system.



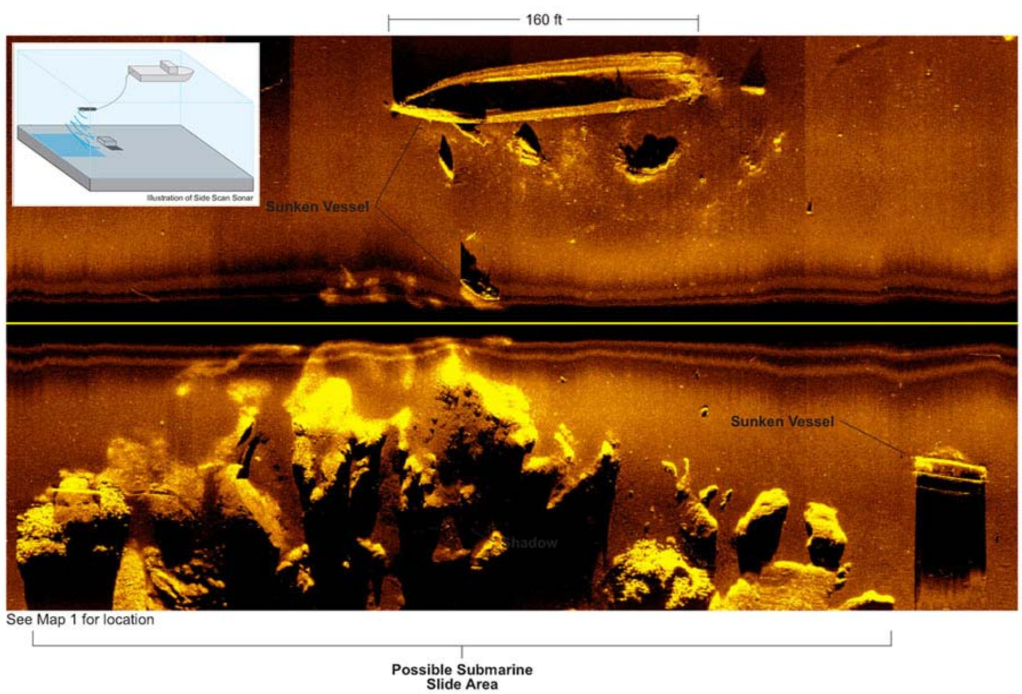


Figure 4. Upper: Location of the two floating bridges that cross L. Washington. Lower: Sidescan image of sunken vessel and landslide along the proposed new bridge alignment, which is the horizontal line down the center of the image

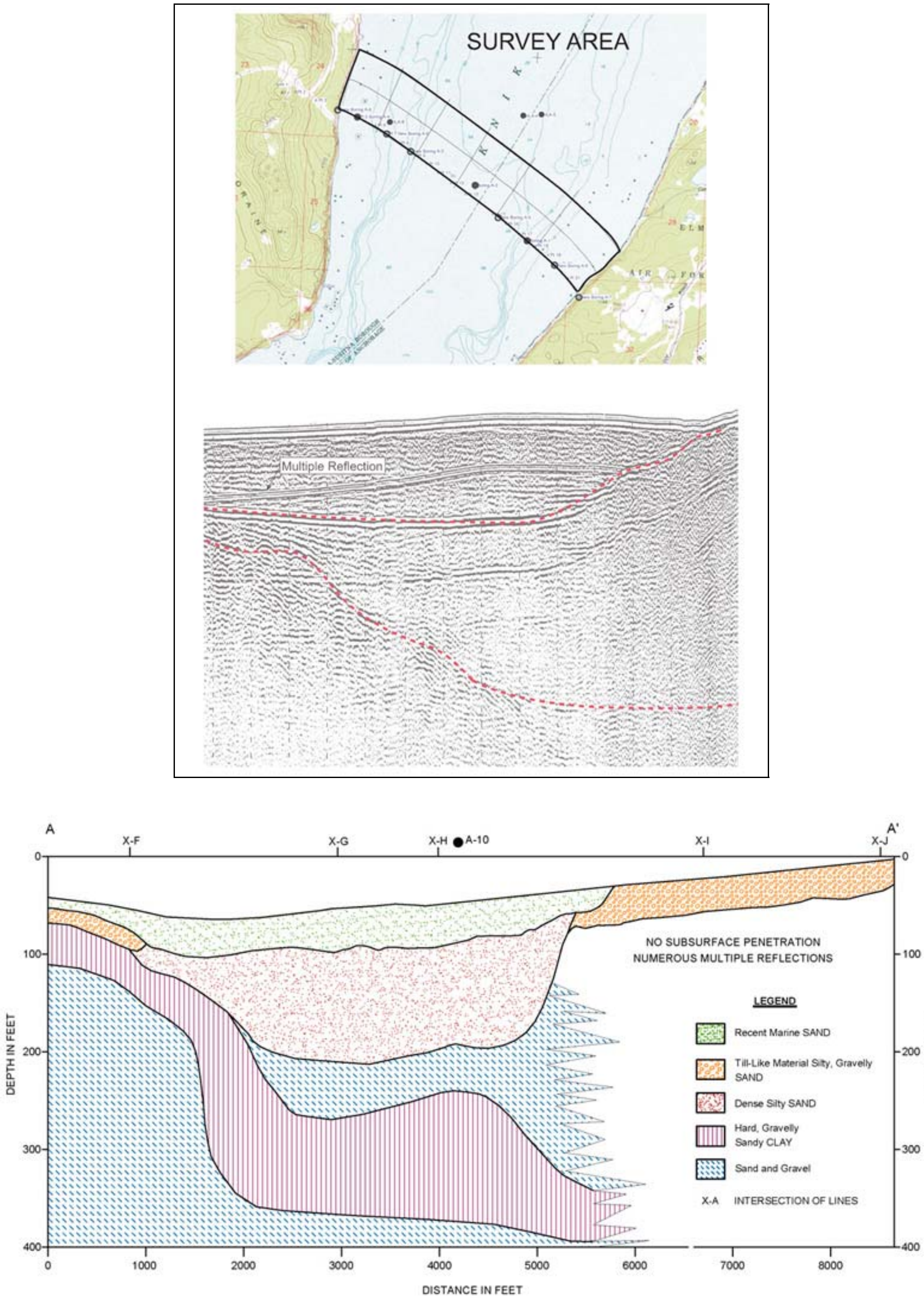


Figure 5. Upper: Geophysical survey area across Knik Arm and example reflection record. Lower: Example of interpreted profile based on geophysical data and geotechnical boring.

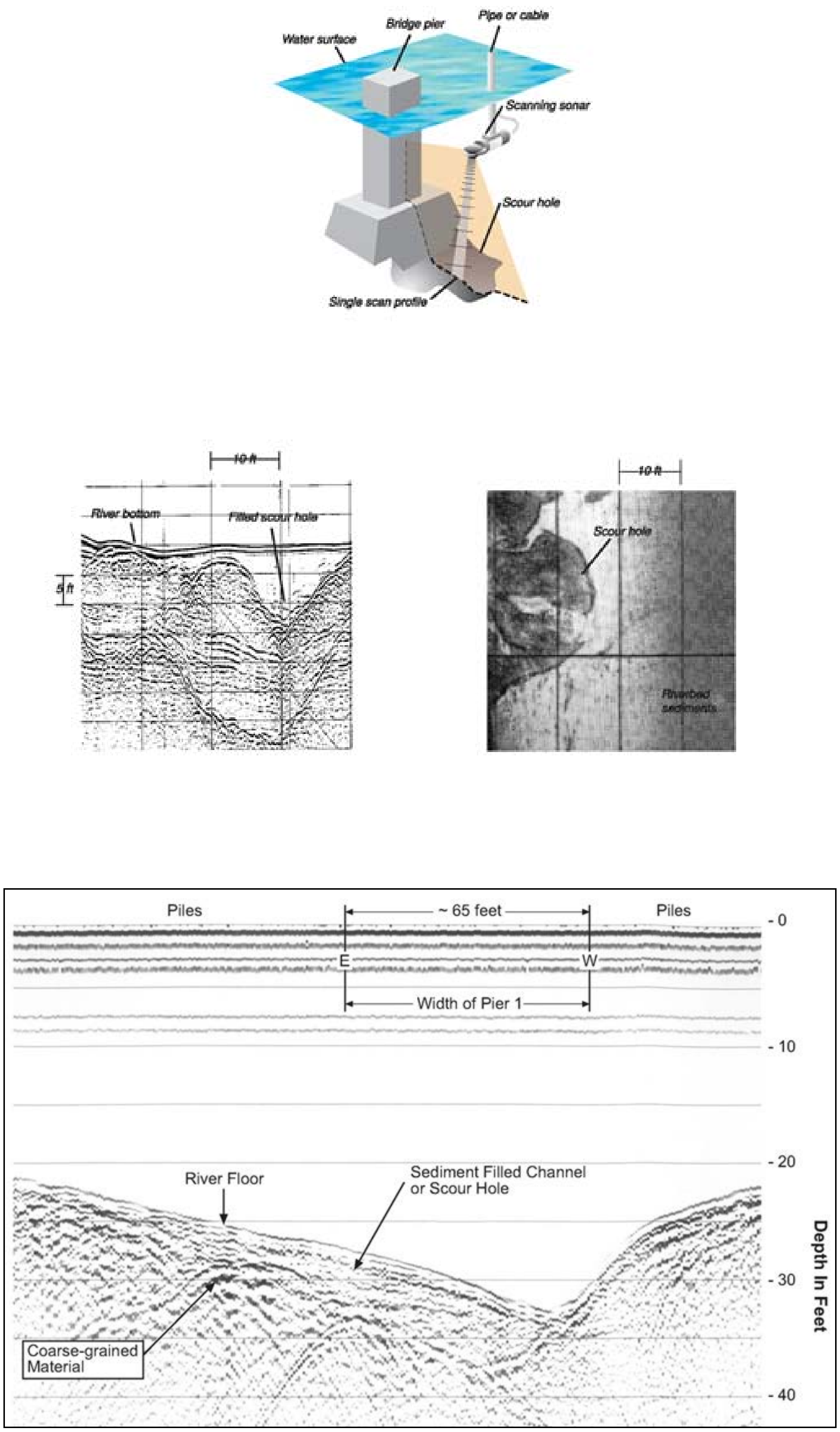


Figure 6. Upper images show a scanning sonar mapping a depression on the river bed. The scan, a plan view of the river bed is below and to the right. The image to the left is a subbottom profile across a sediment filled depression. The lower image shows a filled depression adjacent to a pier or the 14th Street Bridge that crosses the Potomac River.

GEOTECHNICAL CHALLENGES OF THE MON/FAYETTE EXPRESSWAY PROJECT, PA 51 TO I-376, NEAR PITTSBURGH, PENNSYLVANIA

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ABSTRACT

Design and construction of the Pennsylvania Turnpike Commission's 24-mile long northern leg of the Mon/Fayette Expressway into the City of Pittsburgh will complete a large portion of the approximately 64 miles of the total proposed corridor between the Pennsylvania border with West Virginia and the City of Pittsburgh. With a total estimated cost of $1.9 Billion, the PA 51 to I-376 Project meets FHWA criteria for a Major Project – currently there are 15 states with ongoing projects meeting the federal criteria. Divided into 13 separate design sections and design consultant teams, the challenges of designing and constructing a limited-access highway of this size in a heavily-populated, urban environment will be many. From a geotechnical perspective, designers will be required to assess roadway and structure foundation conditions in a historically-populated, industrialized urban land setting subject to past deep and strip mining activity in the Pittsburgh coal; extensive areas of deep colluvium prone to landsliding and stability issues in general; settlement-prone alluvial deposits of the Monongahela River floodplain; and, numerous parcels containing suspected contaminated materials. In terms of structures, approximately 190 bridges, viaducts, walls, and culverts will be designed and constructed, including a major bridge crossing the Monongahela River. This paper will illustrate the key design features proposed for the project and will highlight the geotechnical conditions and challenges facing the designers.

INTRODUCTION

As part of the Pennsylvania Turnpike Commission's (PTC) efforts to design and construct a direct link from the City of Pittsburgh to the West Virginia border to the south at I-68, the Mon/Fayette Expressway (MFE) is one of the largest and costliest on-going highway improvement projects in the country. Comprised of several final design sections, the total 64 miles of four-lane, limited-access toll road has an estimated total cost of approximately $3.3 Billion. Coupled with the adjoining Southern Beltway project, which completes access from the MFE to Pittsburgh International Airport in Pittsburgh's western suburbs, the combined projects will provide approximately 100 miles of roadway at a total estimated cost of $4.3 Billion. Figure 1 is an *Overview Map* illustrating the various final design sections comprising the MFE project and the corridor of the proposed Southern Beltway project.

Three sections of the MFE have already been constructed and are open to traffic. These include the *I-68 to Fairchance*, *Toll 43*, and the *I-70 to Route 51* (70 to 51) Projects. The 15-mile *Uniontown to Brownsville* Project (U2B) is in final design and is slated for construction in 2006. The final segment to complete the corridor is the new 24-mile long section designated as the *PA51 to I-376 Project*. With an estimated cost of $1.9 Billion, the PA51 to I-376 Project is the costliest, and most complex, section of the MFE and meets FHWA criteria for a Major Project, that is a complex project with a cost approaching, or over, $1 Billion, and a high level of interest by the public, Congress, or the [Presidential] Administration. Currently, there are 15 states with 26 ongoing projects meeting the federal criteria. The Record of Decision (ROD) was issued by the FHWA in December 2004. Final design is expected to be completed around 2010.

To complete the design work for the project, the PTC divided the project into 13 final design sections and advertised the work for consultant selection. Figure 2 – *Project Design Sections* – shows the project breakdown of the 13 design sections. Thirteen multi-consultant design teams were selected for each design section from design consultants in Pittsburgh, Altoona/Ebensburg, Harrisburg, and Philadelphia. Consultant teams typically consist of a prime consultant with subconsultants providing geotechnical, surveying, right-of-way, structural, and roadway support services. In total, eight (8) consultant firms will provide geotechnical engineering design services for the 13 design sections.

Design management of the consultants is provided by a team consisting of HDR Engineering, Inc., PBS&J, Inc., the Markosky Engineering Group, and Valley Forge Laboratories, Inc. (DM Team). As a design manager, the DM Team acts as a direct representative of the PTC in establishing technical design procedures, schedules, reporting requirements, QA/QC procedures, and overarching coordination of the work effort between the consultant teams, local government agencies, community groups, railroads, private commercial entities, and property owners. These efforts will be required across all disciplines to provide for uniformity in the design processes and work products, as well as efficient coordination between all potential project stakeholders.

This paper will present some of the key geotechnical factors that will directly affect design of this complex project and illustrate some of the specific design features for three of the more complex design sections on the project.

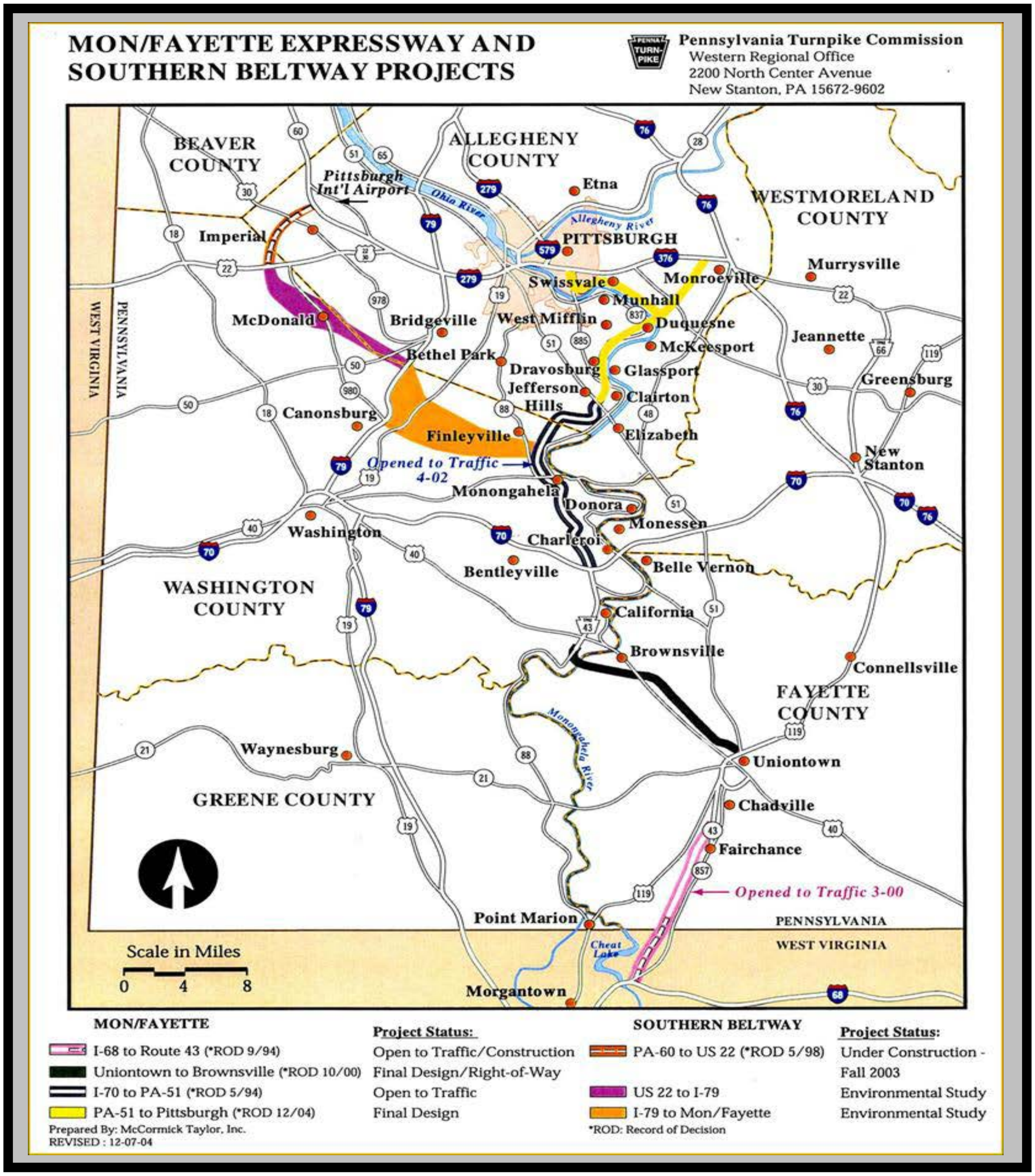


Figure 1 – Overview Map illustrating corridors for Mon/Fayette and Southern Beltway improvement projects

PROJECT SETTING, GEOTECHNICAL IMPACTS, AND CHALLENGES

Referring to Figure 2, the PA51 to I-376 Project continues from the northern terminus of the 70 to 51 Project and proceeds northward through the rural and suburban areas south of Pittsburgh (Sections 53A, B, and C) into the historically-industrialized Monongahela River and Turtle Creek valley areas. The alignment crosses the Monongahela River on a major river bridge structure (Section 53D) and splits at Section 53E into an eastern leg along the Thompson Run valley (Sections 53F and G), and a western leg along the north shore of the Monongahela River (Sections 53H, J, K, M, and N). The eastern and western termini are tie-ins to the Parkway East (I-376), a major, highly-congested route carrying traffic to and from the City to the eastern suburbs. The MFE, PA51 to I-376 Project will provide a high-speed bypass around the Parkway East to relieve traffic congestion, provide direct access to southern Pennsylvania, and provide access and economic stimulation to the economically-depressed Monongahela River valley area.

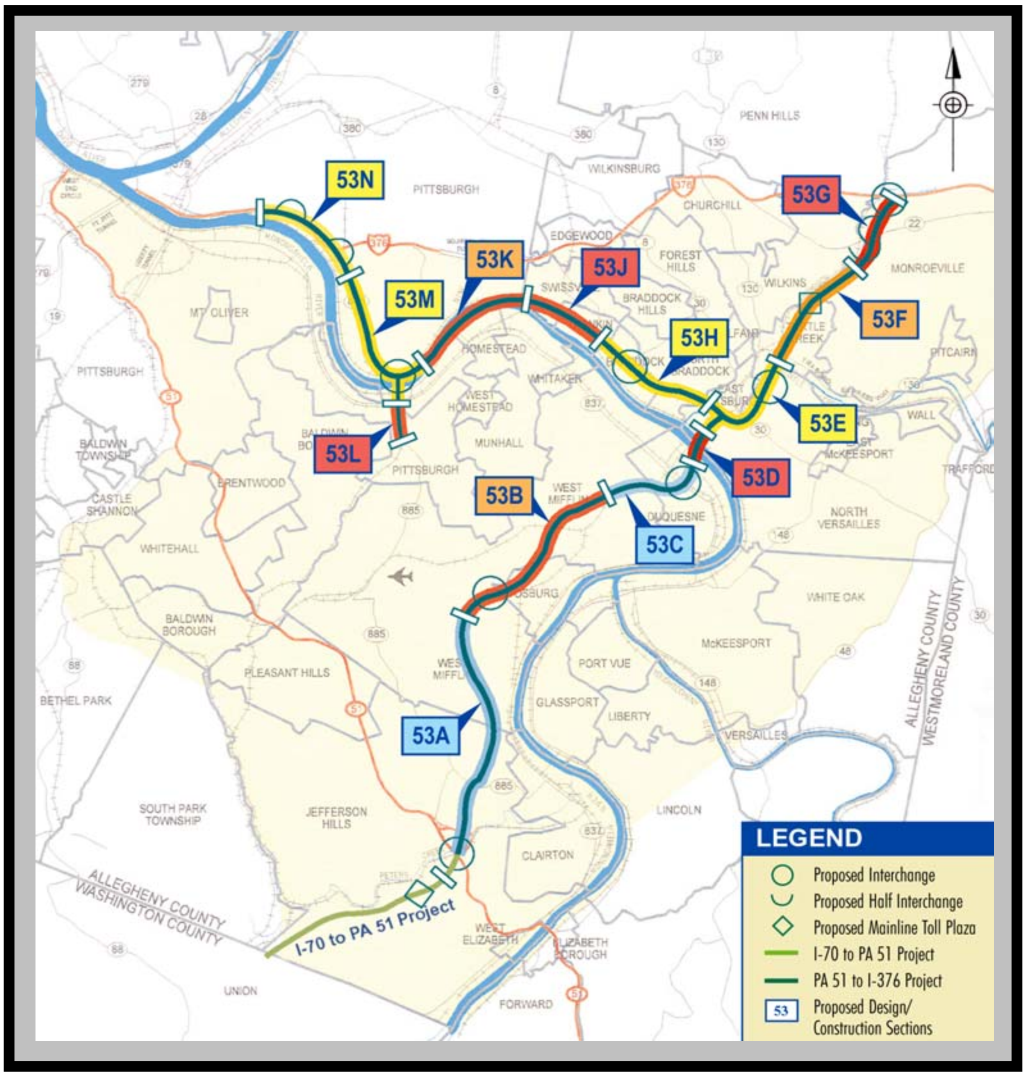


Figure 2 – Project Design Sections

Subsurface conditions are typical of conditions found in the Pittsburgh area. Bedrock consists of Pennsylvanian-Aged strata of the Glenshaw/Casselman Formations, Monongahela Group, and the Waynesburg Formation. Figure 3 is a ***Project Geological Map*** that illustrates the project alignment relative to the project rock formations. The strata comprising these formations are cyclothemic sandstones, siltstones, thinly-bedded to argillaceous limestones, carbonaceous/argillaceous shales, and coals. Figure 4 is a ***Project Geologic Columnar Section*** that illustrates the referenced rock formations and the particular stratigraphic interval of interest on the project. Given the cyclothemic nature of the rock formations and tectonic history of the area, the design of cut slopes in these formations will have to consider the stratigraphy, regional and localized jointing, and the dip of the strata to mitigate potential rockfall hazards. In addition, because each stratum is subject to varying rates of weathering, differential weathering of the formations in a cut slope face can also affect overall slope stability and create rockfall hazards. This is a common problem in southwestern Pennsylvania of many existing cut slopes, especially in the spring and fall when the freeze-thaw cycle is most active. All of these factors will require careful consideration during final design to minimize the potential for creating long-term problem slopes on a newly-constructed project.

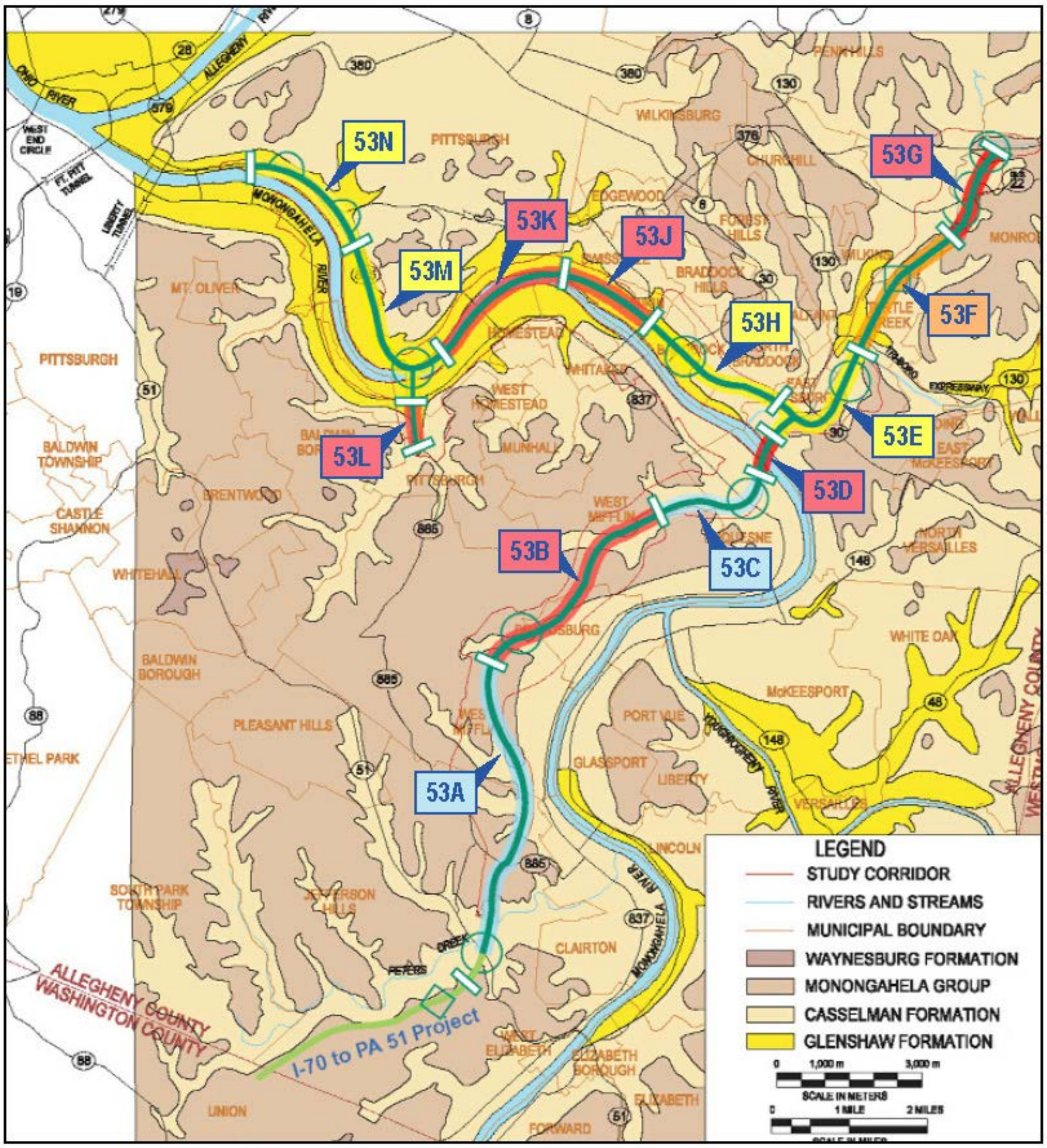


Figure 3 - Project Geological Map

Also characteristic of the rock formations in the Pittsburgh area are numerous soft, claystone/ indurated clay units known locally as "redbeds." These formations are typically present at different locations in the stratigraphy as shown in Figure 4. As the name suggests, these formations are typically characterized as red (sometimes green to gray), claystone units that are usually soft to medium hard, and contain many slickensided planes. Key impacts to geotechnical design are that they are susceptible to erosion, are relatively unstable for construction of cut slopes, are poor subgrade materials, and provide poor bearing for new foundations (including low bearing capacity for spread footings and bearing/relaxation issues for piles). When interbedded with more-durable rock strata in cut slopes, differential weathering (and incipient rockfall) becomes a concern. An erosional byproduct of the redbeds is development of low strength, colluvial soils that are typically found in thick deposits on the hillsides below the stratum. These materials are prone to landsliding and form poor foundation materials for new embankments. Numerous examples of ancient landsliding are found in these deposits in the project corridor. Re-activation of these ancient slides by new construction is an important consideration, one that the project geotechnical designers will have to evaluate and provide solutions for in their designs.

Prominent in the Pittsburgh area stratigraphy are several thick coal seams that were economically viable in the past. Of particular interest is the Pittsburgh coal seam separating the Casselman Formation and Monongahela Group rocks. This coal seam is approximately 10 feet thick and has been mined extensively in the Pittsburgh area. It is a source of frequent subsidence events impacting buildings and roads in southwestern Pennsylvania, particularly where overburden is thin. Referring to Figure 3, much of the area denoted as the Monongahela Group is undermined. Furthermore, the formational contact between the Casselman Formation and the Monongahela Group represents crop line of the Pittsburgh coal. Since deep mining of the coal ceased years ago, many of the abandoned mines are flooded. Depending on the local dip of the stratigraphy, the crop line is a frequent discharge point for Acid Mine Drainage (AMD) emanating from the mines. In addition, most strip mining of the coal occurred along the crop line. Strip mine spoil materials were frequently dumped along the hillsides adjacent to the outcrop (highwall), sometimes over existing unstable colluvial materials. At many locations, the highwalls were buried during reclamation efforts. Underground mine fires in Pittsburgh coal mine workings have been encountered in other sections of the MFE. While there have been none definitively located in the PA51 to I-376 Project, designers will have to be cognizant of the potential and provide solutions should one be encountered. Consequently, relative to the alignment shown in Figure 3 and the underlying geology, geotechnical treatment of the deep-mined/stripped Pittsburgh coal will be an important consideration of new roadway cuts, embankments, and structures, particularly in the southern design sections, 53A, B, and C, and eastern Section 53G.

Because of the urbanization and past industrial land uses in the project corridor, soil deposits are comprised of man-made fill materials from re-grading of existing materials to create useable land, and natural alluvial deposits in the floodplains of the Monongahela River, Turtle Creek, and Thompson Run valley. Generally termed Urban Land, these useable land areas include fill materials placed directly on the alluvial materials at many locations creating complex deposits prone to excessive differential settlements under new loads such as those imparted by new roadway embankments. Slope stability is also a concern when making new cuts in these materials. These settlements can cause severe impacts to new roadways (i.e., pavement distress) and structures (i.e., rotational/lateral distress on substructures, downdrag on piles, etc.). Because of past steelmaking activities, slag is a prominent constituent of the fill materials at many locations. Numerous large, discrete slag deposits are present throughout the corridor as well. Typical problems associated with slag include corrosivity to steel and concrete, and expansion potential. Another concern related to former steel mill and other industrial sites is the presence of massive buried foundations and basements that can impact excavations, create settlement concerns, and interfere with construction of deep structure foundations. On a smaller scale, past demolition of large tracts of residential properties for urban redevelopment and smaller commercial facilities creates a similar concern. These areas will require thorough evaluation during design to locate these features, assess the impacts, and develop workable designs to minimize impacts during construction.

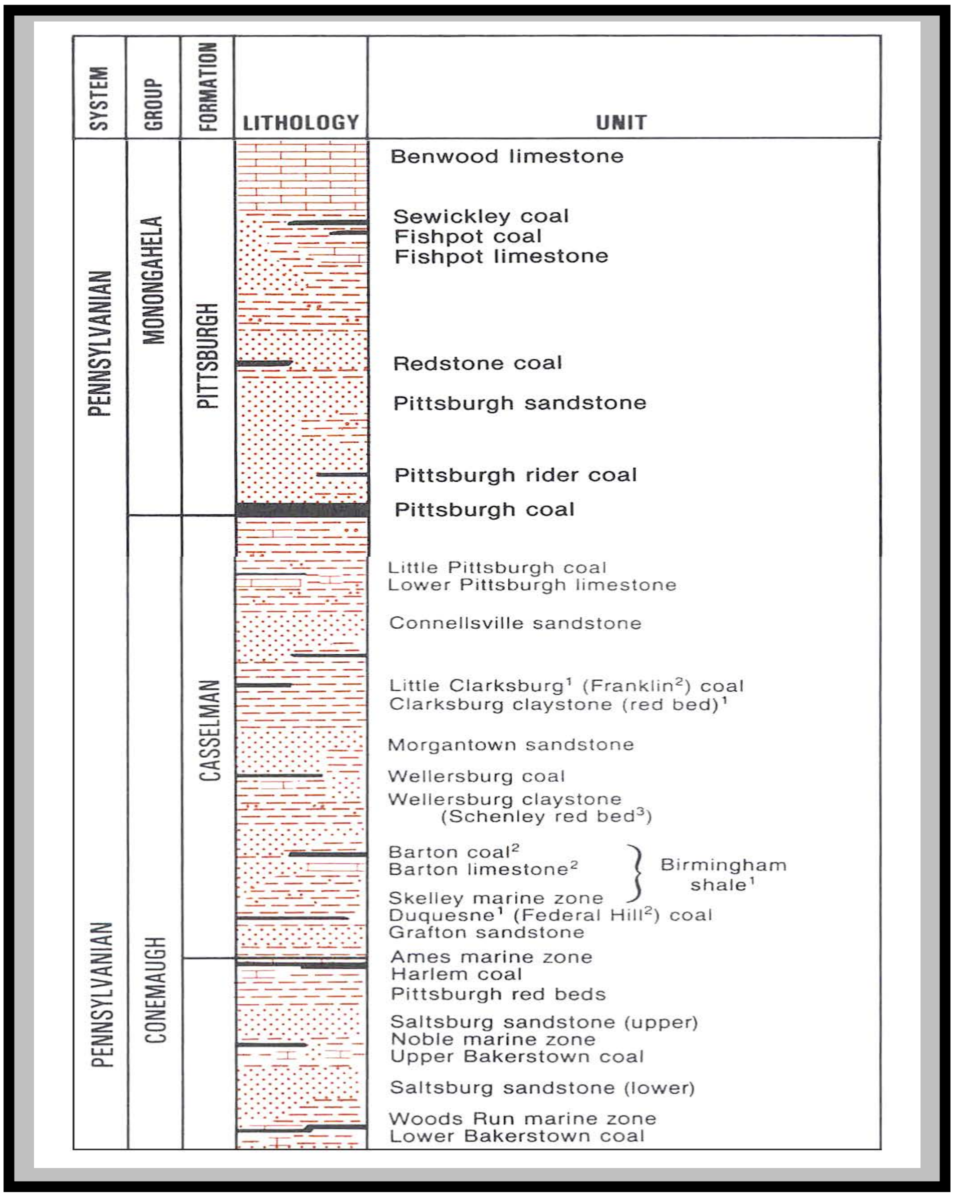


Figure 4 - Project Geologic Columnar Section (showing stratigraphic interval of interest on the project)

In addition to the geotechnical design concerns discussed above, the geotechnical consultants will be required to consider several ancillary issues during their investigations. During development of the Final Environmental Impact Statement (FEIS) leading up to issuance of the ROD, approximately 50 sites were determined to be potentially contaminated across all design sections. These sites range from former gas stations (with underground storage tanks) to large steel mill/ industrial sites. It was recommended that a Phase II Environmental Site Assessment (Phase II ESA) be completed for each site – this work is occurring at the time of this writing. The purpose of the Phase II ESA's is to determine contaminant types and concentrations, and provide guidance for health and safety protocols for the subsequent geotechnical subsurface investigations. Consequently, most geotechnical consultants will be required to perform drilling operations under Health and Safety Plans (HASP's), provide additional characterization of the sites, and develop waste management plans for construction to handle worker safety and provide for disposal of contaminated materials.

Significant railroad coordination will be required for completing the geotechnical investigations in all design sections. Five railroads have active lines, including major rail yards that will require access for completing field reconnaissance, drilling activities, and field view meetings on railroad right-of-way. Each railroad has different requirements for obtaining right-of-entry permits and protective insurance. The DM Team is establishing protocols for each railroad and will coordinate access for each of the geotechnical consultants.

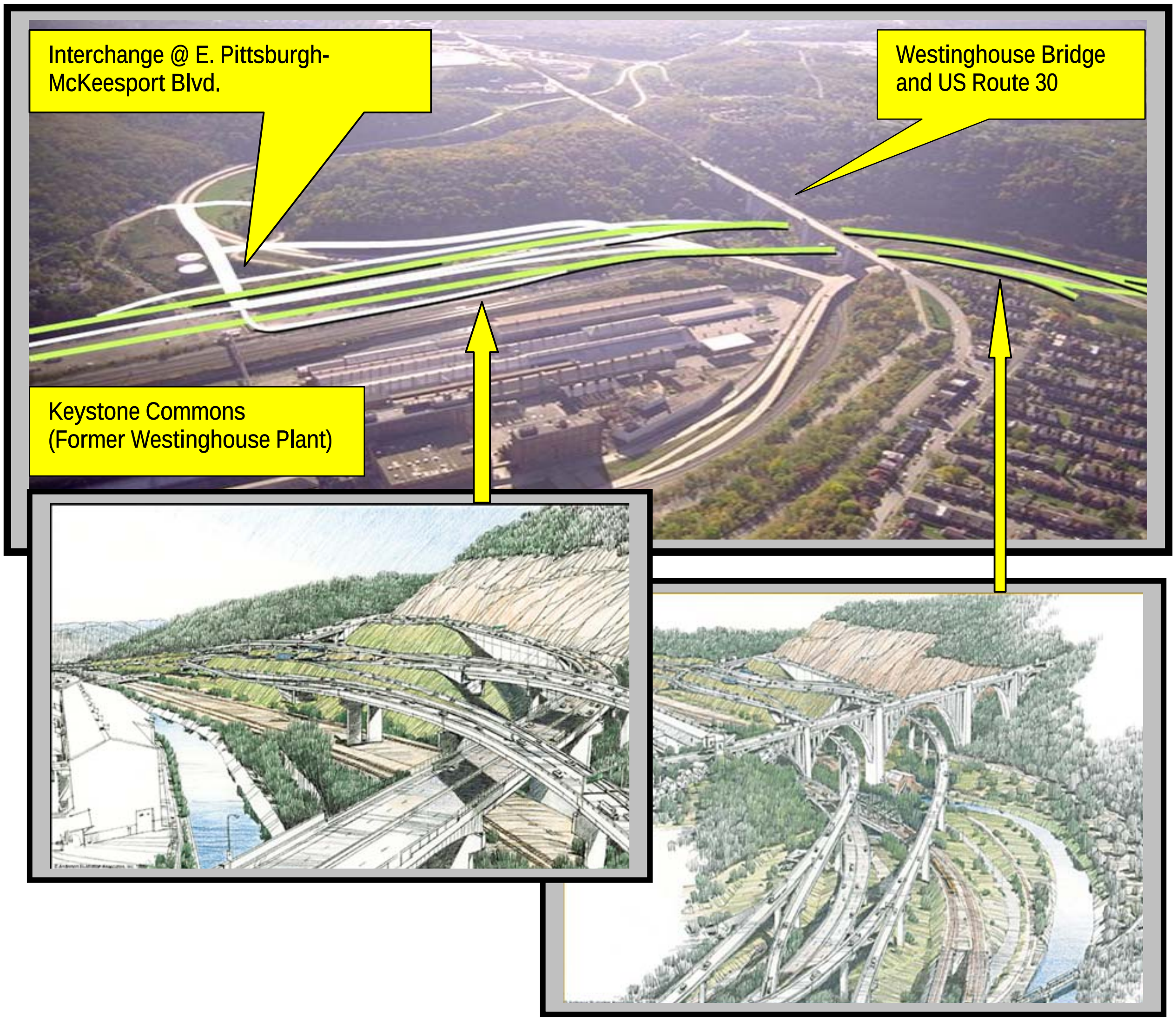
Finally, there are approximately 190 new or existing structures on the project that the consultant teams will be responsible for evaluating. This total includes structures identified at this time – it is likely this number will grow as final design progresses. These structures include new or existing bridges/viaducts (including pedestrian bridges), retaining walls, and culverts. For existing structures, the consultants will be required to inspect the structures and provide suitable rehabilitation designs, or recommendations and designs for replacement structures. Below are listed several examples of the various structure types to illustrate the sizes and design efforts that will be required:

- Section 53B – Culvert parallel to the MFE under a relocated local street (length = 1800')
- Section 53D – Dual bridges carrying the MFE over the Monongahela River, including approaches (lengths = 1500')
- Section 53E – Westbound viaduct carrying the MFE through the Borough of Turtle Creek (length = 2050')
- Section 53G – Retaining wall adjacent to the MFE mainline (length = 2600')

Figures 5, 6, and 7 are provided as "pictorial" illustrations of three typical design sections on the project (Sections 53E, K, and M). The geotechnical conditions described above will be encountered to some degree in each design section and are noted as applicable on each figure. The renderings provide a close approximation of the anticipated designs, and the oblique aerial photos show a birds-eye view of the physiography. From the aerial photos, one gets a good sense of the magnitude and complexity of conditions and the proposed designs.

CONCLUSIONS

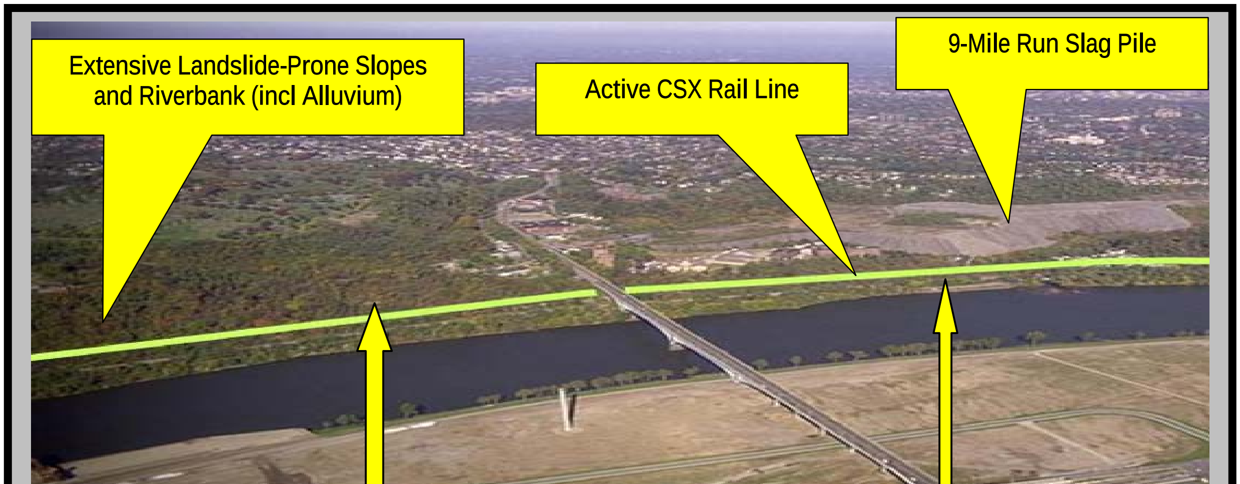
Considering the information presented of geotechnical conditions in the project corridor, the consultant teams will face numerous, unique challenges in evaluating and providing designs for the various improvements required in each design section. Close coordination will be required between design sections and the DM Team to provide consistent, cost-effective designs that adequately address the issues, while providing safe designs for the traveling public.

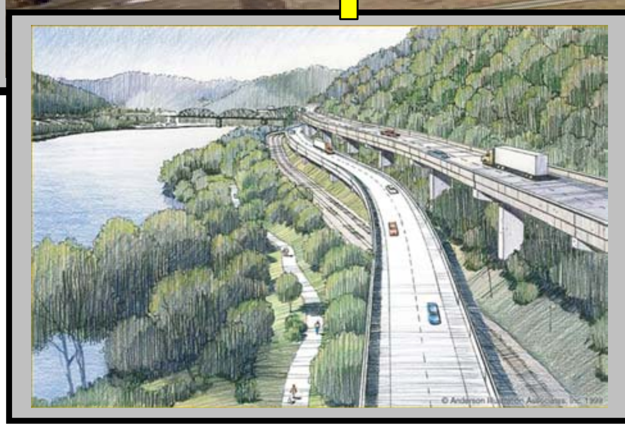


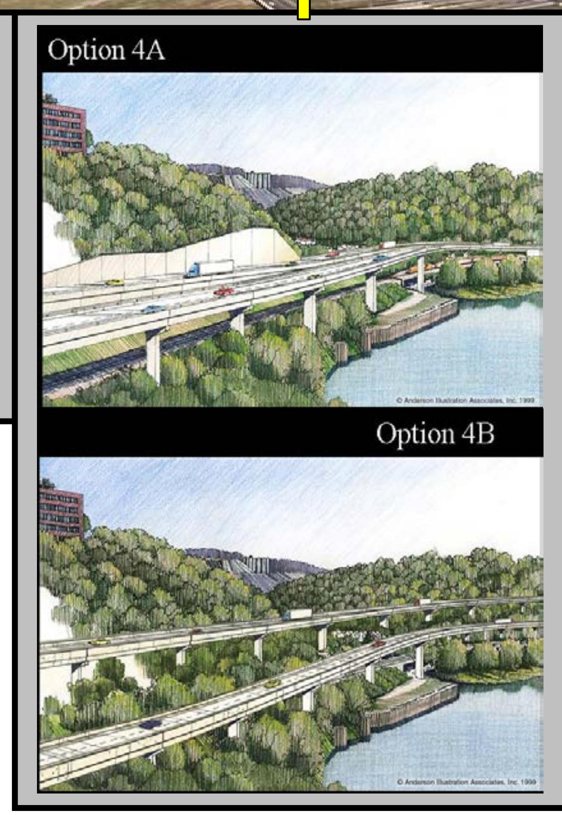
Key Geotechnical Design Issues:

- Complex Urban-Floodplain Soils
- Industrial Sites (Potentially Contaminated)
- Extensive Railroad Coordination
- Complex Structural Design
- Complex Interchange
- Large Roadway Cut
- Community Impact Issues

Figure 5 ***Section 53E – Monongahela River Bridge To Turtle Creek And Braddock Avenue***



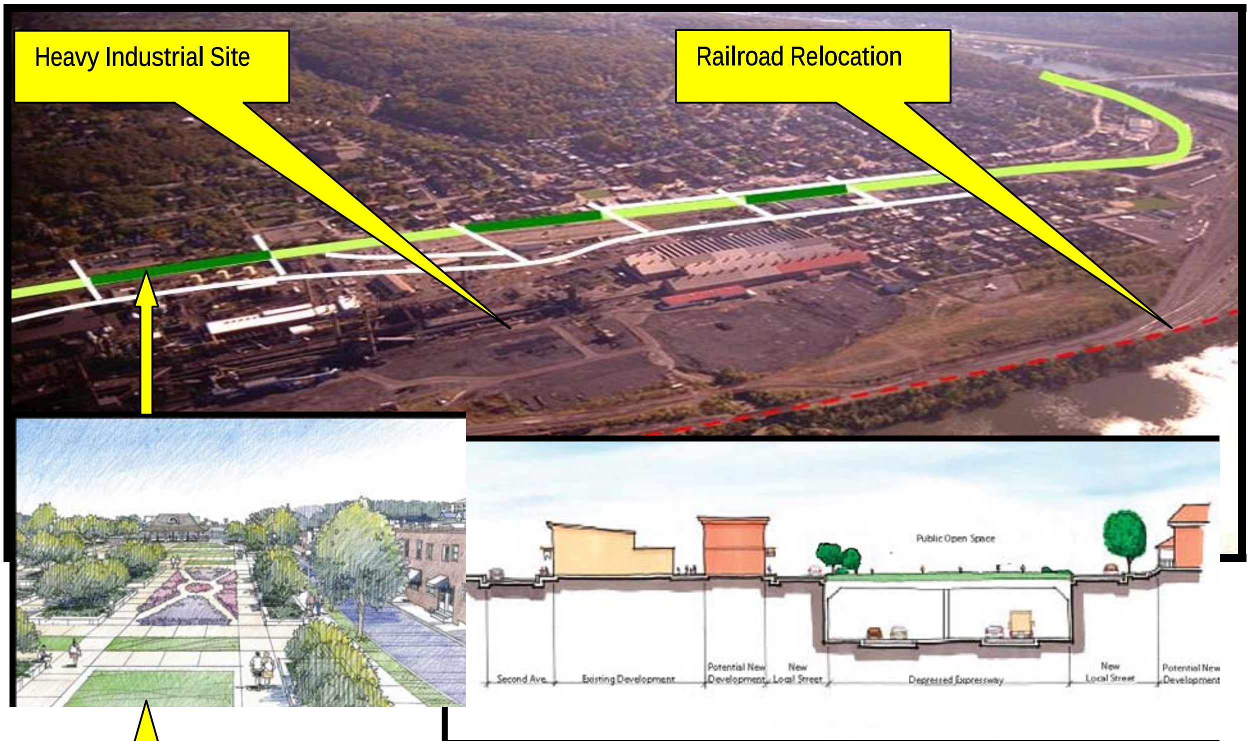




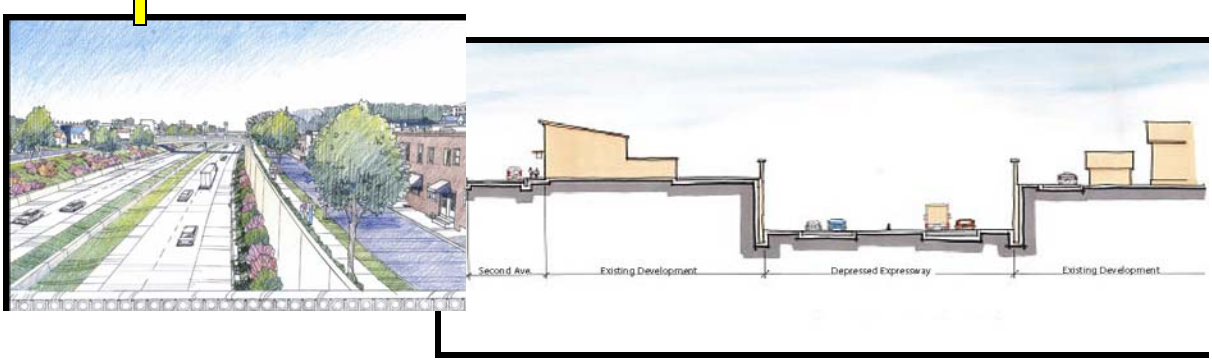
<u>Key Geotechnical Design Issues:</u>

- Extensive Landslide-Prone Colluvium
  (Recent and Ancient Landslides)
- Unstable Alluvium Along Riverbank
- Extensive Walls and/or Viaducts
  (Complex Foundations)
- Extensive Railroad Coordination Issues
- Community Impact Issues

Figure 6 ***Section 53K - Swissvale-Pittsburgh Line To Glenwood Bridge***



Lowered Cut with Lid Option – Typical Section



Lowered Open Cut Option – Typical Section

Key Geotechnical Design Issues:

- Complex Urban-Floodplain Soils
- Industrial Sites (Potentially Contaminated)
- Railroad Coordination/Relocation
- Complex Structural Design
- Complex Interchange
- Large Roadway Cut
- Community Impact Issues

Figure 7 ***Section 53M – Glenwood Bridge To Mobile Street***

ACKNOWLEDGEMENTS

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Performance of Flexible Debris Flow Barriers in Fire Burned Areas, State Route 18, San Bernardino County, CA

by

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Abstract

As presented at the 55th Annual Highway Geology Symposium in 2004, various flexible debris flow barriers were installed near San Bernardino, CA in the summer of 2004 in anticipation of debris flows originating from fire burned slopes. These debris flow barriers were installed in ten distinct debris flow channels upslope from and opening onto state route 18. Each of the barriers was dimensioned based on a unique dimensioning model using data provided by Caltrans including anticipated debris volumes and velocities, and a broad characterization of the expected debris flow compositions, channel geometry and barrier orientations. Each site required a unique barrier design with differing barrier heights, capacities and support infrastructure.

Construction of these barriers was completed in June, 2004. As anticipated, heavy rains in October and winter of 2004/2005 resulted in significant debris flows in all the identified channels. During the October events, all the barriers were impacted to various degrees. The barriers performed exactly as intended, and were subsequently cleaned of debris. As a result of these events, some aspects of designs were identified that could be adjusted to better facilitate cleanout maintenance and improve performance in general. In the winter of 2004/2005, the barriers were impacted again by debris during storm events. Some barriers were completely filled with debris and even somewhat overtopped, and some damage to the barriers was evident due to the greater than expected debris volumes. However, the drainage culverts immediately below the barriers remained clear and effective as intended, thus channeling water flow underneath rather than over the road. Ironically, this protected section of road was now used as a detour for other sections of road that were closed due to problems created by debris flow and rockfall; an opposite scenario from one year earlier.

The application of these barriers can be considered a complete success, performing exactly as intended. Some minor modifications to the barriers will be undertaken to prevent subsequent damage and to better facilitate maintenance.

INTRODUCTION

Following widespread wildfires in late 2003 near San Bernardino, CA and subsequent debris flows resulting from heavy rains, The California Department of Transportation (Caltrans) undertook efforts to prevent further debris flow damage to roads in the area. Much of the prior debris flow damage that occurred on a section of State Route 18 referred to as “The Narrows” was a result of debris clogging culverts, allowing water to then run over the roadway. After repairing the road damage, various measures were taken to prevent future clogging of culverts. One such measure was the installation of so-called VX/UX flexible debris flow barriers in ten debris flow channels intersecting the road along this section (Rorem, 2004).

Systems

The VX/UX barriers are an adaption of the ROCCO ring net rockfall barrier. Using back-calculations of other observed debris flow impacts to these rockfall barriers, and a unique barrier computer simulation program, a concept has been developed to predict the loading characteristics from debris flows. For this project, Caltrans personnel collected the required field data for each debris flow channel including such variables as expected debris volumes and velocities, flow composition and channel gradient, allowing a reasonable estimate of debris flow loading conditions for each site. Using this information, barriers were then designed to accommodate the expecting loading conditions at each site.

The barriers were installed in early summer of 2004. Design energies ranged from 200 kJ to 1,200 kJ, design heights ranged from 3.0 meters to 8.0 meters, widths ranged from 9.3 meters to 22.0 meters, and expected flow volumes ranged from 100 to 500 m^3. For barriers less than 12 meters wide, no posts were required (VX barriers). For barriers somewhat wider than 12 meters, 1 post was placed as additional span support (VX+ barriers). Some barriers required more than one post due to the large channel widths (UX barriers). All ROCCO ring nets were backed with a 2” mesh size chainlink, or comparable hexagonal wire mesh in order to prevent passage of material smaller than the openings of the 12 inch diameter rings in the ring nets.

Performance

Several events subsequent to installation of these barriers provided very quick validation of the concept and an opportunity to evaluate performance.

October 2004 Storms

In mid October, four successive days of rain produced 8.5” of precipitation in the area, with rain measuring 3.5” in one of those days. This resulted in widespread debris flows in the area, including at all 10 of the VX/UX barrier sites; the first true test of the concept on a reasonable scale. In all, the 10 barriers successfully contained approximately 1,000 m^3 of debris. In the interest of brevity, results from the 3 most notable sites will be summarized.

<u>Site 1: UX-150 barrier, 1500 kJ design load</u>

Design debris volume - 360 m^3
Event debris volume - 161 m^3
Event debris depth at barrier – 2.3 meters
Event debris composition – soil 35%, rocks/boulders 40%, vegetation 25%

No debris passed the barrier, and there was no barrier damage. Some braking element engagement was observed however, including complete engagement on one of the post tieback ropes.

Fig. 1 – Site 1 immediately after install. (Courtesy Duffy).

Fig. 2 – Site 1 October debris. (Courtesy Duffy)
Note: barrier is 5 meters tall.

<u>Site 2: VX+ barrier, 400 kJ design load</u>
Design debris volume - 315 m^3
Event debris volume - 263 m^3
Event debris depth at barrier – 3.2 meters
Event debris composition – soil 35%, rocks/boulders 40%, vegetation 25%, but also with very large boulders, and 4 trees trunks of 3-4 feet diameter x 30 feet long.

A small amount of debris did pass the barrier due to breakage of the bottom support ropes. It appeared that a direct impact to this rope by one of the large tree trunks was the probable cause of the breakage. Some braking element engagement.

Fig. 3 – Site 2 after installation.

Fig. 4 – Site 2 October debris. (Courtesy Duffy).

Fig. 5 – Site 2 logs removed from contained debris.
(Courtesy Duffy).

Site 7: VX barrier, 200 kJ design load
Design debris volume - 100 m^3
Event debris volume - 33 m^3
Event debris depth at barrier – 2.75 meters
Event debris composition – soil 70%, rocks 30%, vegetation 0%

Fig. 6 – Site 7 after install.

Fig. 7 – Site 7 October debris. (Courtesy Duffy).

No barrier damage or significant braking element engagement was observed at any of the other 7 sites. For all practical purposes, with the exception of site 2, all debris was contained other than insignificant amounts of sand passing. None of the barriers were overtopped, and no debris passed around the sides. In summary, overall performance was a great success, with remaining capacity in most of the barriers, even before cleanout. Cleanout was undertaken nevertheless to maximize capacity for future events.

Winter 2004/2005 Storms

As has been widely reported, the amount of rainfall produced by the winter storms of 2004/2005 has been unprecedented. The barriers installed at The Narrows have thus been tested to the extreme by multiple events during the winter. Several of the barriers were completely filled with debris, and some were even over-topped. Such volumes and frequency could not have been reasonably anticipated. Normally, the susceptibility of fire burned slopes to debris flow diminishes after a few years. These barriers were primarily intended to provide protection during such a time period, assuming normal or somewhat above normal precipitation. The fact that there were unprecedented rainfall events during such a time period is astonishing.

Observations

Though much of the needed cleanout and repairs were completed after the October events, not all required maintenance was completed prior to these back-to-back and continuous winter events. Further barrier damage was observed from these winter events, as it was not anticipated that the barriers would be subjected to such relentless and unusually extreme events. Consequently, barriers at some of the locations have been removed, while others have been repaired. Regardless, the barriers did contain large volumes of debris and did perform as intended for the expected events, and certainly prevented extensive roadway damage, road closures, and significant costs associated with roadway maintenance.

Much was learned from these many events, and this information will be extremely useful going into the future. Facilitation of barrier cleanout after significant events will need to be improved. For barriers that are not of great height, cleanout can be largely done with backhoes, as was done at The Narrows. The seam rope used for connecting net panels together and to the side ropes (instead of the normal shackling) also proved to be helpful. However, modifications to the seaming will be made on future installations to improve barrier performance and to better facilitate cleanout, including stronger seam ropes and seaming orientation. Additionally, modifications will need to be made in the shackling connections that are still used for connecting the nets to the top support rope, because nets that are extremely loaded put the top rope in significant tension making removal of the shackles difficult.

Evaluation of these sites and the associated barriers is ongoing, including revisiting the upslope drainages and calculation of event energies as a back check on the earlier design parameter assumptions and determinations that were made. This should help with data collection methods for future sites, as well as to determine any mitigation modifications or upgrades needed for these particular sites.

OTHER RECENT INSTALLATIONS – GAVIOTA PASS

Following the anticipated success of the barriers installed at The Narrows, additional debris flow barriers were installed at another Southern California fire burned area known as Gaviota Pass, along U.S. highway 101. Numerous rockfall barriers had already been installed in this area over the last 15 years, and many have stopped small debris flows in the past. Following significant fires in June 2004, debris flows were also anticipated to follow in this area during the next rainy season. Consequently, one VX+ debris flow barrier was installed here in September 2004, and another was installed in November 2004 in the same manner and for the same purpose as at The Narrows. Additionally, a new type of barrier was installed; the so-called Whale Net Barrier.

Systems

VX+ Barriers

The VX+ barrier is the same as the VX, with one middle post to support the long span (> 12 m.), and with top, middle and bottom support ropes, anchored at the sides.

<u>North Site</u>:

- Expected debris volume = 100 m^3.
- Calculated necessary height = 1.5 m.

- Length across top = 7.0 m. (post provided additional elevation to one side).

Fig. 8 – North site before install.

Fig. 9 – North site after install.

Fig. 10 – North site storm debris contained. (Courtesy Duffy).

South Site:

- Expected debris volume = 300 m^3.
- Calculated necessary height = 1.6 m.
- Length across top = 13.1 m.

Fig. 11 – South site before installation.
(Courtesy Duffy).

Fig. 12 – South site after installation.
(Courtesy Duffy).

Fig. 13 – South site debris contained.

Performance

Following heavy rains in late December, these barriers were completely filled with debris, and were even somewhat overtopped once full. No damage was observed and thus no repair necessary. There was some slight braking element engagement, but no replacement necessary. The barriers performed precisely as intended, allowing the adjacent culverts to remain open and clear. The debris was immediately cleared away in preparation for the next rain events. Cleaning of debris took approximately 2 hours per barrier. This first event was merely the first of many. The barriers have been cleaned and re-filled with debris several times during the winter storms of 2005, each time performing exactly as intended and with no damage.

The observed overtopping ironically resulted from too much debris being retained, and specifically, smaller pieces of vegetation such as grasses. The chainlink backing on the ring nets trapped much of these grasses and small branches, creating a sort of damming effect. In retrospect, it is the opinion of some that allowing fine material and smaller rocks to pass through the ring nets would be acceptable, such as would occur if there were no chainlink backing. It is expected that these smaller particles should flow through the culvert without clogging. The ring nets then would simply contain the larger rocks and vegetation, which are the primary culprits for culvert clogging.

Whale Net Barrier

In December 2004, a huge debris flow occurred in a third channel that had no protection, causing a motorhome to be pushed off the road, and a road closure due to debris on the road. It is interesting to note that in all other locations where a barrier had been installed previously, there were no debris problems on the road. In this particular channel, no VX or UX barrier had been placed due to the expectation that the volumes would be too great for such a barrier to be effective. Indeed, the observed event involved approximately 15,000 cubic yards of material, with many hundreds of boulders with diameters of 3 to 5 feet, and many larger. Though there was a large culvert under the road at this channel with a large trash rack at the opening, the trash rack was quickly clogged by the larger debris, and the culvert was overcome resulting in debris on the road. A survey of the upstream debris source area indicated that there was still a substantial source of debris remaining, even after this large event.

Fig. 14 – Whale net site, channel filled with debris. (Courtesy Duffy).

Fig. 15 – Hwy. 101 covered with debris prior to whale net install. (Courtesy Duffy).

Fig. 16 – Boulder debris accumulation prior to whale net install. (Courtesy Duffy).

Fig. 17 – Cleanout of channel before whale net install. (Courtesy Duffy).

Subsequent to this event, Caltrans personnel contacted Geobrugg to inquire about the possibility of installing a so-called Whale Net barrier, similar to that which has been used in Japan. This barrier is a fortified version of the VX/UX barrier. In addition to fortifying the support infrastructure and anchorages of the UX barrier, the primary difference is the ring size of the nets. Each ring in these nets has a diameter of 1.5 meters, made from 33 windings of 4 mm diameter wire. The resulting wire bundle has a diameter of approximately 30 mm. The basic concept for such a barrier is that all but the very largest material in the flow will be allowed to pass through the barrier, preventing the trash rack in front of the culvert from clogging with large debris.

Fig. 18 – Tabata Barrier using whale nets in Japan. (Courtesy Roth).

Design of this barrier was done in a similar manner as for the VX/UX barriers, using back calculations from observed whale net barrier events in Japan, as well as calculations of anticipated volumes, velocities, channel geometry, and so forth. However, this particular design is different from Japan barrier in that no concrete channel lining or substantial concrete anchoring abutments. Such concrete structures were not allowed for environmental reasons, not to mention cost. With the exception of the ring nets themselves, the barrier is actually closer to the VX/UX concept than the Japanese version.

The bottom of the nets do not reach all the way to the stream bed, except at the sides. The resulting gap between bottom of nets and the ground is intended to allow normal stream flow and animal passage.

- Anticipated debris volume = approximately 2,000 to 3,000 m^3.
- Channel gradient = Approx. 10 degrees.
- Calculated necessary height = 5.0 m.
- Length across top = 27.0 m.

Fabrication of the nets was completed in Geobrugg's Santa Fe, NM factory based on manufacturing instructions from the Geobrugg Japan affiliate. The nets were fabricated in two large pieces that were shackled together in the field. Most installation work involved construction of the post foundations and lateral and upslope anchorage. Actual erection of the barrier was completed in a matter of days.

Fig. 19 - Whale net barrier looking up channel.

Fig. 20 - Whale net barrier side profile. Bottom of nets anchored downslope of posts.

Fig. 21 - Whale net barrier in construction.

Fig. 22 - Typical boulder to be stopped by whale
net barrier.

Installation of this barrier was completed in early March 2005. We now await the first impact.

CONCLUSIONS AND OUTLOOK

Invaluable information has been gained and lessons learned from this most extensive application of debris flow barriers to date anywhere in the world. Some key points:

- Anticipating volumes and the nature of expected debris flows can be very difficult. Time and effort spent to determine critical dimensioning parameters is time well spent, and is very important.
- Reasonable probabilities must nevertheless be employed, to maximize benefit vs. costs, meaning that unusually extreme events may overwhelm the designed barriers. There are limitations, and repair may be necessary.
- Chainlink or finer mesh may or may not be beneficial. In cases of the barriers protecting culverts from clogging with debris, we want fines and water to pass. Some situations may justify retaining more of the fine material.
- There is a need for more maintenance friendliness with the barriers, and work is being done in this area.

The future for these barrier concepts looks promising. Indeed, the science and engineering involved with debris flow mitigation is currently evolving at a rapid pace, with significant developments arising continually. There is however much work and research that needs to be done. Regardless, these barriers represent the state of the art for such applications and are superior to many other available options.

As actual field events continue to unfold, further valuable information will be gained. One or more of the barriers at The Narrows or Gaviota Pass may still be instrumented, which may include video monitoring. This data will provide useful information to further develop the concept. Eventual impacts to the new whale net barrier are also anticipated, and will provide useful observations regarding performance and design assumptions.

The next big R&D effort in progress is a test site that is currently being developed, wherein a natural channel produces regularly and naturally occurring debris flows on a semi-predictable basis. The site is conducive to installing monitoring instrumentation for measuring flow density and velocity, flow volume, and various configurations of barriers. This project will provide an opportunity to collect a wealth of data for further development of this concept.

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The Importance of Lateral Stress in Geotechnical Design ...but How Do We Measure It?

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INTRODUCTION

Lateral stress is an important parameter in the design of below grade or retaining structures for transportation projects. However, this parameter is difficult to measure and is usually estimated for use in design from empirical relationships. The need for accurate measurement of lateral stress has become even more critical with the proliferation of computer modeling. The primary difficulty in obtaining an accurate in situ measurement of lateral stress in soil is their sensitivity to disturbance. When a sampler is inserted into the bottom of the borehole, it displaces soil and increases the lateral stress. If a hole is bored to allow a measurement device to be inserted, lateral stresses in the vicinity of the borehole are relieved. Thus, a device that is capable of being inserted into the soil with minimal disturbance and measures lateral stresses becomes a useful design tool. This paper addresses the determination of in situ lateral stresses using the K_o stepped blade test (SBT) device and summarizes its application as it relates to design of earth retaining or below grade structures for transportation projects.

BACKGROUND

It is impossible to insert a measuring device into the soil without disturbing the soil. As a device is inserted into a soil, the soil will respond in an elastic, consolidation, or plastic behavior depending on the amount of lateral displacement[1]. Plastic behavior occurs when stresses acting on the soil have exceeded a limit pressure and the deformation of the soil can continue without existing stress. This condition occurs when the soil is severely disturbed and measurement of this lateral stress level may not be representative of the in situ stress levels. Studies have shown that a peak lateral stress has resulted from as little as 0.06 percent displacement.[2]

The SBT device was developed to introduce systematic increasing levels of disturbance and establish a relationship between stress and disturbance. The SBT device is a 26 in (660 mm) long by 3 in. (76 mm) wide, high strength stainless steel blade with four successively thicker (3, 4.5, 6, 7.5 mm) steps. A 1-inch (25mm) pneumatic pressure sensor is centered within each of the four blade steps. The pressure sensors are connected through the use of a manifold and tubing to a readout box, see Figure 1.

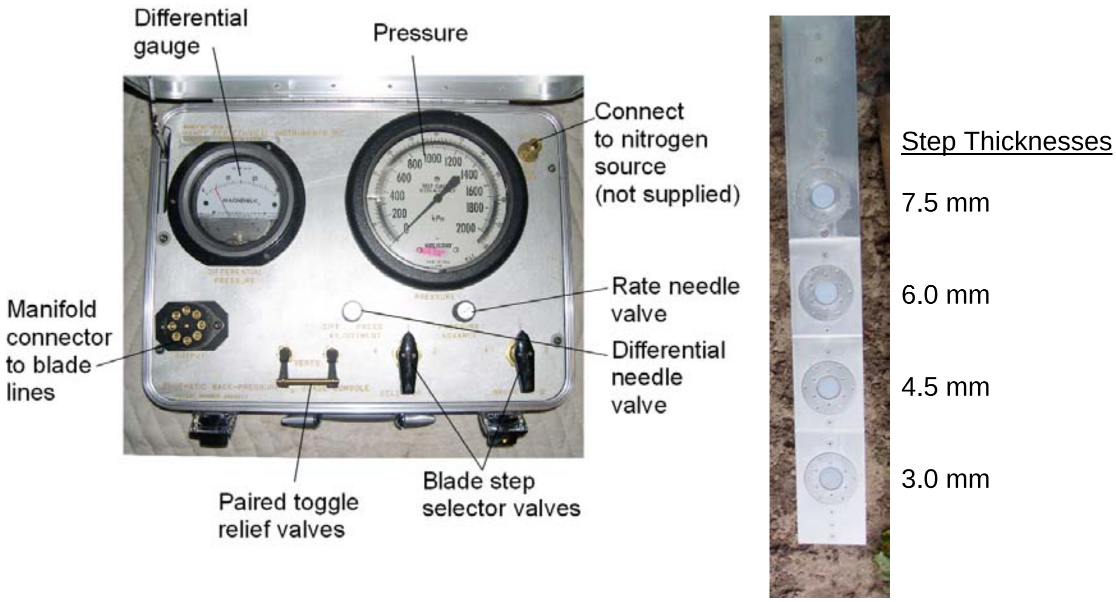

Figure 1. SBT Console and Blade Showing Pneumatic Pressure Cells.

To perform a test, the SBT device is connected to steel drill rods, lowered to the bottom of the borehole or casing, and advanced the incremental distances past the bottom. The pressures at the sensor on each embedded blade step are determined, plotted versus blade thickness and extrapolated to a theoretical "zero disturbance state", see Figure 2.

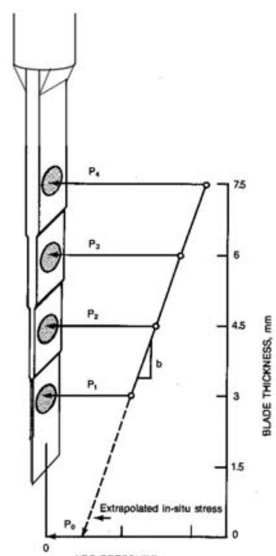

Figure 2. Schematic Showing the Extrapolation Principle. Actual Data Sets are from the same Subdepth. The Extra Length at the Top Allows One More 4-Point Data Set. (from Handy et al., 1982)

Correlation between the blade thickness and measured stress describes an exponential relationship, see Equation 1, which can be explained by an increase in compression modulus with pressure, similar to the linear relationship between void ratio and logarithm of pressure in one-dimensional drained consolidation test.

$$P_o = P_l \, a \, e^{-bt}$$

Where:
P_o = initial lateral stress
P_l = lateral stress after insertion
t = thickness
a, b = regression coefficients

Equation 1. Exponential Relationship Between Blade Thickness and Measured Lateral Stress

The soil response behavior often changes with blade thickness; thus, the data must be examined for validity. Data from different behavior modes should not be mixed. Typically, results from the consolidation behavior are used in the interpretation. However, two common interpretation exceptions have been observed in this behavior mode: 1) the thinnest step may not introduce a breakdown of soil structure, resulting in higher than expected pressure and elastic behavior and 2) in soft soils the thickest step may introduce yielding of the soil and lower than expected pressure. Where such data is detected, the contributing pressures are not included in the data interpretation. Generally, the lateral stress data is plotted versus depth, showing general trends. Often lateral stress is highly variable[3], so data versus depth may appear quite scattered.

Previous Studies

The SBT was invented and built at Iowa State University during the 1980's by Dr. R. L. Handy in research sponsored by the United States Federal Highway Administration and the United States Department of Defense. The SBT has been used on a number of project in the United States and Canada to decipher the soil stress history of a soil including:

- √ Identification of expansive soils (high lateral stress)
- √ Identification of collapsible soils (low lateral stress)
- √ Documentation of soil compaction uniformity
- √ Diagnosis of skin friction and bearing capacity of foundation systems
- √ Identification of soil behavior zones around rammed aggregate piers

Table 1 summarizes results from several documented case histories where SBT's were conducted in wide range of soil conditions. Comments with regard to data trends and comparisons to other measurements and soil properties are provided.

Table 1. Literature summary of K_o Stepped Blade testing

Site Location	Soil Description	Depth (m)	K_o	b (mm^{-1})	c (kPa)	ϕ (deg)	Comments
Gloucester, Ontario (Lutenegger *et al.* 1986)	Sensitive marine clay, lightly OC underlain by glacial till	0.0–15.5	0.56–2.25	0.10–0.40	—*	—	Most K_o values less than 1.5. No clear trend reported.
Raquette River Cemetery Massena, New York (Lutenegger *et al.* 1986)	Marine clays with crust of OC brown fissure clay up to 3 m overlying lightly OC gray clay	0.0–3.0; 3.0–12.0	0.46–6.40; 0.46–3.00	0.03–0.45	5–6 (s_u)	—	K_o generally less than 2 in gray clay below crust.
Mt. Union Cemetery Mt. Union, Iowa (Lutenegger *et al.* 1986)	Plastic silty clay (loess) (2.4 m) overlying highly plastic clay paleosol (gumbotil) composed of montmorillonitic clay	0.0–6.4	1.1–8.6	0.01–0.17	—	—	Decreasing K_o with depth through loess. Increasing K_o with depth through clay paleosol.
STORTZ Omaha, Nebraska (Lutenegger *et al.* 1986)	Soft alluvial silty clay with OC crust with thickness of 4.5 m overlying NC material	0.0–7.6	0.30–5.01	0.02–0.28	15– 40 (s_u)	—	Most K_o values less than 2.0
VH Omaha, Nebraska	Silty loess, moist and somewhat plastic	0.0–7.0	—	—	—	—	Blade penetration caused substantial crushing of the soil structure.
OPPD Omaha, Nebraska (Lutenegger *et al.* 1986)	Silty loess, very dry and brittle	0.0–6.0	—	—	—	—	Only 22 of 130 data were acceptable (K_o values not presented).
Boone, Iowa (Handy *et al.* 1982)	OC loess (2.4 m), overlain by glacial till (11.5 m)	11.5	1.3	0.31	—	—	Tests performed in horizontal and vertical directions. Measured vertical stress within 5% of that calculated.
Turin, Iowa (Handy *et al.* 1982)	UC, low-clay-content loess	11.5	0.9	0.13	—	—	Tests performed in horizontal and vertical directions. Measured vertical stress only one-sixth of that calculated.

Table 1. (continued)

Site Location	Soil Description	Depth (m)	K_o	b (mm^{-1})	c (kPa)	ϕ (deg)	Comments
Mitchellville, Iowa (Handy *et al.* 1982)	Glacial till end moraine Underlying loess	4.9 9.7	1.3 0.5	0.12 0.31	— —	— —	Phenomenon of overconsolidation of till by glacial ice, while underlying loess not OC not previously observed
Spangler Geotechnical Laboratory Ames, Iowa (Mings 1987)	Silty clay loam, oxidized	0.0–4.0	2.0–11.0	0.0 – 0.12	0–48	11–31	K_o decreases with depth from 1.5 m
David City, Iowa (Retz 1987)	Stiff, dark silty clay fill with some sand (1 m) underlain by brown and gray, low plastic silty clay (8 m)	1.6–8.2	0.5–4.8	0.01–0.45	4–13	16–22	Most K_o values of 1.3 to 2.7. Overconsolidation attributed to desiccation.
Garrison, Iowa (Retz 1987)	Black, stiff, silty clay topsoil Light brown, low plastic, stiff, silty clay	0.0–1.0 1.0 – 8.8	3.8–12.7 0.5 – 7.0	0.04–0.48	4– 13	9– 34	Most K_o values 1.0 to 5.0. Prevalent "First Point Anomaly"
Spangler Geotechnical Experimentation Site Ames, Iowa (Thompson 2004)	Recompacted weathered shale, USCS classification CL, lean clay	0.0–0.6	7– 166	0.02–0.13	24	21	Used to indicate soil stress development around piles subject to lateral soil movement. Measured lateral stress approximately 50% higher than Rankine passive earth pressure.
Texas A&M University Research and Extension Center, College Station, Texas (Gan and Briaud 1987)	Uniform, very stiff clay with medium plasticity	1.5–6.1	2.3–8.2	0.02–0.34	130 (s_u)	—	Approximately 4 times higher than PBPMT tests.
University of Houston Pile Research Site Houston, Texas (Gan and Briaud 1987)	Stiff, OC clay with stratums of CL-CH, CL, CH, and ML	1.5–13.7	0.93–6.22	0.02–0.32	—	—	Approximately 4 times higher than PBPMT tests

Table 1. (continued)

Site Location	Soil Description	Depth (m)	K_o	b (mm^{-1})	c (kPa)	ϕ (deg)	Comments
Hunter's Point San Francisco, California (Gan and Briaud 1987)	Fill (1.8 m) overlying loose to medium dense sand (13 m thick)	2.1–6.7	0.60–1.66	0.02–0.14	—	—	Approximately 1.2 times lower than PBPMT tests
Cheshire Bridge 2 Site Connecticut (Gan and Briaud 1987)	Glacial outwash sand that slopes to the south.	1.5–8.7	0.77–2.53	0.05–0.45	—	—	Approximately 1.2 times lower than PBPMT tests.
Des Moines, Iowa (White *et al.* 2002)	Compressible clay and silt overlying highly weathered shale.	0.0–13.0	0.4–2.2	—	—	—	Test performed 70 cm radial distance from stone column.
Des Moines, Iowa (White *et al.* 2002)	Compressible clay overlying alluvial sand and highly weathered shale.	0.0–6.0	0.4–4.0	—	9–38	11–32	Test performed 85 cm radial distance from Geopier Rammed Aggregate Pier.
Weston, Missouri (Handy 1995)	Wisconsin-age loess (15.2 m) overlying glacial till (1.5 m) that overlies shale.	0.0–10.0	0.25–1.40	—	—	—	K_o values less than 0.5 for depths greater than 3.0 m. High surficial lateral stress attributed to moisture cycling and expansive clay minerals.
Omaha, Nebraska (Handy 1995)	Peorian loess (7.9 m) over leached early Wisconsin-age loess (1.8 m) that overlies clay-rich Sangamon paleosol.	1.5–11.0	0.10–1.05	—	—	—	Underconsolidated and collapsible from 2.0 – 4.5 m.

Notes:* Indicates data not reported
OC = overconsolidated
NC = normally consolidated
UC = underconsolidated
PBPMT = pre-bored pressuremeter tests

STEPPED BLADE TESTS (SBT'S) APPLICATIONS

Knowledge of the coefficient of at-rest (K_o) earth pressure for clay and sand deposits is often important for the design of foundation, earth retaining structures and excavations. Generally, the coefficient of at-rest earth pressure will increase as the materials become more over consolidated and decrease as they become underconsolidated (collapsible). Thus, the measurement of lateral earth pressure gives an indication into the in situ stress condition resulting from erosion or other geological processes.

Collapsible Loess

During deposition, the increase in vertical stress should result in a corresponding increase in horizontal stress, i.e., a constant K_o. Laboratory data indicates that the friction angle of loess material typically is between 25 to 31 degrees, which would according to the Jaky relationship ($K_o = 1 - \sin\phi$) correspond to a K_o value of 0.56 to 0.50. However, based on SBT's performed in loess in Omaha, Nebraska and Weston, Missouri, relatively low K_o values, 0.26 to 0.37[4], were measured and are indicative of underconsolidation and probable collapsibility, see Figure 4.

Vertical overburden stress and SBT measured horizontal stresses can be used to estimate Mohr circles or stress paths. As shown in Figure 5, where the stress path determined from the measured data, plots below the K_o line the material is overconsolidated (desiccated crust). On the other hand, where the stress path is above the K_o confirms that this material is underconsolidated and collapsible.

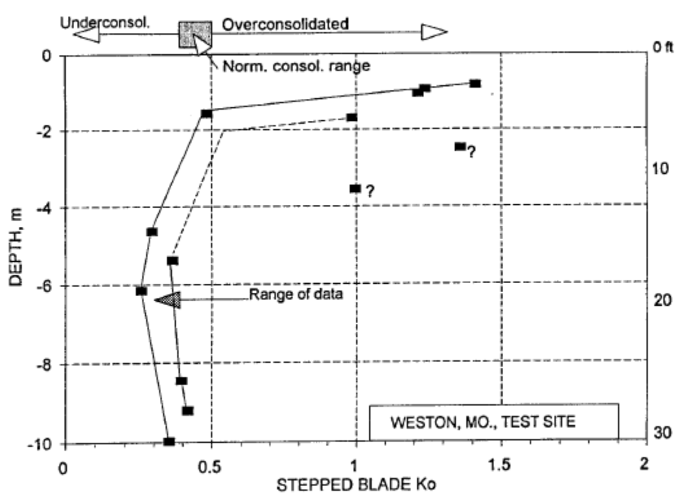

Figure 4. Lateral Stress Ratios Calculated from SBT data at Weston, Missouri.[4]

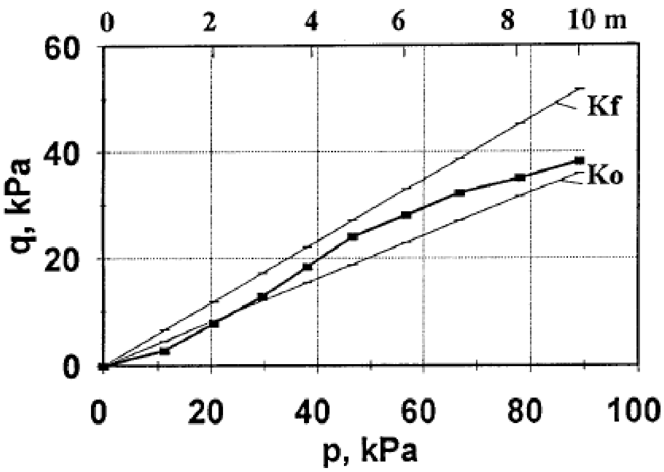

Figure 5. Stress Paths Representing In situ Stresses at Weston, Missouri.[4]

Foundation Loading

A good example of foundation loading is the construction of earthen embankment. As the embankment is constructed, an increase in vertical stress causes the soil to consolidate and proportionally increases the lateral stress. During consolidation, strain in the soil is more nearly proportional to the logarithm of stress (rather than linearly proportional) as the soil densifies and the modulus increases. This increase is permanent as long as the soil structure remains intact[5]. If the vertical stress is continually increased, the K value approaches the passive condition and failure.

This condition was monitored for the construction of an embankment over soft clay deposited within an oxbow lake in Omaha, Nebraska. The SBT data was collected prior to and later after excessive settlements and slope indicators showed lateral bulging outward of the foundation soils[6]. The SBT data indicated that the measured K values exceeded the theoretical K_p value, see Figure 6, computed from shear test data. Thus, the SBT device can be used to estimate bearing capacity failure.

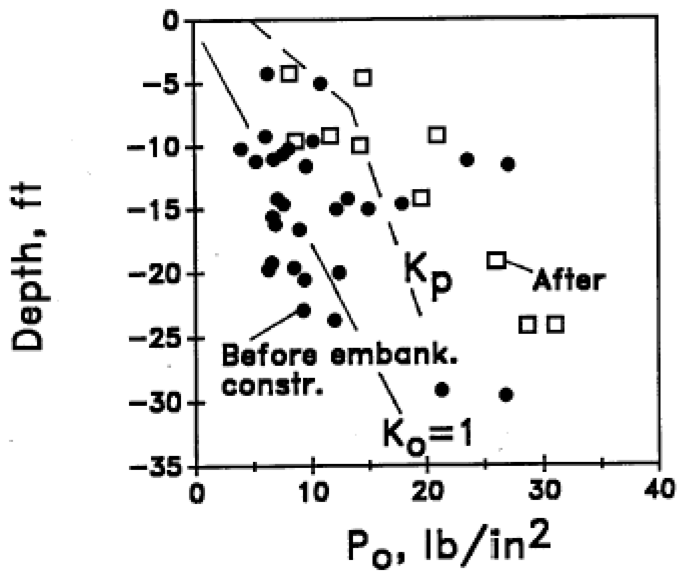

Figure 6. Lateral Stresses in Soft Clay Adjacent to Storz Expressway Embankment
Legend: • before construction, □ after construction, --- K_p line[7]

Earth Retaining Structures

With increasing land cost and right-of-way restriction, the use of earth retaining structures is becoming commonplace during new highway construction. These earth retaining structures consist of mechanically stabilized earth (MSE) walls and reinforced concrete walls. In addition, the site restraints typically require temporary excavation support by the means of shoring, driven pile lagging system or soil mix diaphragm wall. In each of these systems, a main design parameter is the measurement of in situ lateral stresses. For example, at the Ramp L construction at Boston's I-93NB/I-90 Interchange, the long-term design condition was assumed to be at-rest lateral earth pressures[8]. This stress level was used in the design of the soil-mix berm.

During construction of a diaphragm wall for the new subway system in Rotterdam, Netherlands, SBT's were performed prior to excavation of the wall to provide the designers with in situ lateral stress information. Soils at the test site are generally comprised of sand/clay fill underlain by layers of peat, organic marine clay, and sand. The results are being used as input parameters in a finite element analysis to determine stress conditions on future diaphragm walls for a new subway tunnel. As indicated by the K value and effective stress profiles, see Figure 6 and Figure 7, the lateral stress profile can be highly variable. This variability should be accounted for in the design of earth retention structures.

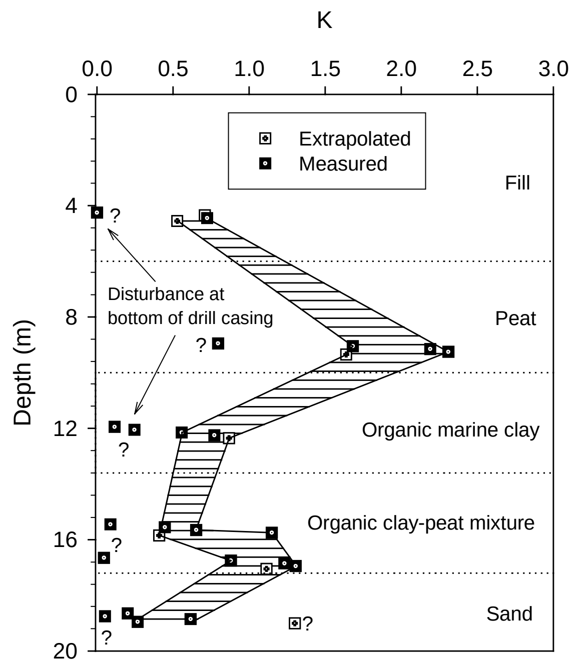

Figure 7. Coefficient of Lateral Earth Pressure (K) Profile versus Depth, Rotterdam, Netherlands[9]

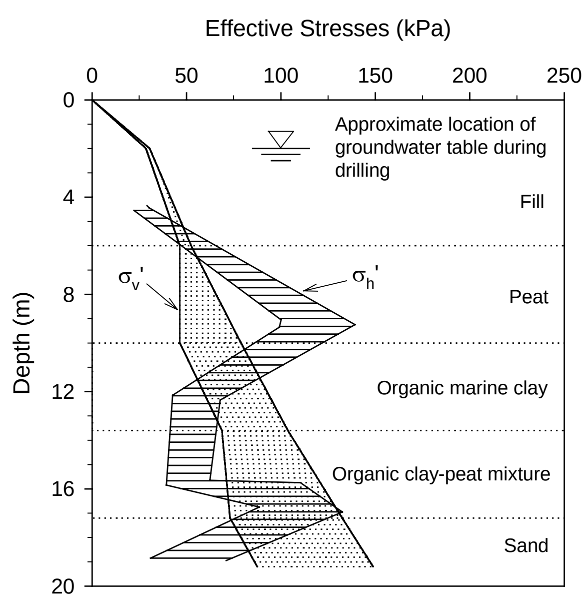

Figure 8. Comparative Results of Calculated Vertical and Horizontal Effective Stress Profiles versus Depth[9]

Conclusions

There are many geotechnical problems that are dependent on lateral stresses. These include the stability of foundations, retaining wall, slopes and embankments. The SBT device may be the most useful tool for measurement of in situ lateral stresses for design and evaluation. The SBT concept, involving extrapolation of test data to obtain a pressure on a zero-blade thickness, has been validated in the laboratory and field tests. The field tests have also demonstrated the variability of in situ stresses and the SBT device appears to provide reliable data on these conditions. Compacted or overconsolidated zones are indicated by measure high lateral stresses that are relic from previously applied vertical loads. Underconsolidated, collapsible zones are indicated by low measured lateral stresses.

The SBT device provides a method for lateral stress measurement that has become critical in the age of computer modeling. The ability to obtain lateral stress measurements increases the validity of our analytical procedures and expansion of the technology.

Acknowledgements

The authors would like to thank Dr. Richard Handy and Dr. Glen Ferguson for providing their insightful wisdom and inspiration throughout the development and use of the SBT device. For without them, the device may still be a mere sketch on a napkin.

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PEAT MAPPING USING RESISTIVITY

Paul Fisk, Keith Holster, Silas Nichols, and Peter Connors

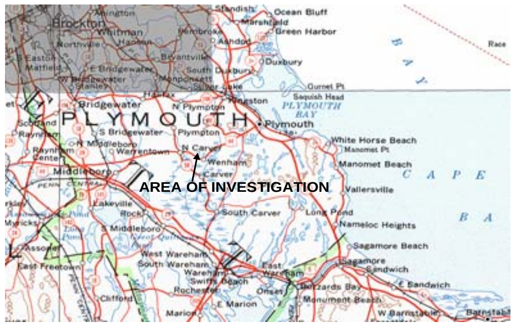
Abstract

A demonstration project was conducted to evaluate the effectiveness of Geometric's OhmMapper system to locate and define the lateral and vertical extents of peat deposits along a highway construction right of way. The test area was in Carver, Massachusetts on a section of Route 44 that is currently under construction. Peat was excavated to depths varying from 15 to 30 feet and replaced with sand. Borings indicated peat deposits below the sand fill. The Geometric's OhmMapper resistivity survey was conducted in an area where borings indicated the presence of peat. After completion of the resistivity survey the area was excavated and the peat deposits mapped for comparison with resistivity survey results. Soil, ground water, and peat samples were obtained from the site materials for laboratory resistivity tests.

The results of the Geometric's OhmMapper investigation were presented as a comparison of the color resistivity cross sections and test pit drag line results. The results indicate a correlation of the low resistivity areas and locations that peat/muck was excavated. The low resistivity values for the peat, limited penetration and the definition of the bottom of the peat layer. The method did outline the lateral and the approximate vertical extent of peat material. This contribution is significant since the most expensive part of a boring or trenching program would be to establish the lateral extent of peat.

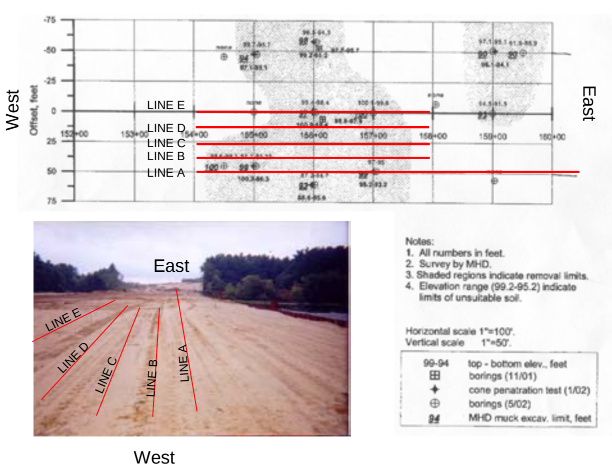
Introduction

This project was a cooperative effort between Geometrics, Massachusetts Highway Department, Federal Highway Administration and NDT Corporation to evaluate the effectiveness of Geometric's OhmMapper system to locate and define the lateral and vertical extents of peat deposits along a highway construction right of way. The test area is in Carver, Massachusetts, and Figure 1, on a section of Route 44 between stations 153+00 and 162+00 that is currently under construction. During highway construction peat was excavated to an approximate depth of 15 feet and replaced with sand. Borings post cut and fill indicated peat deposits were still present below the sand fill.



NDT CORPORATION FIGURE 1

The Geometric's Ohmmapper geophysical investigation was conducted along the lines of coverage shown on Figure 2 in an area where borings indicated peat deposits. After the geophysical investigation was completed, the area was excavated and peat deposits mapped for comparison with geophysical survey results. Soil samples and water samples were obtained from the sand soil fill, peat and ground water for laboratory resistivity values.



NDT CORPORATION FIGURE 2

Geometric's OhmMapper Resistivity Measurements

The Geometric's OhmMapper is a capacitive-coupled resistivity meter. Conventional resistivity surveys use driven metal stakes for electrodes. The Geometric's OhmMapper replaces these metal stakes with cables that provide inductive coupling to the ground. The measurements are made by dragging a set of cables over the survey area. With the Geometric's OhmMapper, resistivity measurements are made much faster and more frequently than with a driven electrode system. Measurements were made at a rate of two times per second as the array is pulled along the ground. In addition, since driven metal stakes are not necessary, resistivity surveys may easily be taken over pavement, asphalt, frozen ground, or solid rock. Maximum depth of investigation for the Geometric's OhmMapper is in the range of 20 meters, actual depth penetration depends on the earth resistivity values at the survey site.

The technique is based on an AC current acting like a capacitor. The Geometric's OhmMapper transmitter generates a 16.5 kHz AC signal that is applied to the metal shield in a coaxial cable. This metal shield acts as one plate of a capacitor the ground acts as the other. The insulation around the coaxial cable serves as a dielectric between the shield and ground plates of the cable-earth capacitor. The 16.5 kHz AC signal flows into the ground through the capacitance of the shield-earth contact. At the receiver the capacitance of the cables are charged by the voltages generated by the transmitter current flowing though the ground. This (voltage) is measured with a sophisticated AC voltage meter in the receiver. Ground resistivities can be calculated from the measured voltage at the receiver, and the known current generated at the transmitter.

The Geometrics OhmMapper uses a Dipole-Dipole electrode configuration. The Dipole-Dipole configuration consists of a pair of current electrodes spaced at a distance "B" (Figure 3 for the system used) apart and a pair of potential electrodes also spaced "B" distance apart. The "B" spacing (adjustable for 5, 10, 15, 20, + meter spacing) determines the area over which the current is applied to the ground and the area over which the voltage is measured. Large separations provide better penetration but lower resolution, smaller separations provide higher resolution but may require high current input for both depth of penetration and resolution. The distance between the center of the current electrode "pair" and the center of the potential electrode "pair" is the "A" spacing. The greater the "A" spacing the greater the depth of penetration and the earth volume over which the data are averaged. To create cross-sections (depth versus resistivity value) a number of different "A" spacings were required to define the resistivity with depth.

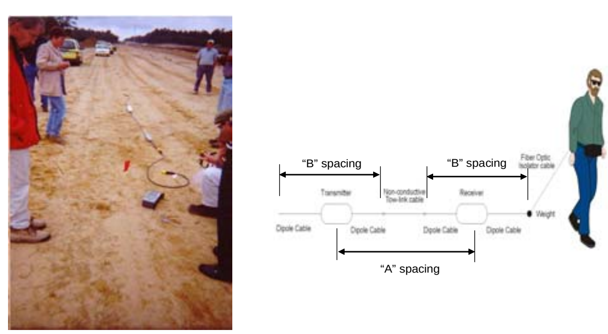


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FIGURE 3

A spacing of 5 meters for “B” was determined to be sufficient for the materials and depths to be investigated; this was kept as a constant for the survey. While data with four different “A” spacings (5, 10, 15 and 20 meters) were collected along each line. One 700 foot line, located 50 feet to the right of the centerline, between station 154+00 to 161+00 and four 400 foot line, located at 37.5, 25, 12.5 and 0 feet right of the centerline, were collected between stations 154+50 and 157+50 see Figure 2.

The resistivity measured by the Geometric’s OhmMapper is an apparent resistivity which is an average resistivity of the volume of material involved in the measurement. This is plotted as a two dimensional cross section (pseudo section) through the earth. The boundary between two layers with different resistivity values results in a smooth resistivity curve which is not unique; a combination of different resistivity and-thickness values can produce the same curve, therefore the pseudo section must be modeled.

The interpretation is by inversion modeling of the data. Known individual resistivities of the earth materials involved will provide constraints to the model. Grouping of the data values for different geometries (dipole and spacings) over the same lines of coverage is input to inversion software to calculate a more realistic resistivity and depth profile. The pseudo-sections produced by modeling provide a reasonable representation of the resistivity-depth sections but because the modeling does not produce a unique cross section of the earth some caution needs to be exercised in the interpretation of the measurements. Experience along with knowledge of the local soils and, bedrock geology (if involved) is beneficial.

Results of Route 44 Investigation

For this survey there were three major layers of interest; 1) the dry fill/overburden; 2) the water table; and 3) the presences or lack of an organic peat/muck layer:

Layer	Laboratory ohm meters	Geometric’s Ohm-Mapper range ohm meters
fill/overburden	700-1,000	>600
water table	205+/-	200-600
peat/muck	20-75	<200

The entire embankment was excavated from 154+00 to 160+00 with survey shots of the bottom of excavation (peat/muck) at 25 foot centers. The test pits were used to confirm the existence of organic muck/peat and determine the lateral and vertical extents. Samples were taken for the “special borrow”, peat/muck and ground water and resistivity values measured in the laboratory. Identification and accurate measurement of the lateral and vertical extent of the peat/muck was questionable at some locations due to the use of a drag line excavation method.

Laboratory resistivity measurements of samples retrieved from test pits are listed below:

Special Borrow	Sample #SO1		
	As received	86,201 ohm cm	862 ohm meter
	Saturated	75,681 ohm cm	757 ohm meter
Special Borrow	Sample		
	As received	97,628ohm cm	976ohm meter
	Saturated	90,378 ohm cm	903 ohm meter
Peat			
	As received	5112ohm cm	51 ohm meter
	Saturated	7,538 ohm cm	75 ohm meter
Peat			
	As received	3,777ohm cm	37 ohm meter
	Saturated	2,759 ohm cm	27 ohm meter
Water			
	As received	20,554 ohm-cm	205 ohm meter

These results indicate that the resistivity contrast between the peat and the special borrow and ground water is large enough to be identified by the resistivity measurements.

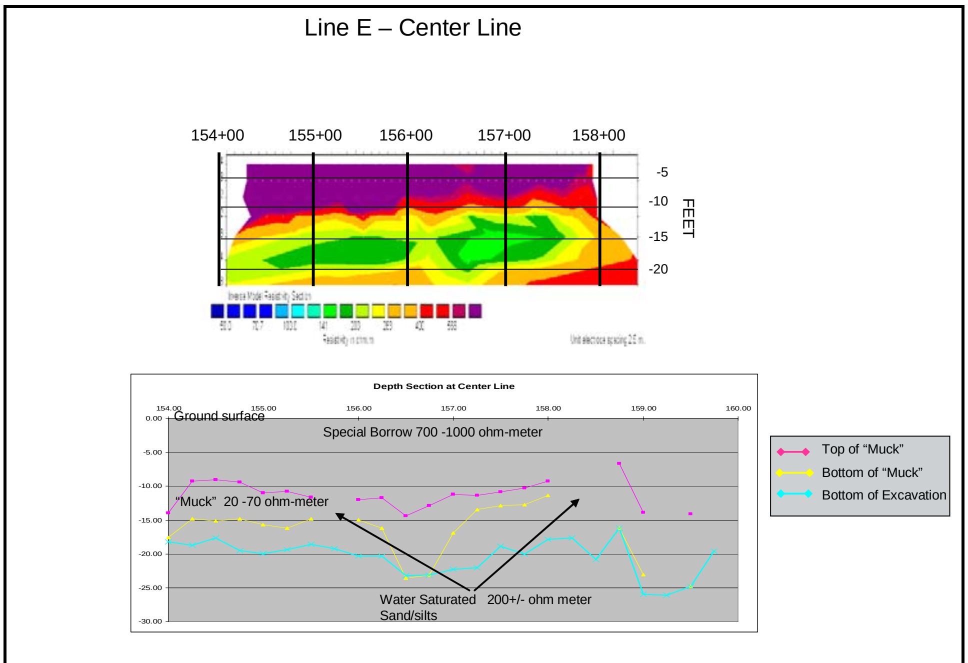

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FIGURE 4

Typical results of the Geometric's OhmMapper investigation are show on colored resistivity cross sections on Figure 4 for the lines shown on Figure 2. A comparison of inverted Geometric's OhmMapper results and test pit drag line results (Figure 4) indicate a reasonable correlation of the low resistivity areas and locations that peat/muck was excavated.

Conclusions:

The Geometric's OhmMapper provided a quick and easy method to perform an electrical survey to define the peat (compressible) material in the area of Rt. 44 construction. With the low resistivity values for the peat, penetration was limited and definition of the bottom of the peat layer is questionable. The method however will outline the lateral extent of peat material. This contribution is significant since the most expensive part of a boring –trenching program would be to establish the lateral extent of peat. The electrical survey will do this and this knowledge would serve to optimize the number of borings or backhoe excavations taken to define the bottom of peat.

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Geotechnical Management Systems
Where do we go from here?

Thomas E. Lefchik P.E.[1]
Kirk Beach[2]

ABSTRACT

An increasing amount of information associated with more complicated projects and reduced staffing are placing greater burdens on geotechnical staff. In the midst of this increasing pressure, state DOTs are looking to information technology to assist them in more efficiently managing geotechnical data, hazards, assets, and projects. The benefits in terms of time savings and cost savings can be substantial. Geotechnical assets and hazards can be more effectively and efficiently managed through the electronic storage and retrieval of data.

Several State DOTs, federal agencies, and other organizations have developed or are in the process of developing electronic geotechnical management systems of varying extent and complexity. These systems demonstrate the effectiveness of electronic geotechnical management systems for organizing and managing subsurface investigation data, geotechnical assets, and geologic hazards. The ideal system would facilitate interchange of information with other agencies and organizations permitting the exchange and use of existing information across arbitrary political boundaries or governmental agency lines.

It is time consuming and costly to develop geotechnical data management systems that work effectively to promote data exchange. Data dictionaries and data standards are critical components in the development of any geotechnical management system. Data exchange with other agencies and groups could be easily accomplished with uniform standards. Software producers could develop products, based on uniform standards, which were nationally marketable and would be compatible with other software developers.

The Federal Highway Administration (FHWA) in conjunction with the Ohio Department of Transportation (ODOT) has formed a Geotechnical Management System (GMS) Group comprised of 12 state DOTs, United Kingdom Highway Agency, USGS, USEPA, US Army Corps of Engineers, and FHWA to develop data dictionaries and data formats for geotechnical management systems. The Ohio Department of Transportation (ODOT) has initiated a pooled fund project to fund the activities of the GMS group. The GMS group is coordinating its efforts with other agencies and groups at both national and international levels.

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INTRODUCTION

State DOTs are faced with the continuing pressure to reduce staff while experiencing increased work loads. Our highway systems have grown significantly and are reaching their optimum design life. Consequently, DOTs are tasked with development and managing of an ever growing transportation system. With limited resources, each DOT is striving to improve the efficiency of their operations and better manage their staff time, funds, and highway systems.

Management systems can provide a means to assist DOTs in managing their data and highway systems while improving decision making and efficiency. DOTs have adopted management systems for pavements, bridges, culverts, traffic signs, and other assets. These systems provide an efficient means for data storage, retrieval and utilization to enhance decision making involving these assets and their maintenance.

Similarly, state DOT's geotechnical specialists are pursuing means to better manage geotechnical data (e.g., boring logs, lab test data), geologic hazards (e.g., landslides, rockfalls, mine subsidence), and assets (e.g., walls, reinforced slopes). Several states have instituted electronic data management systems to manage geotechnical data for large projects. Some states have hazard management systems in place. And some states are beginning to develop geotechnical asset management systems for elements such as piling or retaining walls.

The benefits of adopting an electronic management system for geotechnical information, assets, and hazards are significant. The lost efficiencies due to not adopting geotechnical management systems are equally significant. Consequently, many state DOT geotechnical specialists express an eagerness to adopt systems to manage the flood of incoming geotechnical data.

Ohio DOT at one time performed almost all geotechnical investigations with its own drilling crews. Now, those crews perform only about 10% of the subsurface investigations conducted statewide each year. The geotechnical investigation records for the state drilling crews are stored in a warehouse at the central DOT vehicle maintenance facilities once the projects are completed. Multiple projects are stored in each box and the boxes are indexed by the section of warehouse shelf where they are stored. Over 21,000 index cards are maintained to provide a reference to the project boxes. Frequently, box location and subsequent reference numbers are changed without updating the index cards. This makes the retrieval of information difficult and time consuming. It currently requires 20-30 person hours per week to retrieve information for rehabilitation and widening projects. Another complication involves problems with the storage facility for the data. The information is subject to water damage due to roof leakage as shown in Figures 3 & 4. In many cases, the some of the project information is now virtually inaccessible.

Figure 1 Storage of Ohio DOT subsurface investigation data.

Figure 2 Investigation data storage. Notice that the boxes are stored two deep.

Figure 3 Water stains on the floor from roof leakage.

Figure 4 Water stains on the floor from roof leakage.

This historical information is valuable for nearly all future highway projects including rehabilitation and widening.   The information stored at the central office is valued at $½ billion. An equivalent amount of geotechnical data is also stored at District offices. It is estimated that using the information will reduce the amount of drilling for projects by 10-20% resulting in cost savings of $12-24 million per year.[1]

Subsurface investigation data and reports for consultant designed projects are placed in their respective project files residing at each District office. This information is held in the file until several years (usually about 7 to 8 years) after the completion of the project. Then, the project files are purged and disposed of. This practice results in the loss of geotechnical data valued at an estimated $52 million per year.

Figure 5 This is the final storage location for $52 million of subsurface investigation data every year. The information will be difficult to retrieve in the future

.

The use of electronic data management systems would not only permit data to be more securely stored and more easily retrievable, they would also permit the data to be widely shared throughout the state DOT, with their consultants, and with other agencies. In addition, they could provide information for better planning and more thorough subsurface investigation programs resulting in higher quality designs and fewer problems during construction.

Geotechnical management systems could also incorporate inventories of geologic hazards and geotechnical assets providing not only location information but also construction information, maintenance history, materials data, and other important site information. Hazard management is becoming increasing important because of liability issues with state DOTs. Asset management has become increasingly important because the complexity and extensiveness of our growing highway systems and the corresponding difficulty in tracking asset information. Who knows where the retaining walls are, when they were built, what type they are, and the backfill and material information? This information will be increasingly important as the assets deteriorate with age.

Another important aspect of geotechnical management systems is the ability to provide information for budgeting decisions. The Ohio DOT is now fortunate to have an annual statewide budget for correction of geologic hazards. Ohio DOT has an operating management system for abandoned underground mines and is developing systems for rockfalls and slides. When these systems are operational they will provide valuable information to justify future annual funding allocations.

CURRENT EFFORTS

Virginia DOT and other state DOTs have successfully implemented geotechnical data management systems for specific large highway projects. These systems allowed the state DOT, consultants, and contractor personnel to input and access the subsurface investigation and testing data with security controls. Some state DOTs such as Florida, Kentucky, and Ohio are developing more comprehensive geotechnical management systems that will eventually include subsurface investigation, lab testing, in-situ testing, construction control and testing, assets inventory, hazard inventory and rating matrix, maintenance, and research information.

The United Kingdom Highway Agency (UKHA) has a system in operation that includes boring log data and geotechnical assets inventory and rating information. The system is used to manage their highway system and also to evaluate the effectiveness of the companies that they hire to manage their highway system. The UKHA estimates that proactive maintenance results in up to 80% savings.[2]

The Consortium of Strong Motion Observation Systems (COSMOS) is developing a geotechnical data management system that includes a Geotechnical Virtual Data Center (GVDC) that collects data from numerous utility companies, universities, and local, state and federal agencies and makes that information available for dissemination via the internet. COSMOS has drafted the boring log part of the system and is beginning work on geophysics data and testing data.

The Association of Geotechnical and Geoenvironmental Specialists (AGS), based in the United Kingdom, developed a data dictionary and flat file data format for storage of geotechnical data often used by their members. The data dictionary and data format are widely used around the world. The AGS standards are also used in the UKHA management system and were used as a starting point in the development of the COSMOS system.

WORKSHOP

The FHWA and Ohio DOT jointly funded a synthesis of practice of the use of geotechnical management systems by state DOTs and others. A Geotechnical Management System Workshop, jointly sponsored by FHWA and COSMOS, was held in Newport Beach, California in June 2004 to present the results of the synthesis, to discuss state DOT geotechnical management system needs, and to present the work of COSMOS, UKHA, and AGS. A breakout session of the representatives of the nine state DOTs represented was held at the workshop.

The state DOT representatives were very interested in pursuing the development of standards for geotechnical management systems. These standards would include a data dictionary, and the data format. The data dictionary would define all data terms. The data format would define how the data is presented.

The establishment of a standard data dictionary and data format will allow the exchange of information among local, state, and federal agencies and others. A state DOT highway project could conceivably take advantage of subsurface investigation data obtained in the same area by the state geological survey, state EPA, USGS, US Army Corps of Engineers, USGS, and others. Once in a standardized format, the information could be exchanged electronically via CD or the web and utilized by any software that uses the same data standard.

The adoption of geotechnical data standards by state and federal agencies will have a positive impact on software suppliers and their customers. They now use proprietary data standards which creates problems of compatibility of data exchange between software packages from different suppliers. If a state DOT currently wants to change from one boring log software supplier to another, the old data may not be compatible with the new data base. Many of these problems would be eliminated with a single data standard and would enable access to a larger market for products based on the new data standard.

Starting with the data standards has advantages. The data dictionary and data format are the basis of all other management system work. They are also the most difficult and time consuming elements.

GEOTECHNICAL MANAGEMENT SYSTEM GROUP

Based on the interest from the state DOTs represented at the June 2004 workshop, the FHWA and the Ohio DOT formed a Geotechnical Management System (GMS) Group. The goal of the group is to develop an open and flexible geotechnical management system generic framework that can be web enabled; can be used to store, retrieve, and manipulate data; can store, retrieve, or other wise access geologic information; provides a means to efficiently and proactively manage geotechnical assets and geologic hazards; can store and manage project data and test data; can be used as a tool to share information among interested entities; and can accommodate modifications to meet local needs. The GMS group will direct the development of a data dictionary and data format.

The GMS group will accelerate, enable, and facilitate the development of geotechnical management systems by developing frameworks, standards and protocols that will create a large commercial market and competition for software development, management system maintenance, new software and application tools. All frameworks, standards, and protocols will be open and flexible allowing for customization within agencies, direct interchange of data and information among software from various sources, and future expansion and modification as needed.

A benefit of the work of the GMS group, to all entities in need of a geotechnical management system, will be the reduction of cost and time required to develop their customized systems. This will be accomplished by reducing redundancy in the GMS efforts and by the collaboration that ensures operational compatibility of GMS on both a macro and modular scale.

Members of the Geotechnical Management System Group:

- California DOT
- Florida DOT
- Kansas DOT
- Kentucky DOT
- Minnesota DOT
- Missouri DOT
- North Carolina DOT
- Nebraska DOT
- Nevada DOT
- Ohio DOT
- South Carolina DOT
- Virginia DOT
- FHWA
- FHWA Federal Lands
- United Kingdom Highway Agency
- United States Army Corps of Engineers
- United States Environmental Protection Agency
- United States Geological Survey

From initial meetings, the GMS group decided to use XML schema that is GML compliant in the data standard. HTML or hypertext markup language is used to define how data is presented electronically. It is the most widely used standard for web based presentation. XML or extensible markup language is the new standard that is being adopted because it is much more flexible. XML allows additional data elements to be added to a data base without completely changing the data base. This feature has significant advantages for a state DOT that wants to use the standard but also wants to keep some data that is not included in the standard. GML or Geospatial Markup Language follows XML schema with the addition of geographic tags to locate the data geospatially.

The development of the standards will be funded through a pooled fund project directed by the Ohio DOT. The final products will be a data dictionary and data format for geotechnical data including all or most geotechnical assets and geologic hazards. The development of standards for assets and for hazards will depend upon the availability of sufficiently defined data criteria.

With the cooperation of state and federal agencies and with the international participation of the major associations responsible for geotechnical data compilation, it is anticipated that these standards will be adopted as both a national and international standard.

GEOTECHNICAL DATA COALITION

A Geotechnical Data Coalition was formed with representatives from the University of Florida, AGS, COSMOS, FHWA, Ohio DOT, and the Construction Industry Research and Information Association (CIRIA). A core team from this coalition will consolidate the existing data standards, perform a survey of data needs of the state DOTs and others, and develop the data dictionary and data format. The GMS Group will oversee and approve the work of the coalition.

The survey will include state DOTs and other agencies and groups. The survey will be web based and will be comprehensive. It is vital that this information be complete so that all state DOT needs are adequately considered. There will be a significant effort required by each state and group in responding to this survey. It is expected that a minimum of a person week of effort will be required by each state and group responding to this survey. The GMS group will solicit state and group cooperation and assistance in completing the survey so that each state's needs are adequately considered. It is the intent of this project that the results are applicable and beneficial to all states and participants.

Most of the work by the coalition will be voluntary or contributed by others. The pooled fund project will fund travel expenses for meetings, printing costs, graduate student expenses, and some other costs.

The cooperative nature of this group will permit the work to progress quickly. The first product will be a data dictionary and data format for borehole data. This work should be available in several months following the initiation of the project. The final data dictionary and data format for all data is projected to be completed by mid 2007.

SUMMARY

There is a great need for the development and use of geotechnical management systems by state DOTs because of increasing workload, increasing data, reduced workforce, and the aging highway system. Geotechnical management systems would enable DOTs to efficiently store and retrieve data resulting in efficient use of time and better and less costly subsurface investigations. In addition, GMS systems would permit efficient management of geotechnical assets and geologic hazards. Management systems also provide the means for better budget justifications.

Consequently, there is a great interest among state DOT geotechnical specialists for geotechnical management systems and a desire for prompt implementation. A group of state DOTs and other agencies was formed and is working on the development of a standard data dictionary and data format for geotechnical management systems. The Ohio DOT has issued a pooled fund solicitation to provide funding for the development of the standards.

A coalition of organizations was formed to cooperatively perform the work of consolidation of existing standards, survey of state DOT and other agency needs, and development of the standards. This coalition will perform this work mostly on a voluntary basis.

The data standards will be developed quickly. A draft version of the boring log data dictionary and data format will be issued first. It is anticipated that the entire data dictionary and data format will be finalized by mid 2007.

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MASW – From Detailed Site Investigations to Regional Surveys Along Roadways: Advantages and Limitations

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Abstract
Multichannel Analysis of Surface Waves (MASW) is a surface geophysical technique that uses the dispersive characteristics of seismic surface waves to model the variation of seismic shear-wave velocity with depth. Shear-wave velocity is a key parameter for the evaluation of elastic properties of soil and rock. The application of this technique has become quite popular in the last few years to both environmental and geotechnical projects.

This method, like all field measurements, has site-specific constraints that may limit its usefulness, and is not applicable to all projects. MASW data can be acquired in a variety of environments ranging from soft soils to hard roadways and even shallow marine conditions. Survey parameters can be tailored to provide detailed information over known anomalous features (e.g. paleocollapse and buried channels) or can be used in reconnaissance mode to map regional features (e.g. stratigraphic and structural trends). When the results of MASW measurements are integrated with supporting geophysical data and borings, a more complete and accurate subsurface characterization can be made.

Introduction

Multichannel Analysis of Surface Waves (MASW) is a geophysical method that uses the dispersive characteristics of surface waves to determine the variation of seismic shear-wave velocity with depth (Park, et al., 1999). Shear-wave velocity is a function of the elastic properties of the soil and rock and is directly related to the hardness (N-values) and stiffness of the materials. MASW can be applied to a wide-range of investigations including mapping bedrock, identifying voids and collapse features, and analyzing the structural stability of subsurface materials (Xia, et al., 2004b; Miller, et al., 1999). The method has become quite popular in the last few years to both environmental and geotechnical projects.

MASW is a non-intrusive geophysical method that is performed on the ground surface. The method was adapted from the Spectral Analysis of Surface Waves (SASW) (Stokoe et al., 1994). MASW has advantages over SASW, since data are generally recorded at 24 or more locations at one time compared to only 2 locations with SASW. This maximizes signal-to-noise ratios, provides greater confidence in the identification of surface waves and improves field production rates, essentially providing greater data density (Xia et al., 1999; Park et al., 1999).

Data Acquisition and Processing

Data are acquired by recording the arrival of seismic surface wave energy generated by an impulsive source and received by a linear array of geophones (Figure 1). The seismic source may be a sledgehammer, a mechanical impact device, a shotgun, or explosives, depending upon the depth of investigation and site-specific conditions. Typically, 24 to 48 geophones are used for a single array, with a constant inter-geophone spacing that is optimized for site-specific geologic conditions (which affect the degree of surface wave dispersion, attenuation, body wave contamination, etc.), as well as for the desired measurement depth and spatial resolution. A multichannel seismograph is used to digitally record the data.

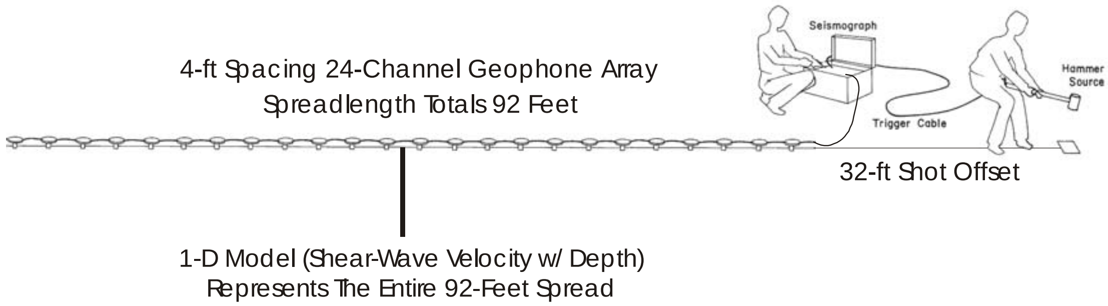

Figure 1. Typical field setup showing 24-channel spread

Acquisition parameters for an MASW survey are based on established procedures (Zhang, et al., 2004) and on-site testing. Traditional MASW measurements are made on land with the use of low-frequency geophones (e.g. 4.5Hz). A constant shot offset from the linear array of geophones is used for each MASW measurement and is chosen to provide a high degree of wave dispersion and signal strength. In softer ground (e.g. grass-covered areas), geophones can be planted in the ground. In areas where the ground is hard (e.g. asphalt, concrete or compacted gravel), geophones can be mounted in a land streamer configuration and the geophone array can be pulled down the survey line for a series of measurements. The land streamer can be pulled using an automobile or ATV, keeping a constant offset between the shot point and the first geophone in the array.

MASW data is then processed to provide 1-D shear-wave velocity models. For each recorded shot, a dispersion curve is picked from a calculated phase velocity frequency spectra (Figure 2). A shear-wave velocity profile (1-D profile of shear-wave velocity as a function of depth) is then modeled from the dispersion curve using a least-squares inversion routine (Xia, et al., 1999; Park, et al., 1999). Each shear-wave velocity profile corresponds to the surface wave response over the entire spread length for a given shot. Therefore, the 1-D model represents bulk shear-wave velocities and corresponds to the middle of the geophone spread. A 2-D cross-section of shear-wave velocity models is built by acquiring multiple shots along a profile line.

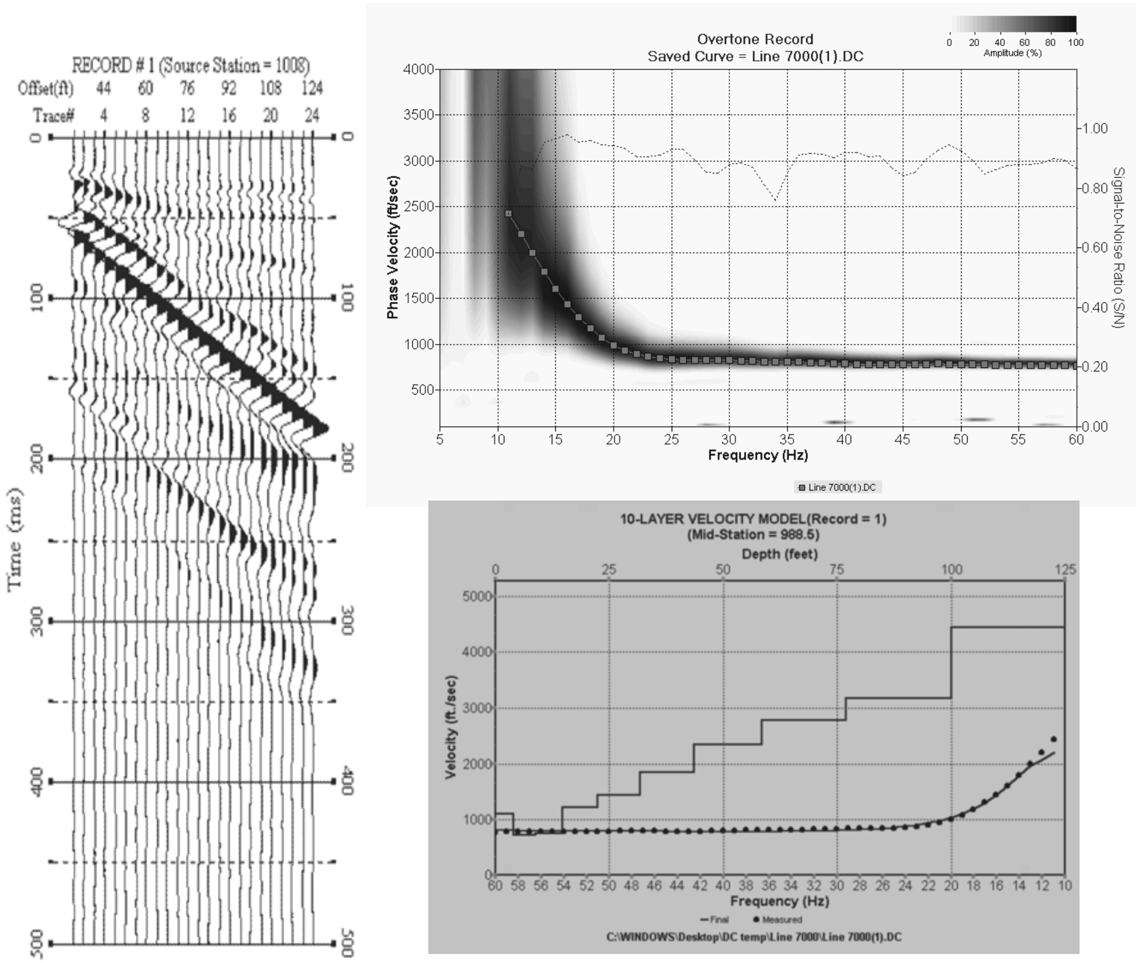

Figure 2. Typical shot gather (left), phase velocity frequency spectra with picked dispersion curve (top right) and resulting 1-D shear-wave model (bottom right)

Table 1 shows general classifications of soil and rock based on shear-wave velocity (BSSC, 2000). In general, lower velocity values correspond with softer or weaker materials.

Table 1. Soil and Rock Shear-wave Velocity Classification

Velocity (ft/s)	Classification
<600	Soft Soil
600 to 1,200	Stiff Soil
1,200 to 2,500	Very dense soil and soft rock
2,500 to 5,000	Rock
> 5,000	Hard Rock

Depth, Resolution, Accuracy and Precision

The depth of investigation is limited by the seismic source, the frequency of the geophones, and the geophone spread length. In general, the maximum depth of investigation is approximately one-half of the longest wavelength of the recorded surface waves. In typical conditions, maximum depths of 60 to 100 feet can be achieved, however this is site specific.

The model resolution of inverted surface waves is related to the accuracy in which the dispersion curve can be defined. Ideally, if error free data are inverted, the model could be perfectly resolved. In the real world, smear in the phase velocity frequency spectra reduces the vertical resolution of the model (Xia et al., 2004a). Vertical resolution is approximately 20% of the depth (e.g. features at a depth of 20 feet, will be averaged over a thickness of approximately 4 feet). Lateral resolution is approximately 25% of the length of the geophone array.

Accuracy (bias) of a MASW measurement is verified by control points (e.g. borehole information), which can be used to constrain the interpretation of the data. Field procedure errors, processing errors, instrument errors, noise, topography, and lateral geologic variability can contribute to errors in the interpretation. Comparisons of shear-wave velocity values obtained with MASW and borehole measurements indicate that MASW velocity models are accurate to within 15% of actual values (Xia et al., 2000, 2002a, and 2002b). Precision (repeatability) of a MASW measurement will be affected by the sources used, placement of geophones, soil conditions, the defining of dispersion curves, and the site-specific noise levels.

Case Histories

The following data examples show the many uses of the MASW method, as well as its limitations:

<u>Paleocollapse, Tarpon Springs, Florida</u>

Technos Inc. made some MASW measurements at a superfund site located in Tarpon Springs, Florida in May of 2004. The general geology at the site consists of blanket sands from the surface down approximately 20 feet over a variable semi-confining clayey layer that lies above weathered limestone bedrock. Geophysical and boring data defined a paleocollapse feature that is approximately 200 feet wide and over 125 feet deep. The paleocollapse is filled with sand and organic material that dates the collapse to approximately 40,000 year ago. A line of MASW data were acquired over this paleocollapse and compared with the complimentary geophysical data sets.

The MASW data are generally of good quality, with frequencies ranging between 10 and 70 Hz, corresponding to maximum depths of approximately 75 feet. Shear-wave velocity values generally increase with depth and range between 400 and 2,900 ft/s. At this site, velocity values less than 1,200 ft/s are interpreted as sand and unconsolidated material, while velocity values greater than 1,200 ft/s are interpreted as soft, weathered limestone. The shear-wave cross-section calculated from MASW data for this survey line is shown in Figure 3. A microgravity profile along the same portion of this line is shown with the cross-section. The cross-section shows the interpreted top of very weathered rock deepening from approximately 22 feet to greater than 75 feet within the paleocollapse. A boring placed near the center of the paleocollapse shows that no rock was encountered to a depth of 125 feet. There is a very good correlation between the interpreted deepening of the weathered limestone and the microgravity low. The low velocity values correspond with the unconsolidated material that has filled the paleocollapse.

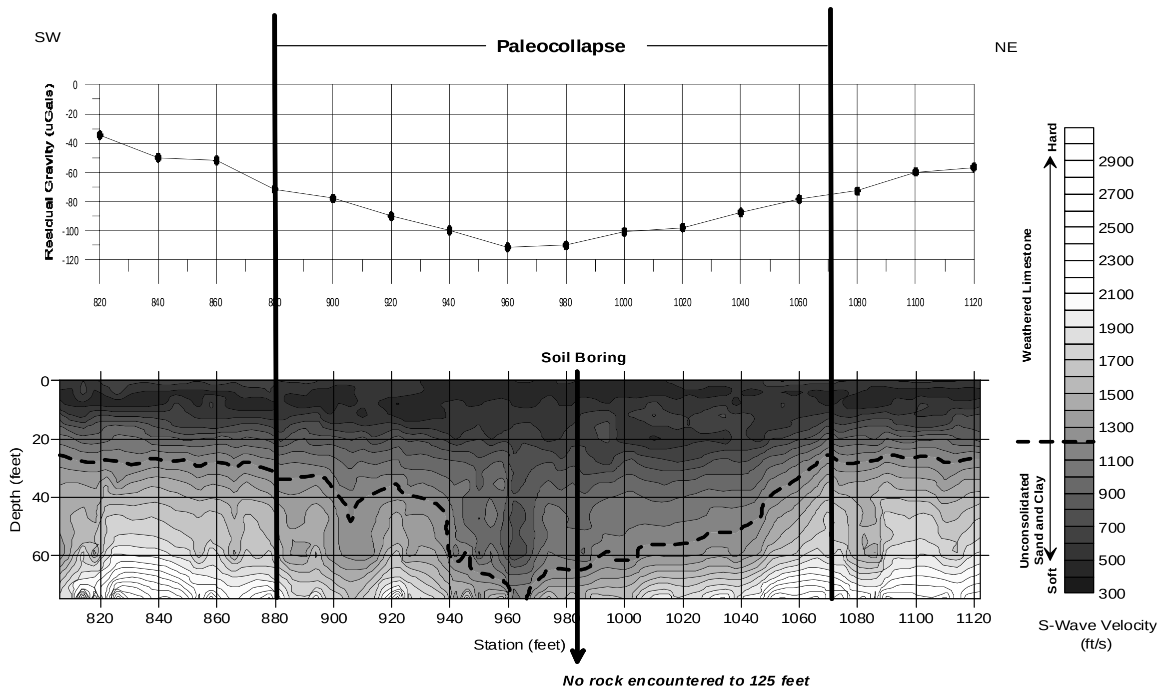

Figure 3. MASW data acquired over a paleocollapse in Tarpon Springs, Florida

Sinkhole, Gainesville, Florida

In the spring of 2004, the University of North Florida (UNF) was conducting a research project at a dry, 1.5-acre drainage basin located in Gainesville, Florida. A former sinkhole was located in the western portion of the basin and was filled with sand and concrete by the Florida Department of Transportation (FDOT). The general geology at the site consists of surface sands over a variable zone of weathered limestone bedrock that lies above a deeper, more competent limestone. MASW data were acquired within the drainage basing along survey lines where several borings were previously made by FDOT. The borings provided data showing the depth to the top of weathered limestone bedrock as well as any voids that were encountered within the limestone.

The MASW data are generally of good quality, with frequencies ranging between 8 and 70 Hz, corresponding to maximum depths of approximately 75 feet. The soft surficial sands at the site attenuated higher frequency energy (>30 Hz) in many of the shots. Shear-wave velocity values generally increase with depth and range between 400 and 3,500 ft/s. At this site, low velocity values (<1,200 ft/s) are interpreted as sand, mid-range velocity values (1,200 to 2,300 ft/s) are interpreted as soft, weathered limestone, and high velocity values (>2,300 ft/s) are interpreted as harder limestone. The shear-wave cross-section calculated from MASW data is shown in Figure

4. There is a very good correlation between the interpreted top of weathered limestone and the confirmed top of weathered limestone reported in the boring data. The MASW model shows a highly variable top of weathered rock along the survey line, as well as several low-velocity zones, potentially due to areas of soil raveling or voids within the limestone. One of the low-velocity zones corresponds to an area within the former sinkhole in which a void was encountered in one of the borings.

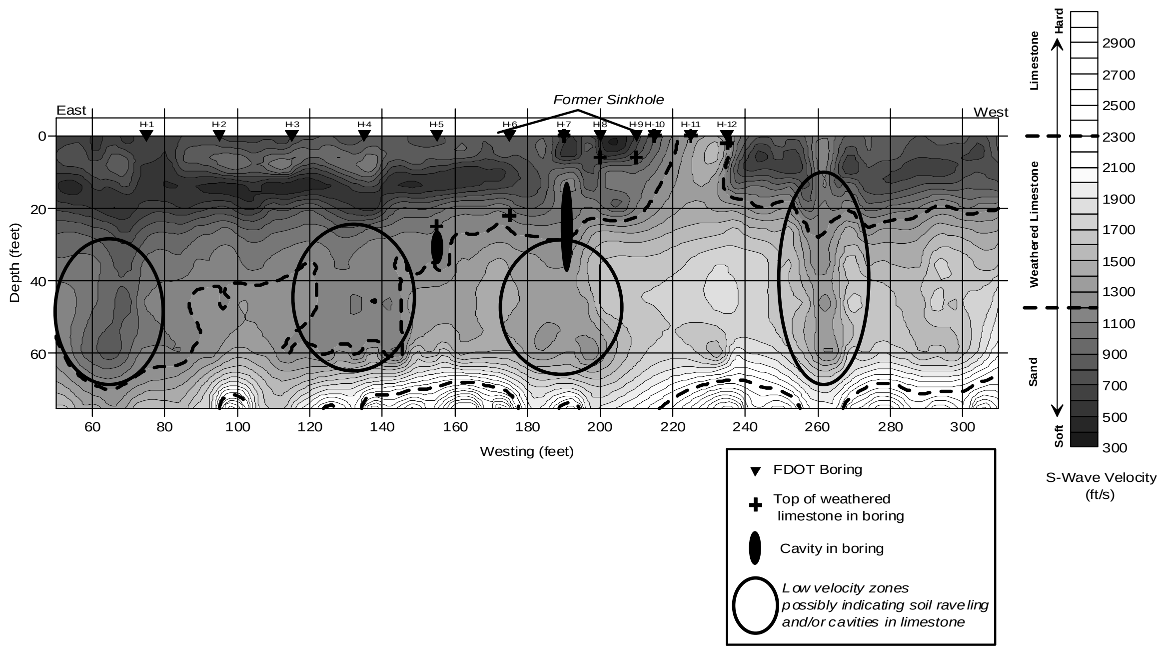

Figure 4. MASW data acquired within a drainage basin in Tallahassee, Florida

<u>Shallow Marine Environment, Virginia Key, Florida</u>

Traditional SASW and MASW measurements are made on land with the use of low-frequency geophones that are either planted in the soil or pulled along the surface with a landstreamer. In marine environments, gimbaled geophones and ocean bottom seismometers have been used to obtain surface wave measurements (Luke and Stokoe, 1998; Bohlen et al., 2004). Because of the high amplitude nature of surface waves, conventional hydrophones laid at or near the bottom can be as effective as underwater geophones for detecting surface waves (Park et al., 2000; Klein et al., 2000).

MASW data were acquired in a shallow marine environment near Key Biscayne, Florida to assess the data quality of MASW measurements obtained with a hydrophone streamer. Measurements were made on land adjacent to the marine tests using a traditional geophone array to allow a direct comparison of the data quality obtained with the hydrophone streamer (Figure 5). The results of the tests indicate that sufficient quality surface wave data can be obtained with the use of hydrophones in a shallow marine environment. The dispersion curves and resulting shear-wave profiles are consistent with land-based measurements and correlate with expected geologic conditions. The data are an effective complimentary measurement to more traditional marine seismic reflection and refraction surveys.

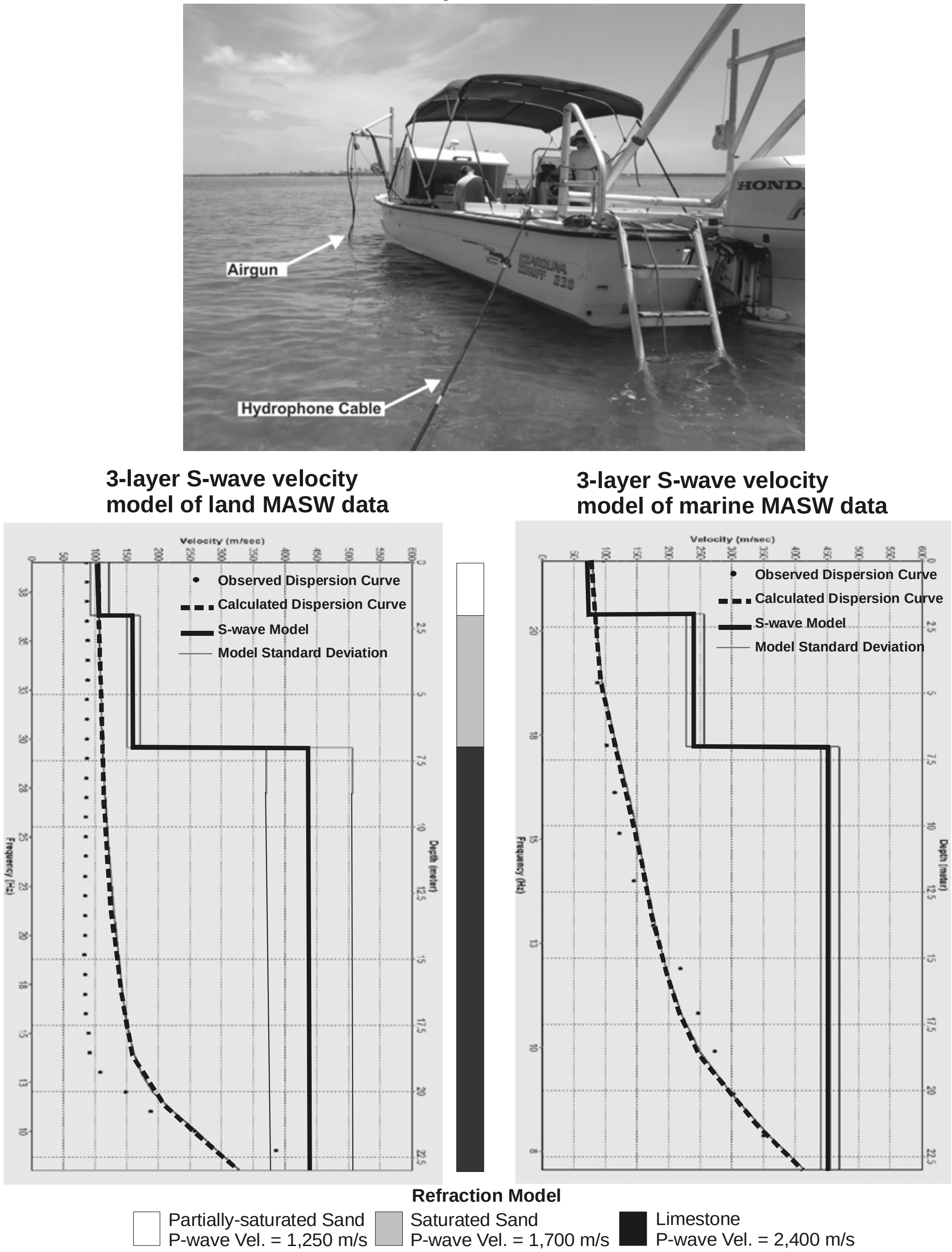

Figure 5. Comparison of MASW data acquired using a hydrophone streamer and standard land-based geophones

The usefulness of underwater shear-wave measurements is immense. Since S-waves cannot be transmitted through water, the shear-wave velocity is directly related to the elastic properties of the sub-bottom sediment and rock. In contrast, compressional wave (P-wave) methods such as seismic refraction are influenced by the presence of water, especially in soft, saturated sediments where the P-wave velocity is close to that of water.

Faults/Karst, MSFC, Huntsville, Alabama

In the second half of 2004 and early 2005, a large-scale karst investigation using surface and borehole geophysical methods was carried out by Technos, Inc. at the George C. Marshall Space Flight Center located in Huntsville, Alabama. The purpose of the investigation was to characterize the geologic conditions and to identify features that act as preferential pathways for groundwater flow. MASW data were obtained along 66 survey lines over several miles across the site. While most of the survey lines were located within specific groundwater contaminant source areas, sixteen of these survey lines were established as part of a site-wide regional study, some of whose lengths spanned more than one mile across the site. The geophones were used in a landstreamer configuration along most of the survey lines, attached to weighted metal plates that could be dragged along the surface (Figure 6). Much of the MASW data were recorded along the sides of major roadways that ran across the site. The general geology at the site consists of unconsolidated clayey residuum overlying limestone bedrock. The residuum is a by-product of limestone weathering and is described in geologic logs as clay, silt, and sand with varying amount of chert and limestone fragments.

The MASW data were generally of good quality across the site, with frequencies ranging from 7Hz to much greater than 60 Hz, corresponding to maximum depths of approximately 100 feet. Shear-wave velocity values generally increase with depth and range between 400 and 4,500 ft/s. At this site, low velocity values (<1,400 ft/s) are interpreted as soil (clays), mid-range velocity values (1,400 to 2,400 ft/s) are interpreted as soft, weathered limestone, and high velocity values (>2,400 ft/s) are interpreted as harder limestone.

The MASW models reveal a varying bedrock depth with a varying degree of weathering across the site. Figure 7 shows a shear-wave model cross-section calculated using data collected with the landstreamer along a gravel road. A microgravity profile along the same survey line is shown with the cross-section. The cross-section shows the interpreted top of weathered limestone and deeper, denser limestone.

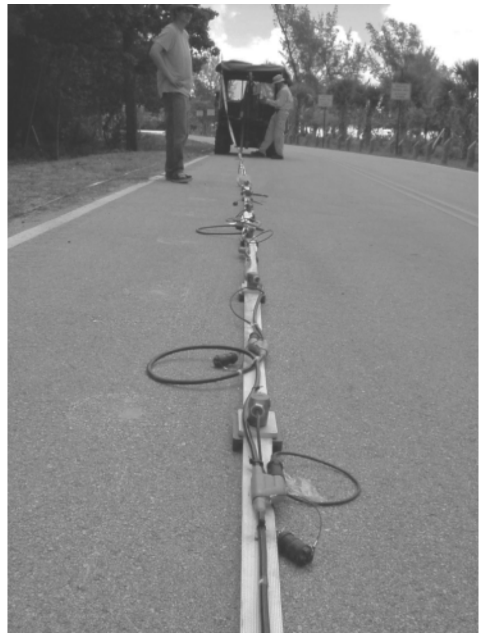

Figure 6. Landstreamer setup for a 24-geophone array pulled with an ATV along a road

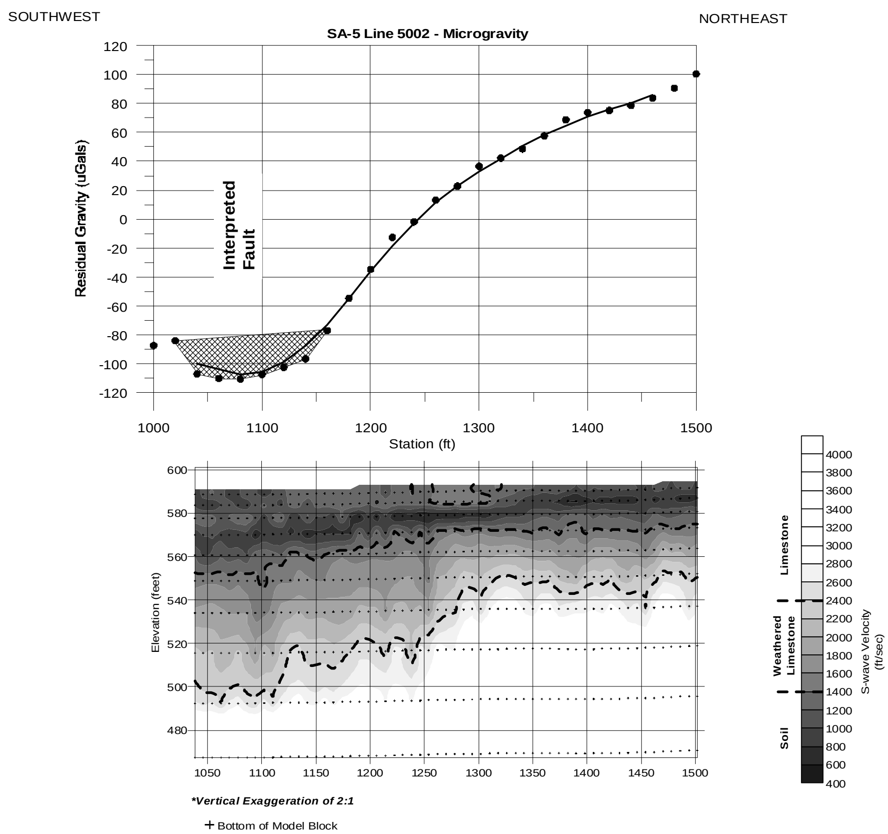

Figure 7. MASW data acquired with a landstreamer along a gravel road, MSFC, Huntsville, Alabama

In Figure 7, the MASW model shows a decreasing elevation of the top of rock and greater weathered zone thickness toward the west. An interpreted fault at the western portion of the survey line correlates remarkably well with the shear-wave model. There is also very good correlation with the microgravity data, which shows decreasing values towards the west suggesting a deepening bedrock surface and/or decrease in subsurface density (i.e. greater amount of bedrock weathering).

Poor Quality MASW Data

The previous MASW examples illustrate good quality data. However, there are advantages and disadvantages with all geophysical techniques. As with all seismic techniques, noise can be introduced by a variety of sources such as vehicle traffic or machinery operating at some nearby facility. This type of noise can affect the quality of the dispersion curve and subsequent shear-wave models. In addition, all of the standard requirements for a seismic survey need to be met. The quality of the MASW data can also be affected by natural geologic conditions that may not produce well-defined dispersion curves and cannot be used to calculate reliable shear-wave velocity models. Two examples are presented.

MASW data were acquired over several test lines where a thin (<10 feet) layer of soils was overlying limestone bedrock. The relatively shallow depth and uniform velocity structure of the limestone bedrock resulted in nearly non-dispersive seismic surface waves. In addition, the surface wave energy that was recorded at the site was confined to a very narrow frequency band (~25Hz to 45Hz) due to the site-specific surface conditions and wave energy attenuation. Figure 8 shows the calculated phase velocity frequency spectra from one of the test shots at the site. The definition of the dispersion curve is very limited. Over such a narrow frequency and velocity range, any small variations in the dispersion curve (e.g. discrepancy in the dispersion curve picks between different interpreters) can drastically change the final model output. Therefore, such data will not yield reliable shear-wave models.

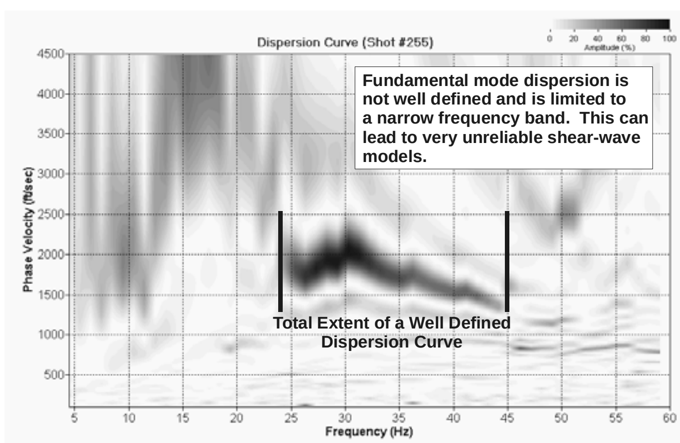

Figure 8. Example of poorly defined surface wave dispersion

Figure 9 shows the phase velocity frequency spectra for sequential shot records along a survey line. Rayleigh waves travel along the surface in multiple modes of propagation. When picking a dispersion curve, it is the fundamental mode that is used in the inversion process (although it has since been suggested that the use of higher modes can improve model resolution, Feng et al., 2005). In this case, higher mode dispersion curves are prevalent in the records (Figure 9, top left). The fundamental mode dispersion curve is not well defined over a very large frequency range. Due to changing subsurface conditions as the shots progress down the survey line, the 1st higher mode dispersion curve merges into the fundamental mode dispersion curve. This yields an apparent fundamental mode dispersion curve (Figure 9, bottom right), which does not define the true dispersion of the fundamental mode Rayleigh wave. Improperly selecting this apparent dispersion curve as the fundamental mode dispersion curve will result in a model with incorrect shear-wave velocities. This example shows the importance for a sufficient knowledge and understanding of the method in order to successfully apply it to each unique geologic situation.

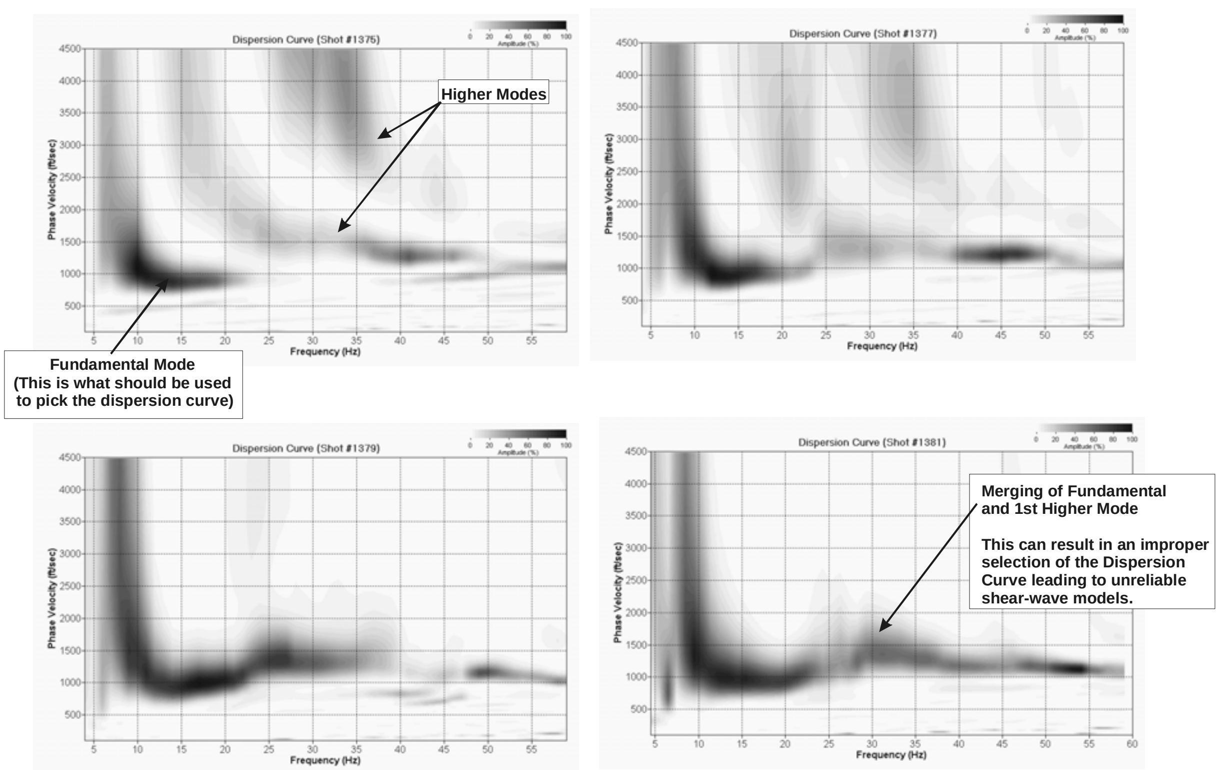

Figure 9. Phase velocity frequency spectra for sequential shot records (from left to right, top to bottom) showing higher modes and merging modes of Rayleigh wave propagation

Conclusions

As the previous cases have shown, MASW data can be acquired in a variety of environments, ranging from soft soils to hard roadways and even in shallow marine conditions. Survey parameters can be tailored to provide detailed information over known anomalous features (e.g.

paleocollapse and buried channels) or can be used in reconnaissance mode to map regional features (e.g. stratigraphic and structural trends).

This method, like all field measurements, has site-specific constraints. This means that MASW may not be applicable to all projects, and other methods may be deemed more appropriate to solving the problem at hand. When used appropriately, however, the results of MASW measurements can be integrated with supporting geophysical data and boring information, and a more complete and accurate subsurface characterization can be made.

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I-40 SLOPE REPAIRS IN WESTERN NORTH CAROLINA

By: Nilesh M. Surti, P.E., NCDOT Geotechnical Engineering Unit

ABSTRACT

The 2004 hurricane season wreaked havoc in western North Carolina from four different storm events with immense rainfall. These rains caused a massive amount of damage to the communities and transportation facilities in western North Carolina. The Pigeon River, swollen with runoff from Hurricanes Jeanne and Ivan and a flood release from the Walters Dam scoured away the toe of several embankment slopes supporting Interstate 40 near the North Carolina-Tennessee border. On September 17, 2004, several landslides occurred between Mile Markers 1 and 4. Portions of eastbound I-40 fell into the river. I-40 was closed in both directions and traffic was rerouted.

The NCDOT was faced with the challenge of re-opening all lanes of traffic on I-40 to the traveling public as soon as possible. Numerous units from the Design Branch and the Division Construction staff of the NCDOT had to work together within a tight schedule in order to accomplish this task.

PROJECT INFORMATION

Most of the large slope failures occurred along a stretch of I-40 from one mile east of the Tennessee border to approximately 5 miles into North Carolina. The embankment failures were along the eastbound lanes of I-40 which parallels the Pigeon River. A tunnel is present near MM4 and a tunnel detour road is located on the southern side where the Pigeon River runs parallel. The slides and their locations are listed below:

Slide Number	Location on I-40	Approximate Lengths
1	MM4 near Tunnel	350 feet
2	Near MM 3.5	600 feet
3	MM3	200 feet
4	MM1	100 feet
5	Along tunnel detour	750 feet

Slides were numerically categorized from the tunnel westward beginning with Slide Number 1. The tunnel slide was discovered later and then was numbered Slide 5.

BACKGROUND

North Carolina and many other states in southeastern United States faced an onslaught of rains in the 2004 hurricane season. Hurricanes Charley, Frances, Ivan, and Jeanne each made a path through the piedmont and western North Carolina from August to September 2004. In September, three tropical storms (Frances, Ivan, and Jeanne) had a major impact on the total rainfall amounts over much of western North Carolina. The rainfall totals ranged from 10 to 25 inches, which was 200 to 500 percent of normal. The city of Asheville set a new record for their September rainfall amount with 13.71 inches. The previous record was 9.12 inches set in 1977. Haywood County, which was the location of the major landslides along I-40, received almost 12 inches of rain. Excessive rainfall of this magnitude caused numerous flash floods, several mudslides, slope failures, and serious river flooding.

Interstate 40 parallels the Pigeon River in the western most part of North Carolina. The massive water flow of the Pigeon River from Tropical Storm Ivan and releases from Walters Dam undermined the toe of embankments in numerous locations along I-40 on September 17, 2004. From USGS data, the Pigeon River had a flow of 19,800 cfs (cubic feet per second) on September 8th during Hurricane Frances and 17,100 cfs on September 17th during Hurricane Ivan. The daily mean gage height of the Pigeon River has been approximately 3 feet but on September 17, 2004 the gage height increased to a maximum of almost eighteen (18) feet.

This tremendous amount of water caused slope failures at almost every bend in the Pigeon River along I-40. The most notable slope failure was between Mile Marker 3 and 4 where an almost 90-degree curve occurred in the river. At that bend, a 200-ft long section of the I-40 shoulder had fallen into river with the guardrail hanging like a thread along the slope. State troopers were alerted and on site to close down the Eastbound lanes of I-40 for the safety of the traveling public.

DESIGN SCHEDULE

The upper management of NCDOT quickly decided to utilize in-house designers to determine solutions for the repair of the failed slopes and open the lanes of I-40 to the traveling public as soon as possible. The decision was made to complete the design and prepare the contract documents for Slide #1 (slide near tunnel) and Slide #2 (large slide) in one contract and handle the other slides soon after. The NCDOT Highway Design Branch immediately established a timeline to have plans prepared for our Division office for letting. Figure 1 shows the aggressive 3-week schedule set up to develop and prepare the design and contract documents.

Interstate 40 Milepost 3 & 4
Emergency Slide Project Schedule

DATE	ACTIVITY	STATUS
September 22, 2004	Location and Surveys Place Panels	Completed
September 22, 2004	Photogrammetry Flies Project	Completed
September 23, 2004	Highway Design Branch Meets to Determine Scope of Work	Completed
September 24, 2004	Location and Surveys to Provide Base Line Surveys and Panel Controls to Photogrammetry	Completed
September 24, 2004	Geotechnical Engineering to Start Borings on I-40 Roadway Level	Completed
September 24, 2004	Division to Obtain Permit for Lower Borings in River	Completed
September 28, 2004	Photogrammetry to Provide Shell Mapping and Preliminary DTM to Roadway Design	Completed
September 29, 2004	Location and Surveys to Provide Obscured Areas for DTMs to Photogrammetry	Completed
October 1, 2004 (A.M.)	Roadway Design to Provide Preliminary Plans and Cross Sections to Geotechnical Engineering, Hydraulics, Structure Design and Division	Completed
October 1, 2004 (P.M.)	Photogrammetry to Provide Final Surveys and DTMs to Roadway Design	Completed
October 4, 2004	Roadway Design to Provide Final Plans and Cross Sections to Geotechnical Engineering, Hydraulics, Structure Design and Division	Completed
October 4-8, 2004	Roadway Design, Geotechnical Engineering, Structure Design, Hydraulics, and Division Coordinate on Final Design	Completed
October 13, 2004	All Plans are Turned in to Roadway Design	Completed
October 14, 2004	Roadway Design to Submit Plans to Division for Letting	Completed

FIGURE 1. Design Schedule for I-40 Slides at Site #1 and #2

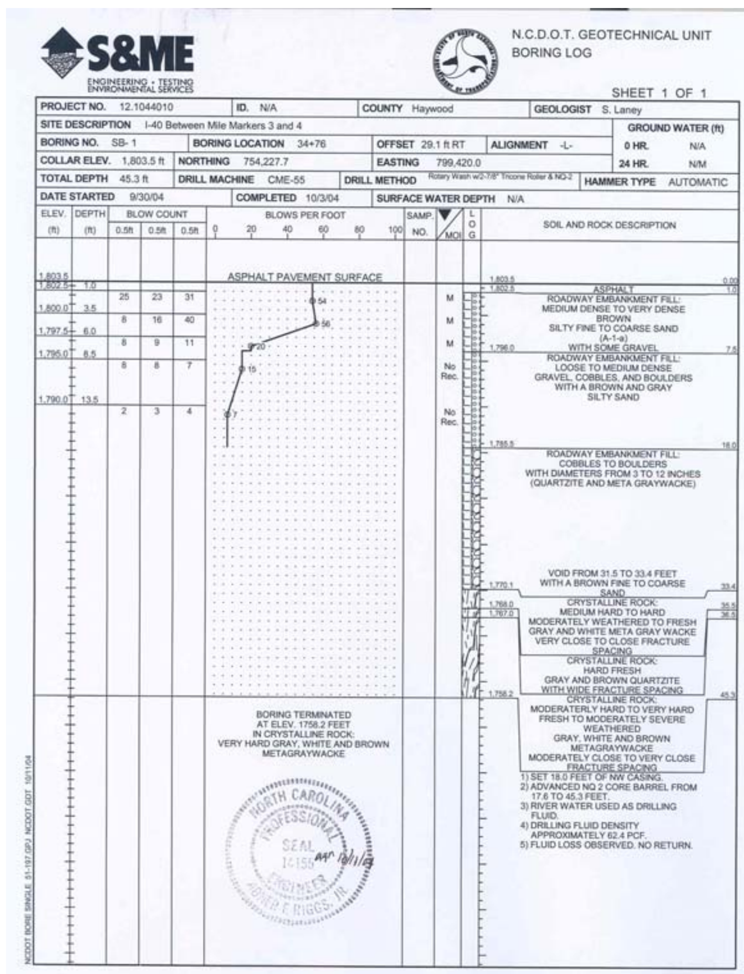
SUBSURFACE CONDITIONS

From a geotechnical standpoint, obtaining the existing subsurface conditions was a crucial priority for us to develop our designs for the repair of these slopes. The Consultant Coordination group within the NCDOT Geotechnical Engineering Unit responded to this emergency by acquiring the investigation services of our Contract drilling firms to obtain subsurface information. The following firms assisted us at the slide locations listed with the number of borings drilled.

Slide#1: 7 borings,	2 firms	Trigon, S&ME
Slide#2: 13 borings,	3 firms	F&H, Trigon, S&ME
Slide#3: 7 borings,	3 firms	MACTEC, Trigon, S&ME
Slide#4: 4 borings,	1 firm	MACTEC
Slide#5: 4 borings,	1 firm	MACTEC

In general, the existing subsurface under I-40 roadway embankment fill consisted of silty sands and some gravel underlain by dense gravel, cobbles, and boulders. Below this layer was mostly cobbles and boulders ranging in diameter from 3 to 12 inches made of quartzite and meta-graywacke rock fragments. Crystalline rock was encountered below the boulder layers and classified as meta-gray wacke. Figure 2 is an individual boring log from Slide #1.

From previous subsurface and roadway information, it appears that the I-40 roadway embankment was constructed of blasted rock from the adjacent mountainside along this corridor.



S&ME ENGINEERING • TESTING ENVIRONMENTAL SERVICES

N.C.D.O.T. GEOTECHNICAL UNIT
BORING LOG

SHEET 1 OF 1

PROJECT NO. 12.1044010 | ID. N/A | COUNTY Haywood | GEOLOGIST S. Laney
SITE DESCRIPTION I-40 Between Mile Markers 3 and 4
BORING NO. SB- 1 | BORING LOCATION 34+76 | OFFSET 29.1 ft RT | ALIGNMENT -L-
COLLAR ELEV. 1,803.5 ft | NORTHING 754,227.7 | EASTING 799,420.0
TOTAL DEPTH 45.3 ft | DRILL MACHINE CME-55 | DRILL METHOD Rotary Wash w/2-7/8" Tricone Roller & NQ-2 | HAMMER TYPE AUTOMATIC
DATE STARTED 9/30/04 | COMPLETED 10/3/04 | SURFACE WATER DEPTH N/A
GROUND WATER (ft): 0 HR. N/A; 24 HR. N/M

ELEV. (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	MOI
1,803.5						
1,802.5	1.0	25	23	31	54	M
1,800.0	3.5	8	16	40	56	M
1,797.5	6.0	8	9	11	20	M
1,795.0	8.5	8	8	7	15	No Rec.
1,790.0	13.5	2	3	4	7	No Rec.

ASPHALT PAVEMENT SURFACE

SOIL AND ROCK DESCRIPTION

1,803.5 — 0.00
1,802.5 ASPHALT 1.0
ROADWAY EMBANKMENT FILL: MEDIUM DENSE TO VERY DENSE BROWN SILTY FINE TO COARSE SAND (A-1-a) WITH SOME GRAVEL
1,796.0 — 7.5
ROADWAY EMBANKMENT FILL: LOOSE TO MEDIUM DENSE GRAVEL, COBBLES, AND BOULDERS WITH A BROWN AND GRAY SILTY SAND
1,785.5 — 18.0
ROADWAY EMBANKMENT FILL: COBBLES TO BOULDERS WITH DIAMETERS FROM 3 TO 12 INCHES (QUARTZITE AND META GRAYWACKE)
VOID FROM 31.5 TO 33.4 FEET WITH A BROWN FINE TO COARSE SAND
1,770.1 — 33.4
1,768.0 — 35.5
1,767.0 — 36.5
CRYSTALLINE ROCK: MEDIUM HARD TO HARD MODERATELY WEATHERED TO FRESH GRAY AND WHITE META GRAY WACKE VERY CLOSE TO CLOSE FRACTURE SPACING
CRYSTALLINE ROCK: HARD FRESH GRAY AND BROWN QUARTZITE WITH WIDE FRACTURE SPACING
1,758.2 — 45.3
CRYSTALLINE ROCK: MODERATERLY HARD TO VERY HARD FRESH TO MODERATELY SEVERE WEATHERED GRAY, WHITE AND BROWN METAGRAYWACKE MODERATELY CLOSE TO VERY CLOSE FRACTURE SPACING

1) SET 18.0 FEET OF NW CASING.
2) ADVANCED NQ 2 CORE BARREL FROM 17.6 TO 45.3 FEET.
3) RIVER WATER USED AS DRILLING FLUID.
4) DRILLING FLUID DENSITY APPROXIMATELY 62.4 PCF.
5) FLUID LOSS OBSERVED. NO RETURN.

BORING TERMINATED AT ELEV. 1758.2 FEET IN CRYSTALLINE ROCK: VERY HARD GRAY, WHITE AND BROWN METAGRAYWACKE

NORTH CAROLINA PROFESSIONAL SEAL 14155 ENGINEER ABNER F. RIGGS, JR.

NCDOT BORE SINGLE 51-197.GPJ NCDOT.GDT 10/11/04

FIGURE 2. Soil Boring from Site #1 (Slide near the Tunnel)

DESIGN PROCESS

The NCDOT Highway Design Branch had to coordinate amongst various disciplines to produce the necessary designs required to develop the concepts and solutions to repair the slope failures at Slide #1 and Slide #2. The Units involved for this emergency repair included Roadway Design, Structure Design, Photogrammetry, Locations & Surveys, Hydraulics, Project Services, and the Geotechnical Engineering Unit (GEU). From the aggressive 3-week schedule established by the Branch Manager, every Unit had a time critical role to play in this project delivery.

Since this was predominantly a geotechnical-related issue, the Geotechnical Engineering Unit was instrumental in developing the appropriate designs. Our Unit had to not only coordinate with our external colleagues but also with our internal personnel. Resources were utilized from every office within our Unit to complete our portion of the tasks.

The first initiative for this emergency project was to define the design and construction criteria. The major criteria items are listed in highest priority order:

1) Safety	**- Stabilize slope, protect construction personnel**
2) No Maintenance	**- Design and build a permanent, durable system**
3) Open I-40 lanes ASAP	**- Our main customers, the traveling public**
4) Non-rigid system at toe	**- Flexible system that can withstand water and rocks**
5) Prevent future toe scour	**- Protect the toe from future undermining**
6) Environmental concerns	**- Minimize any impacts to the Pigeon River and area**

Now that the criterion was established, the design teams could proceed with some clear direction and focus towards an appropriate repair. The greatest challenge was to determine a system that would satisfy all the criteria and also be constructable by our understanding of conventional means. Our goal was to use the “best” retention system available to stabilize the landslide and re-open the lanes of I-40. The GEU Manager assigned a design team to develop the retention system at the top of the slope and another team to develop a system to protect the toe.

After brainstorming numerous ideas and concepts, both teams had determined a conceptual plan for both systems. For the top of the slope, a tieback-anchored wall system was chosen to rebuild and retain the I-40 roadway and to stabilize the failed slope. For the toe, an innovative design was contemplated consisting of using rockfall catchment fence to wrap large boulders into a net that was anchored into the parent bedrock in the slope and riverbed. Figure 3 illustrates in detail our initial design concept in graphical format.

Tieback walls most commonly are used in cut situations and consist of steel soldier piles that are anchored with cement grouted, prestressed tendons or bars that are installed into soil and/or rock. Timber lagging is used to retain the soil between the soldier beams temporarily until the excavation has reached full height and the finished cast-in-place concrete facing is poured. This retention system was chosen for its redundancy and suitability in this application.

The toe scour protection was an innovative concept developed by the Structure Design Unit and Geotechnical Engineering Unit of the NCDOT. This concept is basically a magnified and expanded version of a typical wrapped-face fabric wall. Instead of using fabric, this design incorporates the state-of-the-art ring nets used typically for rockfall catchment fences and instead of using select backfill, two-foot to four-foot diameter boulders are used for encapsulation. The idea was to protect the toe from future large floods and toe scour. Also, the system is flexible and can withstand impact from other large boulders carried by the waters during a major flood event.

After the design concepts were finalized, all the Units had to work together to compile this information into contract documents for let.

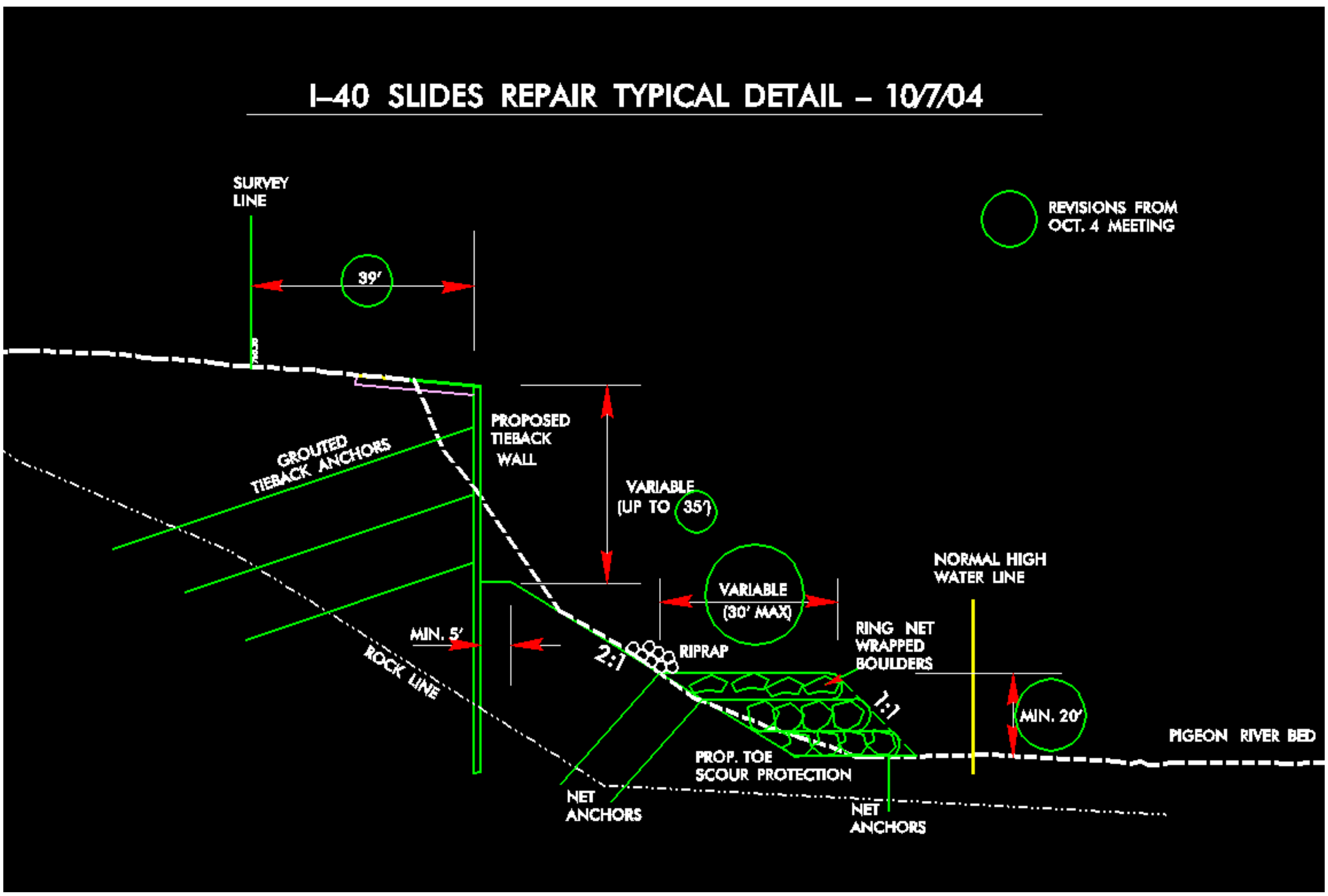


FIGURE 3. Conceptual Design for the repair at top and toe of slope

ALTERNATIVE CONTRACTING METHOD

Since the repair of the upper portion of the failed slope required a retention system, a specialty contractor experienced in the field of geotechnical construction was definitely required. With this in mind, the NCDOT had to obtain the services of a Geotechnical contractor in the contracting method. The traditional low bid selection process may not have obtained the most experienced company so an alternative method of contract delivery was introduced.

Our Unit invited a few of the Geotechnical Contractors experienced in major highway, retaining wall, and/or marine construction to present their proposals for the repair of these slides. The selection of the firm was based on a grading criteria related to the established design and construction criteria. The geotechnical contractors were Brayman Construction Corporation, Haywood Baker, Richard Goettle, Inc., and Schnabel Foundation Company.

The Grading criteria was set up as a Quality Credit scoring system as follows:

Safety Plan	30 points
Schedules and Milestones	25 points
Long Term Maintenance	20 points
Innovation	10 points
Environmental Stewardship	10 points
Oral Interview	5 points

RESULTS OF WALL SUBCONTRACTOR PRICE PROPOSAL OPENING

ENGINEER'S ESTIMATE = $2,275,000.00

QUALITY ADJUSTED PRICE RANKING (I-40 Emergency Project/Haywood Co.)

Vendor	Technical Score	Quality Credit (%)	Price Proposal ($)	Quality Value ($)	Adjusted Price ($)
BRAYMAN CONST.	97	54.00	$4,062,450.00	$2,193,723.00	$1,868,727.00
HAYWARD BAKER	70	0.00	$2,174,468.50	$0.00	$2,174,468.50
RICHARD GOETTLE,INC	86	32.00	$4,695,000.00	$1,502,400.00	$3,192,600.00
SCHNABEL FOUNDATION COMPANY	77	14.00	$2,160,000.00	$302,400.00	$1,857,600.00

FIGURE 4. Geotechnical Contractor Bid Comparisons

The selection of the contractors was based on an innovative two-step process. Step one was the selection of the geotechnical subcontractor and Step two was selection of the Prime or General Contractor.

Schnabel Foundation Company had the lowest adjusted price and was awarded as the Geotechnical subcontractor for this project.

The General Contractor was selected based on the typical lowest bidder method. Phillips & Jordan, Inc. was awarded as the GC and had to work with the already selected geotechnical subcontractor.

CONSTRUCTION

The contract the emergency repair of Slide #1 and #2 was awarded in October 2004 and the available date was November 1st, 2004.

Schnabel Foundation Company had proposed a variation of the most commonly used tieback retaining wall system for this slope stabilization. Schnabel proposed an anchored tieback wall with steel circular pipe piles as the soldier beams and using ground anchors through concrete walers across the pipe piles. Also, in lieu of a cast-in-place concrete face, Schnabel proposed a shotcrete facing on the retaining wall to accelerate the construction process.

Phillips & Jordan, Inc. began construction of the toe scour protection as soon as Schnabel had installed all the soldier beams and the first row of tiebacks.

All lanes of traffic on I-40 were opened to the traveling public on February 25, 2005. On the weekends, all lanes will remain open. During the weekdays, one eastbound lane of I-40 is temporarily closed to facilitate construction of the toe scour system. The estimated final completion date for Slides #1 and #2 is June 1st, 2005.

Phillips & Jordan, Inc. through a supplemental agreement with NCDOT Division office are repairing Slide #3.

Tentatively, repair of Slides #4 and #5 will be advertised in the June 2005 letting.

REFERENCES

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2. NC Weather Review, 2004. *Precipitation data for August and September 2004.*
3. Hering, D., 2005. *Soil boring by S&ME from Site #1*.

I-40 TOE SCOUR PROTECTION SYSTEM

BY

JOSEPH BIGGER
REGIONAL MANAGER EASTERN NORTH AMERICA
GEOBRUGG NORTH AMERICA, LLC.
NEW LONDON, CONNECTICUT

ABSTRACT

Heavy rains from Hurricane Ivan in September 2004 caused stream and river levels to rise resulting in flooding, mudslides and stream bank scour. The water level in the Pigeon River rose significantly and out of its channel to severely scour the I-40 embankment at approximately milepost 3. The extent of the scour damage included the loss of the East bound lanes and severely restricted traffic flow.

The North Carolina Department of Transportation in response to the emergency situation quickly developed plans to stabilize the embankment, rebuild the lanes and prevent further toe scour. Retaining walls were designed and built to stabilize the embankment at the road level and allow the reconstruction of the lanes.

A Toe Scour Protection System is a rock embankment designed by the Department of Transportation to prevent further scour. It is placed at the edge of the river and extends over 1,000 feet along it. The system is a rock embankment that incorporates Geobrugg's ROCCO ring nets as layer reinforcement and to cover the rock face. The system has 4 layers of large diameter rock and boulders plus riprap and the layer thickness is 4 feet. The face is set at a 1:1 slope. The installation has experienced problems ranging from the tension bolts to the handling and placement of the nets. The installation is near completion and the success of the design and installation will not be known until heavy rains occur.

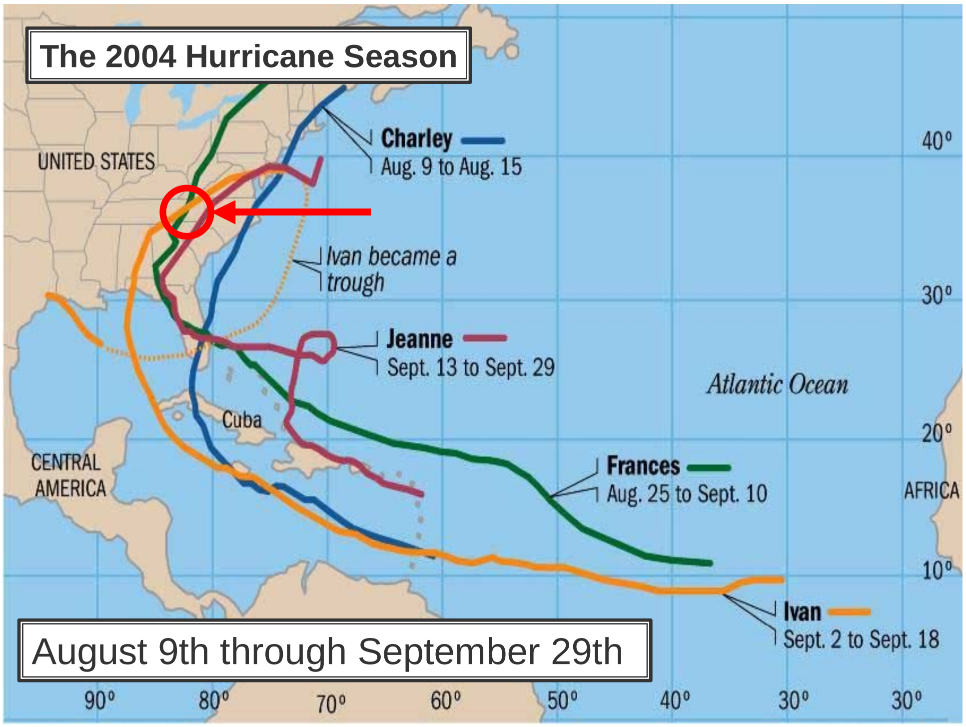
INTRODUCTION

Interstate highway I-40 is a major artery through the Smokey Mountains in Western North Carolina and Eastern Tennessee for the movement of goods and people. Without it, the route through the mountains is along 2 lane roads and the trip takes much more time. When I-40 was closed several years ago by a landslide near the North Carolina/Tennessee state line, the need to keep it open became obvious because of its' impact on commerce.

The highway follows the Pigeon River and passes through a tunnel at milepost 4 before crossing into Tennessee. At the tunnel, the river goes around the mountain it and rejoins the highway.

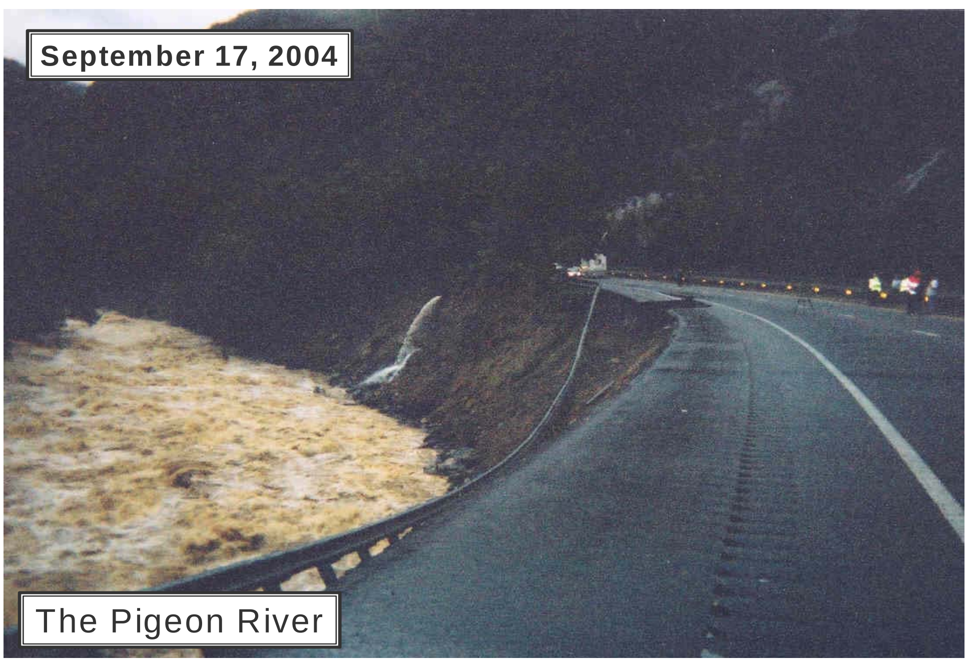
PROBLEM

The 2004 Hurricane Season was an unusual year for weather in Florida and North Carolina. Hurricanes Charley, Ivan, Francis and Jeanne hit both states causing wide stead damage. Two of the hurricanes, Francis and Ivan, crossed over Western North Carolina and left behind inches of rain between August 9th and September 29$^{th.}$



The rain inundated the region and, in turn, saturated the soil and filled all the small steams. The saturated soil conditions lead to slope failures and mudslides throughout the region. The steams eventually overflowed and flooded property and homes. The steams eventually drained into the Pigeon River causing the water level in it to rise out of its' channel. All this water came down the river in a large torrent and, unfortunately, the river curved against the highway at the West portal of the tunnel. It was at this location where the embankment was severally eroded and scoured away.

VIEW LOOKING UPSTREAM



VIEW LOOKING DOWNSTREAM

RESULTS

The resulting scour and erosion caused damage at 5 locations along I-40 with the worst damage at Slide 1 and 2 to the extent I-40 was reduced to 2 lanes.

I-40 EASTBOUND LANE AFTERWARDS

The erosion and scour of the embankments at the other 3 locations did not damage I-40 but did require some type of mitigation against any future problems. The result was the closing of the eastbound lanes of I-40 at slide areas 1 & 2 until the embankment could be stabilized and allow rebuilding the eastbound lanes.

THE SOLUTION

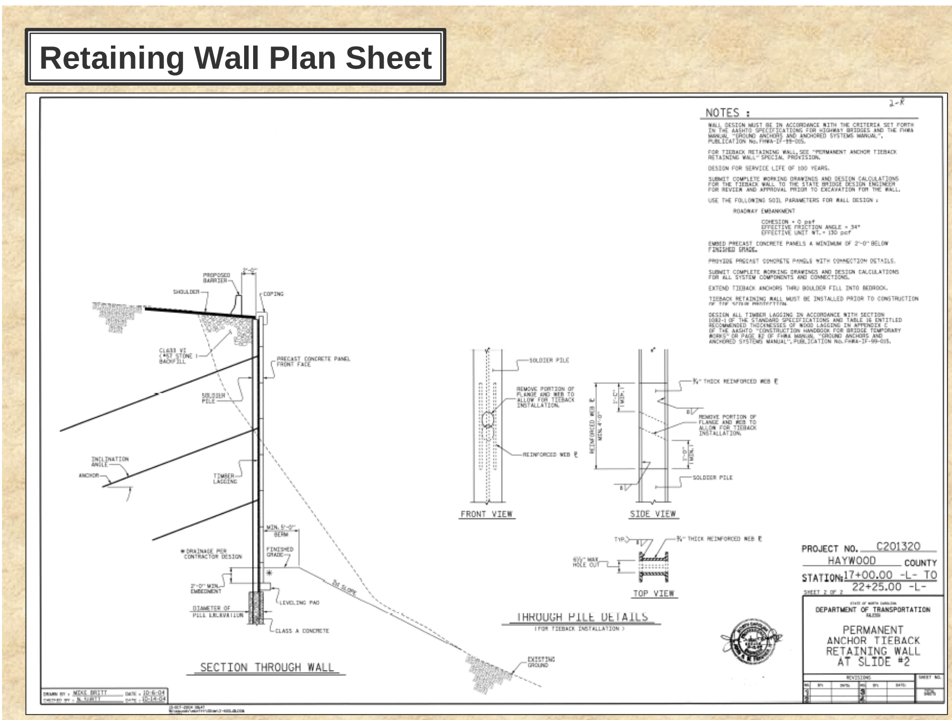
After the water subsided and the extent of the damaged was determined, the next step was to develop a solution. The North Carolina Department of Transportation also decided to have their Highway Design Branch develop a solution based on the following criteria:

1) Safety - Stabilize slope
2) No Maintenance
3) Open I-40 lanes ASAP
4) Non-rigid system at toe
5) Prevent future toe scour
6) Environmental concerns

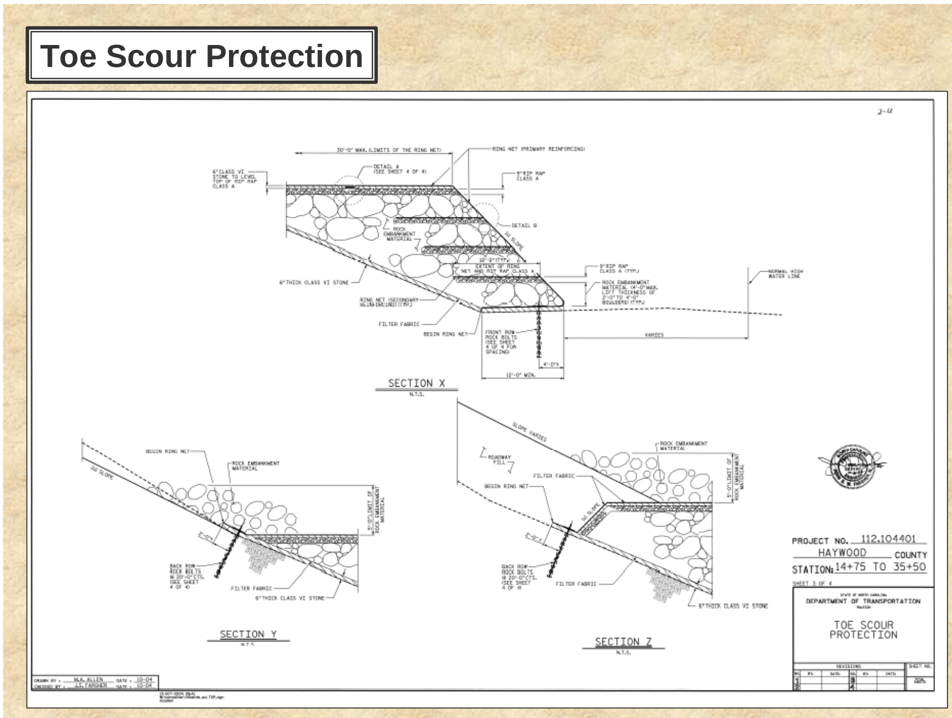
The Highway Design Branch was tasked with developing a solution as quickly as possible for slides area 1 and 2 and they had a final design completed including contract plans by October 14, 2004. The contract plans were issued to 4-selected geotechnical and 4 general contractors who were invited to bid on the work.

The solution was divided into 2 parts – (1) a retaining wall so the roadway could be rebuilt and reopened and (2) a toe scour system to protection the slope from future problems. The retaining wall, shown below, is the subject of another paper. The wall was required only for slide areas 1 & 2 in order to reopen the highway. The toe scour protection system specified for slide areas 1 & 2 was added to slide area 3.

The geotechnical contractors were asked to submit bids to install the wall and this was separate from the other work. After a geotechnical contractor was selected, the general contractors were asked to submit bids for the balance of the work. Schnabel Foundation Company was selected for the walls and Phillips & Jordan was selected as the general contractor.



The Highway Design Branch considered several alternatives for the toe scour system and the selected final design met the criteria of being non-rigid and able to prevent future problems from erosion and scour. The design is essentially a reinforced rock slope using rings nets in lieu of fabric and large diameter riprap as shown in the following drawing.



The system has 4 layers of larger diameter rock with a layer thickness of 4 feet and incorporates rock that is 2 feet to 4 feet in diameter. The nets are George North America's ring nets made with 11.8-inch nominal diameter interlocking rings and the rings are made with 3-millimeter diameter galvanized high strength alloy steel wire. The wire is coiled into rings and each ring has 12 coils of wire.

Before placing the large diameter rock on top of the ring nets, a 9-inch layer of riprap was placed over them. The face of the Toe Scour Protection system was lay back at a 1:1 slope with ring nets covering it. The layers on ring nets extend 12 feet from the face into the slope. The top layer runs back 30 feet and is pinned using rock bolts per the original design. Rock bolts were installed along the outside edge of the bottom layer of ring nets to hold everything in place.

Slide Area 3 was added to Phillips & Jordan's contract and the design was modified. The modified design called for placement of concrete barriers on top of the ring nets, drill through the barriers, install the upper anchors and pin the nets under the barriers.

The ring nets are attached together using 1/2-inch screw pin anchor shackles instead of the TECCO Compression Claw shown on the plans.

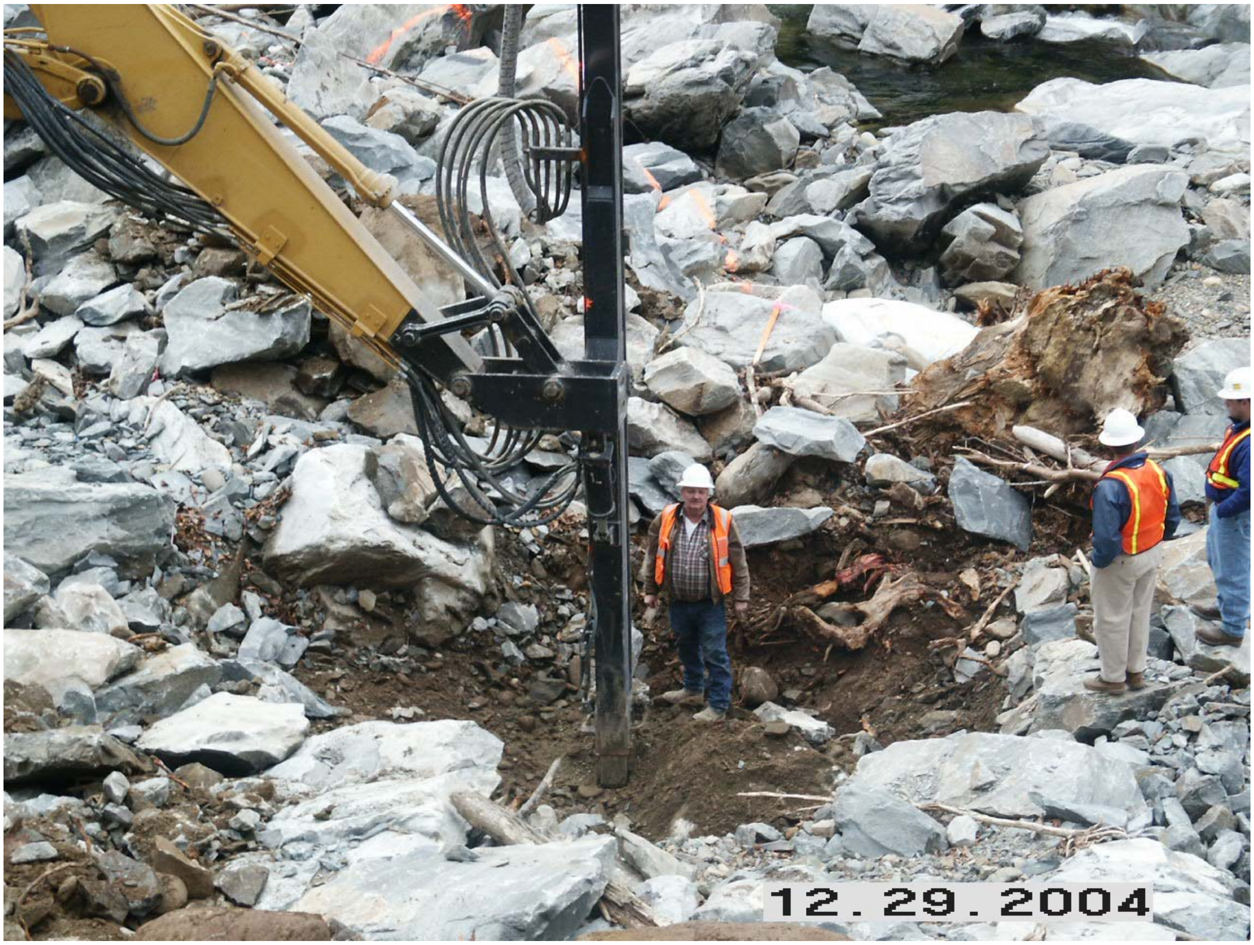
VIEW OF CONCRETE BARRIER WITH RING NET WRAPPED AROUND IT

INSTALLATION

With the design complete and a contractor selected, the next step was to install the Toe Scour Protection System. This could not happen immediately at slide areas 1 & 2 as the retaining walls were to be completed first. Slide area 3 was added to the contractor's scope of work after the award of the first 2 areas and it did not have a wall but the slope had to be cleared and cut to a 1:1 slope.

The first step was to install the rock bolts along the edge of the river and the contract plans called for the bolts to be tensioned. This proved to be a problem. After drilling 3 to 4 feet into rock, a layer, up to 20 feet thick, of sand was hit. The Highway Design Branch reviewed the problem and the design. The revised design called for un-tensioned rock bolts drilled only into the top layer of rock. However, the straight line of bolts was adjusted to allow hitting rock.

Once the bolts were in place, the bottom layer of rings nets could be placed and held down by steel plates.



VIEW OF DRILLING FOR ROCK BOLTS ALONG EDGE OF PIGEON RIVER

The next problem encountered by Phillips and Jordan was how to handle the 18-foot by 30-foot and 10-foot by 15-foot ring nets. The nets are not rigid and will collapse when hoisted off the ground. The problem was solved by using spreader bar at the top to hoist the nets plus intermediate spreaders to hold the nets open. Once this was done, the work proceeded smoothly.

VIEW OF RING NET BEING POSITIONED

VIEW OF PARTIALLY COMPLETED TOE SCOUR SYSTEM AT SLIDE AREA 2

CONCLUSION

The installation of the toe scour protection system at slide areas 1, 2 & 3 is continuing and should be completed by the end of May 2005. The work for toe scour protection systems for slide areas 4 & 5 will be let in May 2005.

The Department is confident the toe scour protection systems will protection the slopes along the Pigeon River and prevent the same problems from occurring in the event of high water in the river. Of course, no one hopes to have a repeat the rains from last year and all the related problems.

ACKNOWLEDGEMENTS

The North Carolina Department of Transportation's Geotechnical Unit at the Highway Design Branch for their assistance and information.

Phillips & Jordan for their assistance and providing site photographs.

Assessing the Potential Environmental Impact of Acid Rock Drainage (ARD) and Metal Leaching (ML) for the Sea to Sky Highway Improvement Project Between Vancouver and Whistler, British Columbia

Stephen Barrett, Rens Verburg, Valerie Bertrand, Cheryl Ross, Jeff Fillipone and Dave Munday[1]

ABSTRACT

This paper discusses the study completed as part of the Environmental Impact Assessment to assess the Acid Rock Drainage (ARD) and Metal Leaching (ML) potential of proposed new rock cuts that will be constructed as part of the Sea to Sky Highway Improvement Project. The Sea to Sky Highway, that extends from Vancouver to Whistler, British Columbia, is currently being upgraded in preparation for the 2010 Winter Olympic Games. An ARD/ML assessment was required to evaluate potential environmental effects to freshwater and marine environments from rock cuts and waste rock generated during the highway upgrade. The paper presents the approach to and findings of the ML/ARD assessment, and concludes by describing key factors for consideration when planning such studies, as well as some options available to mitigate potential impacts.

1.0 PROJECT BACKGROUND

The British Columbia Ministry of Transportation (MoT) is in the final stages of negotiating a Design Build Finance Operate (DBFO) Contract for upgrading of the Highway 99 corridor from Horseshoe Bay to Whistler. The project involves selective widening of the existing highway to 4-lane and 3-lane sections and construction of safety upgrades throughout the remainder of the corridor.

As part of the environmental review process, a study was undertaken by Golder Associates Ltd. to determine the potential environmental effects from ARD and ML generated from the new rock cut faces and waste rock generated during excavation. This involved characterizing the rock over a 44 km section from Horseshoe Bay to Squamish (Preliminary Alignment (PA) Sections 1 to 8) and a 23 km section from Cheakamus Canyon to Function Junction in Whistler (PA Sections 14 to 16) where the new rock cuts were proposed (Figure 1).

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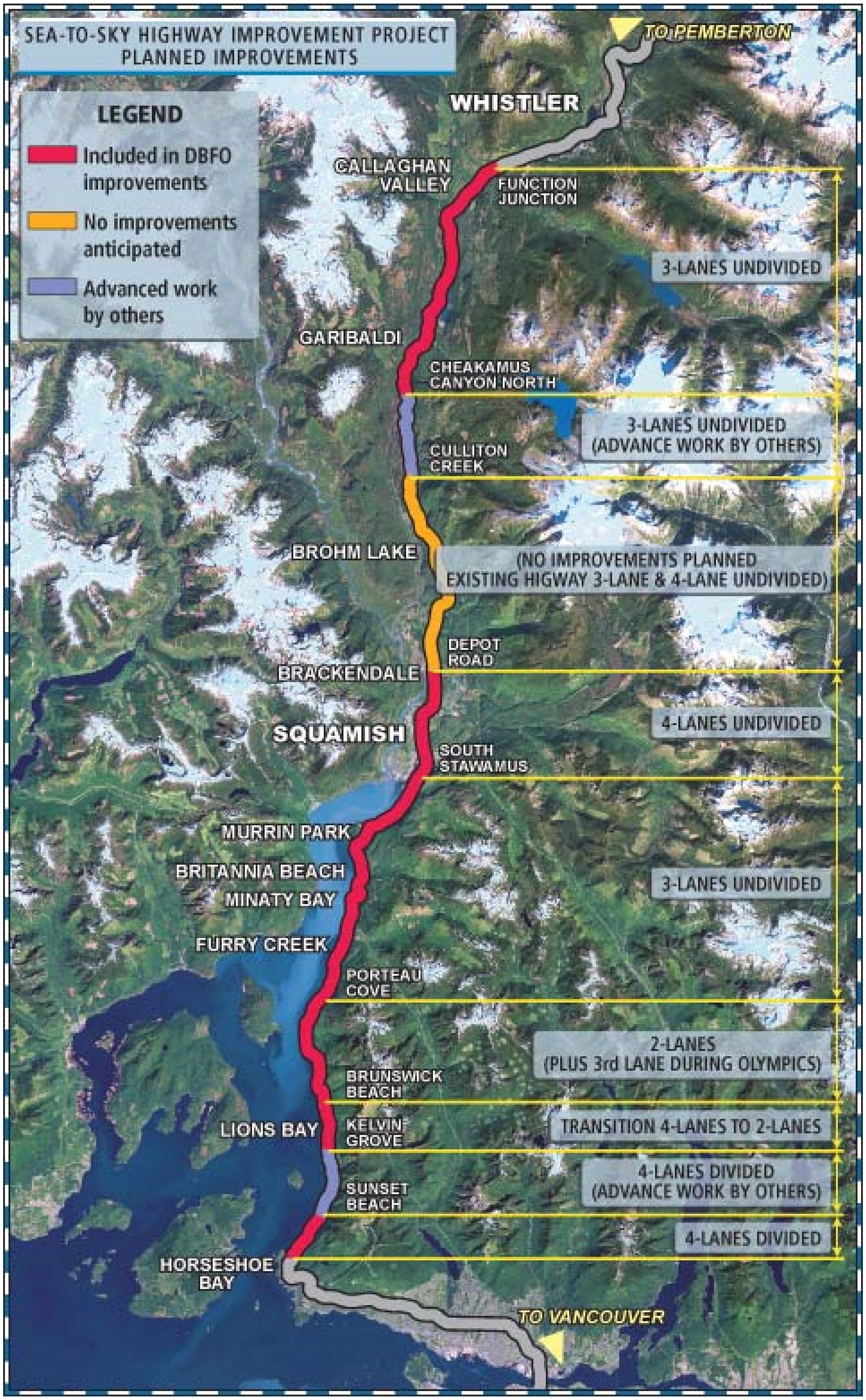


Figure 1 - Project Location Plan

Preliminary design for the project proposed a total of approximately 70 rock cuts that would produce approximately 1.8 million m^3 of excavated rock (neat line – no swell factor applied). Of the total rock excavated, 250,000 m^3 was estimated to be surplus, although the local imbalances at either end of the project were much larger (SNC, 2003).

The objectives of the study were 1) to characterize the material to be excavated with respect to its ARD/ML potential; 2) to identify environmental effects based on the assessed ARD/ML potential; and 3) to determine suitable re-use or disposal options for the excavated material. This included assessing the potential for re-use of the non-acid generating material as manufactured aggregate products for sale to interested third parties.

2.0 ARD / ML EFFECTS ON AQUATIC ECOSYSTEMS

Metal loading to the environment can adversely affect aquatic ecosystems. Although metal leaching may occur under neutral pH conditions, because the mobility of many metals increases as pH decreases, acidic conditions generally enhance metal leaching. Generation of ARD occurs when reactive sulfides such as pyrite, a common iron sulphide, are exposed to water and oxygen, resulting in formation of a solution that is generally characterized by a low pH, elevated dissolved metal concentrations, and total dissolved solids, with sulfate being the principal contributor. Runoff from rock cuts where sulfide oxidation has occurred may result in metal loading (e.g., iron (Fe), aluminum (Al), copper (Cu) and zinc (Zn)) to freshwater and marine environments. The chemical and mineralogical characteristics of the rocks will determine which metals and the quantity of metals that will be released, while the characteristics of the receiving environment will control the mobility of the metals and other constituents.

In freshwater and marine environments, exposure of aquatic organisms, such as fish and benthic invertebrates, to metals may occur through a variety of pathways including dermal uptake, absorption though the gill membrane, and ingestion of water and/or prey.

Specific examples of potential adverse environmental effects include, but are not necessarily limited to:

- Toxic interactions from reduced pH and increased metal concentrations;
- Sediment toxicity where trace metal concentrations are high; and
- Reduced habitat availability caused by precipitate settling.

Adverse impacts to organisms may range from acute toxicity to bioaccumulation in the food chain to behavioural or reproductive effects. Degradation of the aquatic environment has the potential to result in reduced species richness and abundance and/or a shift from pollution-sensitive to pollution-tolerant species.

Typically ARD / ML issues are identified at mine sites, but they have also been identified on some civil engineering infrastructure projects, such as the access road to Halifax Airport. If the potential problem is not identified early on in the design process so it can be mitigated, it can become very expensive to control after the project has been constructed.

3.0 MATERIAL CHARACTERIZATION

3.1 FIELD MAPPING PROGRAM

The first step in the ARD / ML assessment was to develop a geologic model, which identified the number, type and location of the significant lithological units present along the project corridor (Figure 2). The model was based on existing geological mapping, supplemented by a limited field program to verify the location of geologic contacts, lithologic type and mineralogy, as well as to document alteration/mineralization, rock texture or structure relevant to an ARD / ML assessment. Once completed, the geologic model was superimposed over the preliminary highway designs to select sampling locations for the laboratory testing program.

The final model consisted of a relatively simple geologic framework consisting of three overall lithologic packages from Horseshoe Bay to Squamish and eight individual lithologies from Cheakamus Canyon to Function Junction. The lithological groupings were as follows:

Horseshoe Bay to Squamish (PA Sections 1 to 8)

1) *Twin Island Group* ($_{P}J_{T}$): Metamorphic rocks of the pre-Jurassic Twin Island Group
2) *Coast Plutonic Complex* (CPC): Intrusive, granitic rocks of the Mesozoic Coast Plutonic Complex, consisting of late Jurasssic granodiorite (gd_2), Squamish Pluton granodiorite (gd_1), granite (g) and quartz diorite (qd); and
3) *Gambier Group* (IK_G): Marine sedimentary and volcanic rocks of the lower Cretaceous Gambier Group.

Cheakamus Canyon to Function Junction (PA Sections 14 to 16)

1) *Garibaldi Group* (PRG): Basalt flows
2) *Gambier Group* (KG_v): Dacite and andesite flows.
3) Metadiorite (JKdi): Chloritized, locally foliated diorite.
4) Slollicum Schist (MS1): Chlorite, pyllite and slate
5) Quartz Diorite (m/Jqd): Quartz diorite and minor granodiorite
6) Greenstone (TrJgs): Metavolcanics and schists
7) Gneiss (gn): Quartzofeldspathic gneiss
8) Granodiorite (Kgd): Cretaceous Granodiorite

3.2 GEOCHEMICAL CHARACTERIZATION PROGRAM

The geochemical characterization program was designed to provide sufficient information to assign a bulk ARD / ML potential to each lithology to be excavated. This evaluation included sampling and geochemical characterization of rock samples collected along the alignment. At select sampling locations, field-scale leach testing (i.e., wall washing) was conducted for comparison to the results of lab-scale leaching tests. Water quality sampling of creeks was conducted to establish background conditions and to assess the environmental impacts of existing rock cuts.

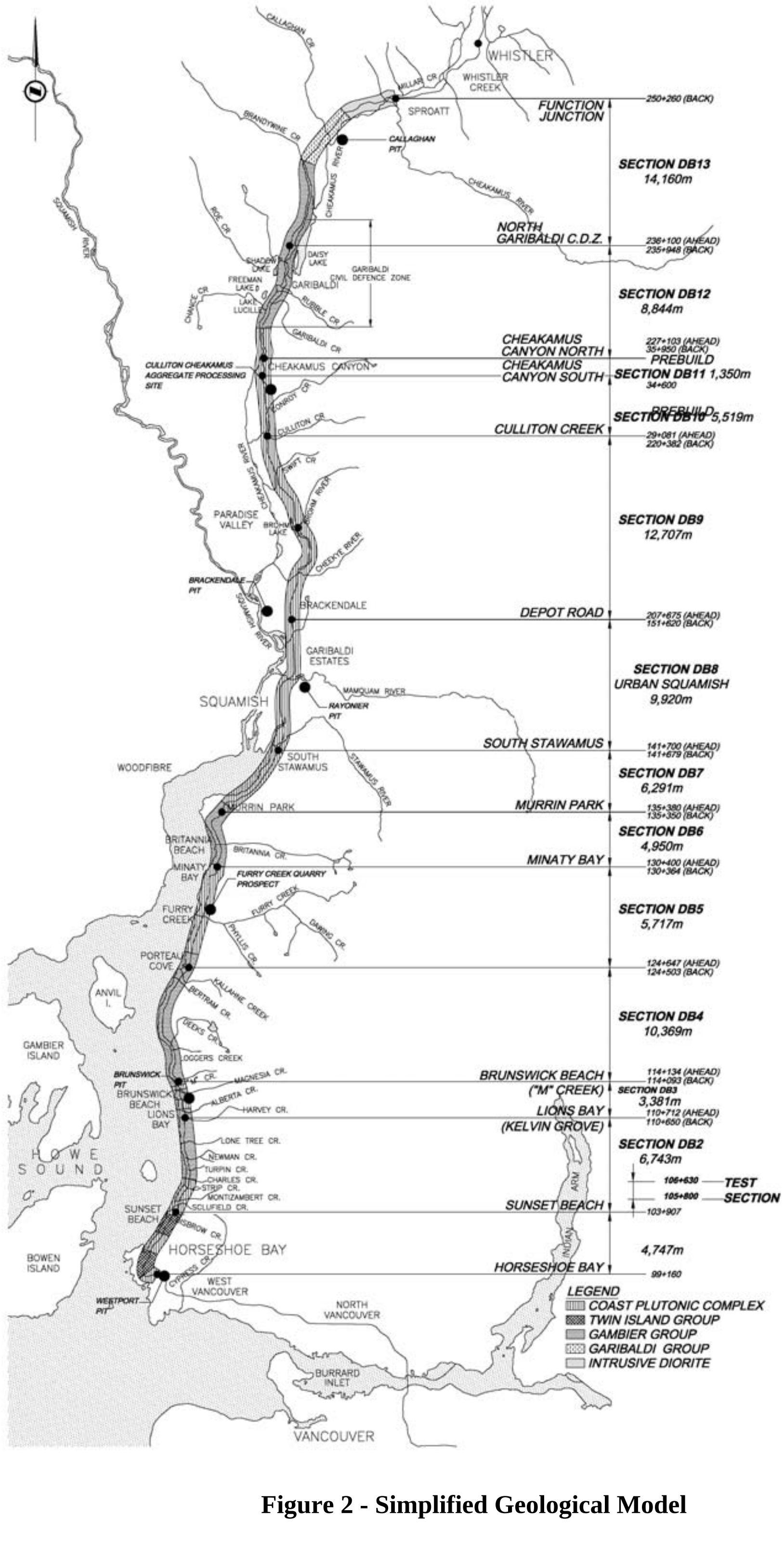


Figure 2 - Simplified Geological Model

Rock samples were collected for geochemical characterization. The sampling density was sufficient to provide a screening level of study, with an understanding that more detailed testing would be completed during detailed design, following finalization of rock cut locations. At each sample location, a 5-kg to 8-kg composite sample was collected from the rock face over a linear distance of two to three meters. Individual samples were confined within one lithology or to a zone of distinct character within a lithology.

A total of 54 rock samples were tested in the laboratory, including four duplicates for evaluation of quality assurance/quality control (QA/QC). To characterize the ARD and ML potential of the samples, the following analyses were conducted:

- Acid Base Accounting (ABA) including the following analyses:
  - Acid Potential (AP) by sulphur speciation (total sulphur and sulphate sulphur) analysis;
  - Bulk Neutralization Potential (NP) by modified Sobek;
  - Carbonate Neutralization Potential (CaNP) by total inorganic carbon (TIC) analysis; and
  - Paste pH.
- Whole rock chemistry and mineralogical analysis; and
- Satic leach testing (i.e., Shake Flask Extraction (SFE)).

ABA and whole rock chemical analyses were conducted on all 54 samples. Ten samples were submitted for mineralogical analysis by X-ray diffraction (XRD), focusing on the identification of acid buffering minerals such as carbonates and acid generating minerals such as sulphides (pyrite). Representation of each of the major rock types was considered in sample selection. Sample selection was biased toward those samples with high sulfide contents or high total inorganic carbon, indicating the possible presence of carbonate minerals. This analysis was intended to determine the nature of neutralizing and acid generating minerals and further determine the amenability of the mineral assemblage in the rock to generate ARD.

Metal leaching potential was evaluated on a sub-set of 30 samples using SFE. The SFE test represents a standard, short-term, static leach test aimed at determining the readily-soluble component of a material. Sample selection considered spatial and volumetric representation of rock units along the highway alignment. SFE samples were crushed and split into coarse (2.8 to 9.5 mm) and fine (less than 2.8 mm) fractions and subjected to a 24-hour test using de-ionized water as the lixiviant. Testing of both a coarse and a fine fraction allowed evaluation of the effect of grain size on metal leaching. Test leachates were filtered and analyzed for dissolved metals.

At six sampling locations, field-scale leach testing (wall washing) was conducted for comparison with the laboratory testing results. The wall washing tests were conducted following the standard procedure outlined in Price, 1997. The wall washing tests were performed prior to the start of the Fall and Winter wet season on the west coast. The results of these tests were therefore considered to be representative of conditions that reflect extended weathering throughout the dry season. Generally, the “first flush” of the wet season results in peak or worst-case metal loading, as stored acidity and metals in secondary minerals that have accumulated on the rock face over the dry season are removed and collected in the rinsate.

Water quality samples were also collected from creeks and from Howe Sound to establish background water quality. Each water sample was analyzed for total and dissolved metals, major anions, hardness, total dissolved and total suspended solids (TDS and TSS), conductivity and pH.

An example of the drawings used to summarize the results of the field mapping and laboratory testing programs is shown in Figure 3. Information shown on these drawings included:

- Geologic units;
- Lithologic descriptions;
- Rock sample locations and identification numbers;
- Approximate proposed road cut limits;
- Indicators of ARD potential;
- Metal concentrations in SFE test leachates exceeding fresh water and marine aquatic life guidelines; and
- Surface water sample locations and identification numbers.

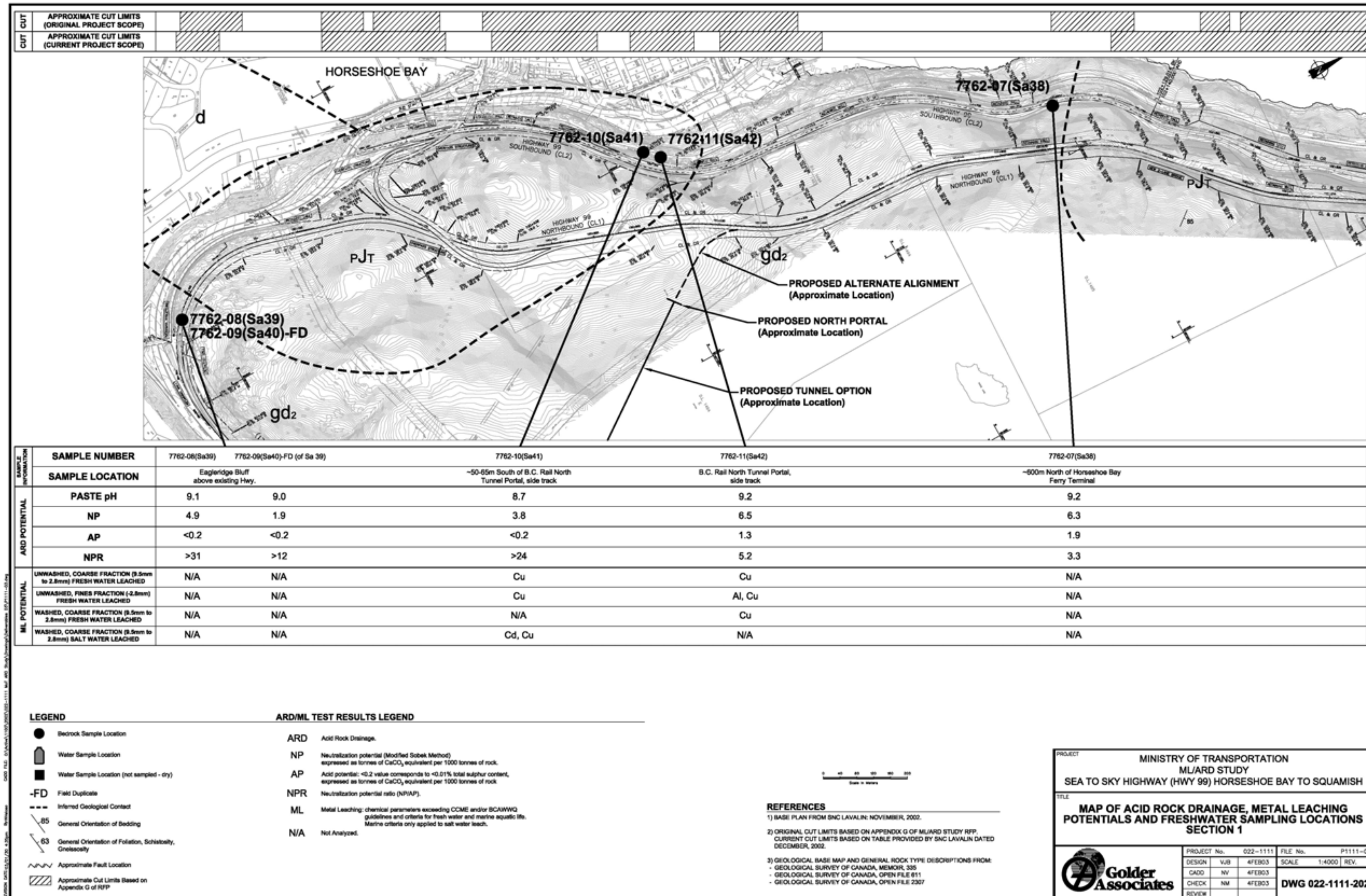
4.0 DETERMINATION OF POTENTIAL ENVIRONMENTAL EFFECTS

The approach developed for this study considered the potential effect of ARD/ML from final rock cut faces and from re-use or disposal of the excavated rock. A quantitative method was used to predict metal loading from rock cuts, while potential environmental effects from the re-use and disposal of excavated rock were discussed qualitatively and mitigation strategies were proposed to preclude or minimize environmental impacts.

The approach is represented graphically in the linkage diagram shown in Figure 4. This diagram provides a conceptual framework to describe and evaluate the potential environmental effects that the rock cut faces and the re-use and disposal options for the excavated materials could have on freshwater and marine aquatic life.

4.1 ARD POTENTIAL

As no regulations or guidelines currently exist with respect to the analysis of reactive rock material encountered during road construction, rock sample analyses follows those outlined for the mining industry in Price 1997. Following these procedures, the potential of a geologic material to generate ARD is evaluated by comparing the amount of neutralizing minerals expressed as neutralization potential (NP), to the amount of sulphide minerals expressed as the maximum acid potential (AP) present in the rock. This ratio is referred to as the Neutralization Potential Ratio (NPR). Price's suggested guidelines for interpretation of the NPR are presented in Table 1 below.



SAMPLE INFORMATION	SAMPLE NUMBER	7762-08(Sa39)	7762-09(Sa40)-FD (of Sa 39)	7762-10(Sa41)	7762-11(Sa42)	7762-07(Sa38)
	SAMPLE LOCATION	Eagleridge Bluff above existing Hwy.		~50-65m South of B.C. Rail North Tunnel Portal, side track	B.C. Rail North Tunnel Portal, side track	~600m North of Horseshoe Bay Ferry Terminal
ARD POTENTIAL	PASTE pH	9.1	9.0	8.7	9.2	9.2
	NP	4.9	1.9	3.8	6.5	6.3
	AP	<0.2	<0.2	<0.2	1.3	1.9
	NPR	>31	>12	>24	5.2	3.3
ML POTENTIAL	UNWASHED, COARSE FRACTION (9.5mm to 2.8mm) FRESH WATER LEACHED	N/A	N/A	Cu	Cu	N/A
	UNWASHED, FINES FRACTION (-2.8mm) FRESH WATER LEACHED	N/A	N/A	Cu	Al, Cu	N/A
	WASHED, COARSE FRACTION (9.5mm to 2.8mm) FRESH WATER LEACHED	N/A	N/A	N/A	Cu	N/A
	WASHED, COARSE FRACTION (9.5mm to 2.8mm) SALT WATER LEACHED	N/A	N/A	Cd, Cu	N/A	N/A

Figure 3 – Typical Map Showing ARD / ML Potentials For Rock Types Encountered Along Highway Alignment

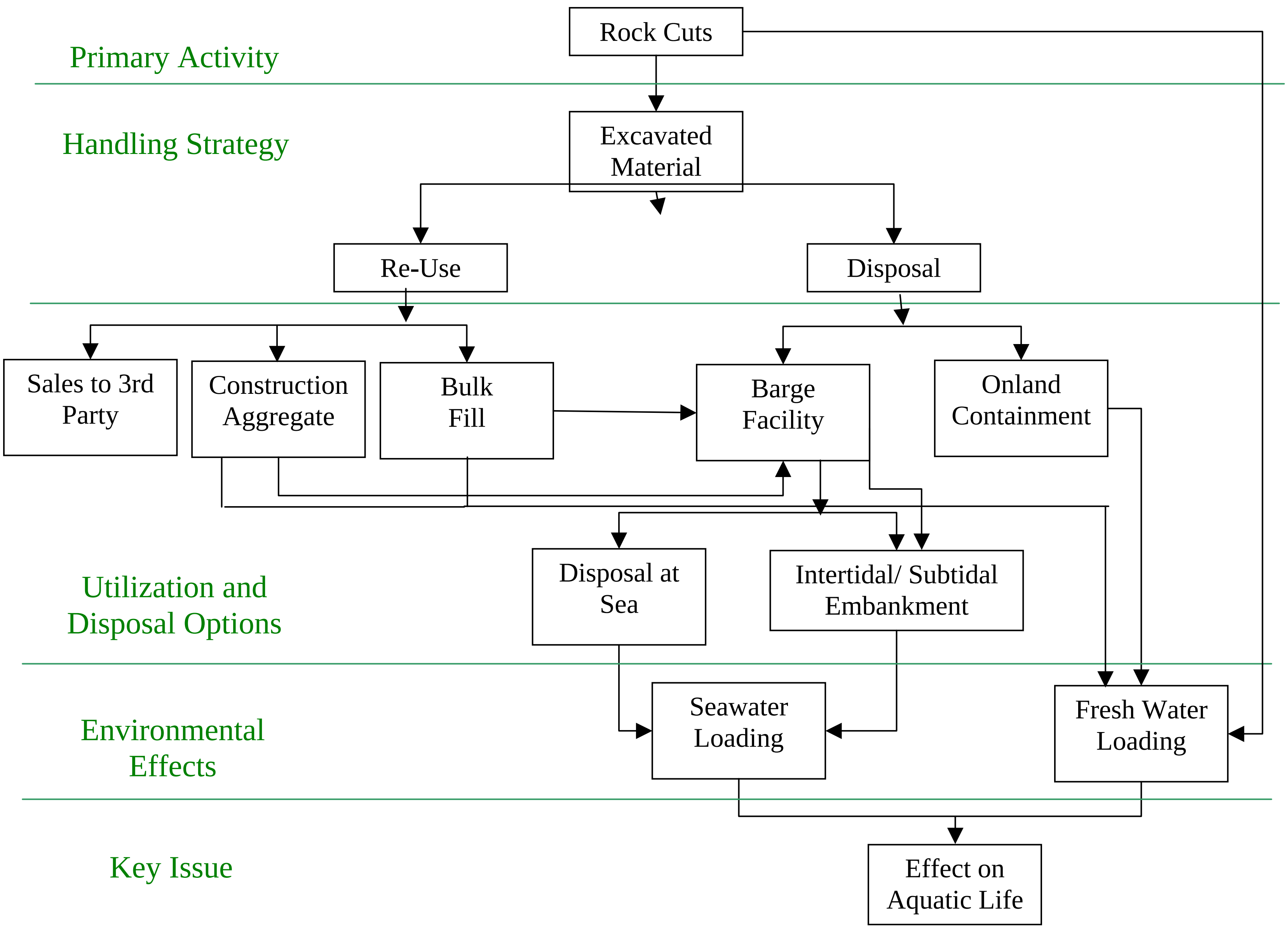


Figure 4 – ARD / ML Environmental Assessment Linkage Diagram

TABLE 1: ACID ROCK DRAINAGE SCREENING CRITERIA (PRICE 1997)

Potential for ARD	Initial Screening Criteria	Comments
Likely	NPR <1	Likely acid generating, unless sulphide minerals are non-reactive.
Possible (uncertain)	1<NPR<2	Possibly acid generating if NP is insufficiently reactive or is depleted at a rate faster than sulphides.
Low	2<NPR<4	Not potentially acid generating unless significant preferential exposure of sulphides along fractures planes, or extremely reactive sulphides in combination with insufficiently reactive NP.
None	NPR >4	

For the purposes of the assessment, rock having a NPR value above 4 was considered to be non-acid generating. All rock having a NPR less than 4 was considered as having the potential to generate ARD. For rocks with NPR values between 2 and 4, classification as potentially acid-generating (PAG) was considered conservative. After further analysis, these materials may not require special handling or disposal procedures depending on their potential to leach metals.

Price (1997) also presents a number of other criteria which may be applied to assess the ARD potential of a rock. While these criteria were not used as the primary screening tool in this study, they were used as secondary means to check the conclusions drawn from the NPR's.

The first criteria, evaluates acid generation potential using the sulphide sulphur content and the paste pH. Materials with a sulphide sulphur content less than 0.3 wt. % and a paste pH greater than 5.5 are considered non-potentially acid generating (NPAG) and require no further environmental testing. Exceptions occur where the rock matrix consists of base poor minerals (e.g., quartz), or where the sulphide minerals contain metals that may leach under weakly acidic to alkaline conditions. Rock characterized as having an NPR value below 2, and having a total sulphur content greater than 0.3 % is considered as PAG material under this criterion and consequently, would require additional investigation.

The second criteria, evaluates ARD potential by calculating the Net Neutralization Potential (NNP), which is the difference between AP and NP values (NNP = NP – AP). A negative NNP value is deemed to represent rock having a potential to generate acidic drainage, whereas a positive NNP reflects a likelihood that any acid generated by the rock will be neutralized.

4.2 ML POTENTIAL

Metal leaching rates (i.e., amount of metal per unit surface area (kg/m^2)) were calculated for each of the major rock types using SFE and wall washing test results. Leaching rates were calculated for aluminum and copper, as these were considered to be the metals most likely to leach at concentrations above water quality standards based on the results of the SFE tests. To quantify the potential environment effects due to ML loading from the final rock cut faces, the following additional work was undertaken:

1. Certain drainages were selected for detailed loading analysis from the proposed rock cuts. For each of these, the total surface area of all planned rock cuts within the basin was calculated. The total surface area of the rock cuts was multiplied by the ML potential to calculated the mass (kg) of aluminum and copper loading.

2. Hydrological analysis was undertaken to derive two scenarios:

   - The predicted typical summer low flow for selected creeks along the alignment. The 10 year, June to September, 7 day low flow in any stream was correlated with the contributing drainage area, based on calibration data from applicable nearby stream gauging stations; and

   - Resultant predicted peak stream flow corresponding to a typical fall "first flush" storm event. For the Horseshoe Bay to Squamish section of the study area, this analysis was based on a 4-day, 60 mm rainfall event recorded at the Britannia Beach / Furry Creek climate station from September 12-15, 1996, and extrapolated to other selected drainages. For the Cheakamus Canyon to Function Junction section of the study area, this analysis was based on a 2-day, 44 mm rainfall event recorded at the Cheakamus River near Brackendale from October 13 to 14, 1999.

3. Metal concentrations were calculated at three locations for the "first flush" fall storm event: (1) concentrations in runoff from rock cuts prior to any dilution in the receiving environment, (2) the in-stream concentration once the runoff is fully diluted, and (3) the concentration within Howe Sound based on the loading derived from the drainage basins under consideration.

Simulation of metal loading from the first flush storm event was intended to be representative of a worst case metal loading scenario. First flush storm events generally carry a high chemical load, due to the accumulation of oxidation products containing stored metals and acidity on rock surfaces over the summer months (due to lower precipitation).

Predicted metal concentrations were compared to the following criteria for the protection of freshwater and marine aquatic life to determine potential environmental effects:

- The Canadian Council of Ministers of the Environment's (CCME) Canadian Environmental Quality Guidelines (CEQGs) (updated 2001) (Selected Maximum Acceptable Concentrations: Al < 5 µg/L and Cu < 2 µg/L);
- The Ministry of Water, Land and Air Protection's (MWLAP) British Columbia Approved Water Quality Guidelines (Criteria) (1998; updated 2001); and,
- The MWLAP's A Compendium of Working Water Quality Guidelines for British Columbia (1998; updated 2001).

The two sets of water quality criteria for B.C. are collectively referred to as the BC Water Quality Guidelines (BCWQGs).

5.0 POTENTIAL ENVIRONMENTAL EFFECTS FROM ARD

Based on the results of the ABA testing, the following rock types along the corridor exhibited a NPR < 4 in the samples tested:

TABLE 2: Lithologies Containing Samples with a NPR <4

Design Section	Lithology and Rock Type	Number of Samples Tested	Number of Samples with an NPR < 4
PA Section 1	Coast Plutonic Complex gd2 (Granodiorite)	3	1
PA Section 2	Coast Plutonic Complex qd-d (Quartz Diorite, Diorite)	3	1
PA Section 2	Strongly oxidized, sulphur stained shear zone within Gambier Group IKG (Andesite)	1	1
PA Section 3	Gambier Group IKG (Andesite)	1	1
PA Section 3	Coast Plutonic Complex qd (Quartz Diorite)	1	1
PA Section 3	Gambier Group IKG (Volcanic Breccia / Conglomerate)	2	2
PA Section 4	Gambier Group IKG (Andesite and Tuff)	2	2
PA Section 4 / 5	Gambier Group IKG (Argillite)	3	3
PA Section 7	Gambier Group IKG (Argillite, Siltstone and Sandstone)	2	2
PA Section 7	Gambier Group IKG (Basalt and Feldspar Porphyry)	1	1
PA Section 8	Coast Plutonic Complex gd1 (Granodiorite from Squamish Pluton)	4	1
PA Section 15	Greenstone TrJgs (Chloritized Amphibolite)	1	1

While the Coast Plutonic Complex (CPC) rocks did contain samples with NPR < 4, their bulk ARD potential was classified as non-acid generating, as the total sulphur contents were less than 0.3 wt. % and paste pH values were greater than 7. These rocks were typically characterized by moderately low bulk (Sobek) NP values ranging from 2 to 13[2]. For all but one sample, carbonate NP values were lower than Sobek NP values, indicating that carbonate minerals were not considered to provide significant neutralization capacity. NP was considered to be likely provided by less reactive alumino-silicate minerals, which was supported by the results of mineralogical analysis. The maximum AP value recorded was 1.9, from a total sulphur content of 0.06 %. Only six of the 19 CPC samples had measurable concentrations of total sulphur, with the highest concentration measured at 0.06 wt. %.

The Twin Island Group ($_PJ_T$) rocks had no samples with NPR < 4, and as such, were also classified as non-acid generating.

All of the rock types (andesite, argillite, tuff, volcanic breccia and basalt) in the Gambier Group Lithology (IKG) exhibited potential to generate ARD (NPR < 1). With the exception of one sample from a small rock cut in PA Section 2, paste pH values for IKG samples were neutral to alkaline. The samples had NP values ranging from 3.5 to 18.1 and AP values ranging from below the detection limit (<0.2) to 40.3. Carbonate NP values were generally much lower than Sobek NP values, indicating that carbonate minerals do not appear to provide any significant neutralization capacity. NP was again considered to be likely provided by less reactive alumino-silicate minerals. The Gambier Group rocks exhibited a wider range of sulphur and sulphide content than rocks in the other lithologies along the alignment. Total sulphur contents ranged from below detection (< 0.01 wt. %) to 5.2 wt. %, with an average value of 0.53 wt. % (sulphide values below detection were assumed equal to one half the detection limit in the average calculation).

The overall ARD potential of rocks between Squamish and Whistler (PA Sections 14 to 16) was found to be as low, with the exception of one sample in the Greenstone lithology (TrJgs) with a NPR of 0.7. The low ARD potential was attributed to low total sulphur contents, with over 80% of samples in this section of the highway reporting total sulphur below 0.1 wt. %. As noted earlier, a minimum sulphide content of 0.3 wt. % is generally applied as the threshold, above which, a rock type may have ARD potential.

6.0 POTENTIAL ENVIRONMENTAL EFFECTS FROM ML

Based on the metal loading analyses completed for the study, aluminum and copper concentrations were found to exceed CEQG's and sometimes BCWQG's at the base of the proposed rock cuts, but were found generally to be below the CEQG's and the BCWQG's in the receiving stream or waterbody. Drainage basins where the CEQG's were exceeded in the receiving stream are listed in Tables 3 and 4. When reviewing these results, it was noted that in each case, the background water quality already exceeded the CEQG's requirements for the respective metal.

[2] NP and AP values expressed in units of tonnes $CaCO_3$ equivalent per 1000 tonnes of rock.

Table 3: Predicted Aluminum Concentration in Receiving Stream

Receiving Stream	Total Rock Cut Area	First Flush Flow	Background Dissolved Aluminum	Aluminum Load	Predicted Aluminum Stream Concentration
	m^2	m^3/s	ug/L	g	ug/L
Sclufield	2,300	0.094	18	3	18.3
Rundle	7,280	0.079	23	63	31.7

Table 4: Predicted Copper Concentration in Receiving Stream

Receiving Stream	Total Rock Cut Area	First Flush Flow	Background Dissolved Copper	Copper Load	Predicted Copper Stream Concentration
	m^2	m^3/s	ug/L	g	ug/L
Sclufield	2,300	0.094	2	1	2.1

The small increases in predicted metal concentrations in the receiving environment during the first flow event were also considered to be conservative, because of the assumption of direct discharge from the rock cut face to the receiving stream or water body. In reality other factors may diminish the concentration of dissolved metals entering the receiving environment. These could include:

- Dilution – it is likely that impacted rock drainage water will combine with other drainage sources such as runoff from the widened highway prior to entering the receiving watercourse / waterbody;
- Attenuation – depending on the distance to the receiving watercourse / waterbody and the substrate encountered, metals dissolved in the rock drainage water may adhere to soils and other surfaces through variety of physicochemical processes; and
- Precipitation – metals dissolved in solution may react with other compounds and precipitate out of solution if the concentrations are high enough. In many cases, the newly synthesized compound is chemically inert.

In terms of the impact on Howe Sound, the total predicted increases in aluminum and copper loading for all drainages analyzed were 66 g and 3 g, respectively. These additional loads were considered minor when compared to the background loads from large drainages such as Britannia, which alone were estimated to discharge 8.2 kg of Al and 5.7 kg of Cu into Howe Sound over the course of the same 24-hour first flush event.

While the results of the metal loading analysis indicated that the environmental effects would likely be small, it was considered prudent that a surface water quality monitoring program be established to verify the environmental assessment results both during and after construction. Should localized effects be identified by the monitoring program, mitigation measures could be initiated to minimize the impact. Mitigation options proposed included shotcreting of cut faces to isolate them from surface water, lining of the ditches with lime or limestone and construction of a more complex leachate collection and treatment systems.

7.0 POTENTIAL EFFECTS FROM RE-USE AND DISPOSAL OF EXCAVATED ROCK

The preliminary design project scope indicates that approximately 1,140,000 m^3 of rock will to be excavated along the highway corridor between Horseshoe Bay and Squamish and a further 640,000 m^3 of rock will be excavated along the highway corridor between Cheakamus Canyon and Function Junction (SNC, 2003). Of the total excavated, 140,000 m^3 was anticipated to be PAG material between Horseshoe Bay and Squamish, with remainder anticipated to be NPAG. No PAG material was anticipated to be excavated between Cheakamus Canyon and Function Junction.

Once the PAG material was identified, potential environmental effects were determined for each of the potential re-use and disposal options, including any handling and temporary stockpiling issues as the material is taken from the excavation and applied to its designated end-use or disposed of in a waste rock pile or ocean disposal site. Areas where ARD/ML could be generated and discharged if not properly managed include:

- Muck piles at the site of the excavation;
- Temporary stockpiles required for aggregate processing;
- Temporary stockpiles required prior to fill placement;
- Waste rock dumps;
- Temporary stockpiles required for ocean disposal; and
- At barge loading sites.

Any discharge from these sources would be additive to the metal loading from the rock cut faces. In addition, due to the larger surface area of the blasted rock compared to that of the excavated cut face, the metal loading could be larger from the excavated PAG material than from the cut face itself.

As the potential for re-use of the PAG material as bulk fills or aggregates was determined to be limited, a decision was made towards the end of the study to dispose of all PAG material generated by the project either in a designated Ocean Disposal Site to limit its ability to oxidize or within the abandoned Britannia mine workings located in PA Section 6. The Britannia mine option was considered to be feasible from an ARD/ML perspective, because the groundwater discharge from the mine has been designated for treatment irrespective of the highway upgrades, as part of the mine's closure plan. By adopting this materials management strategy, the potential environmental effects from the excavated PAG material were again considered to be small and any local effects which may occur during the transportation of the material to the disposal site could be mitigated using standard environmental best management practices for contaminated soil.

8.0 CONSIDERATIONS WHEN PLANNING FUTURE STUDIES

Based on our experience conducting this assessment, important factors which should be considered when planning similar studies are as follows:

1. Early definition of the project scope is important, as the results of the assessment are highly dependent on the location of the proposed cuts and the volume of excavated material to be generated from them. If the assessment is completed when the project scope is still poorly defined, there is a risk that multiple revisions will need to be undertaken, at additional cost to the Owner;

2. Because the project scope may change considerably during the conceptual and preliminary design stages of a project when the environmental assessment needs to be undertaken, and because of the associated cost of late scope changes outlined above, it is best to conduct the environmental assessment using a phased approach. The initial sampling and testing programs should provide a screening level of assessment, with comprehensive testing of individual cuts reserved until later in the design process, when the cut locations, excavation volumes and potential ARD/ML issues are better known and understood;

3. When conducting detailed testing of individual cuts, it is important to test the entire cut length within a potentially problematic lithology, even if adjacent samples tested during the screening level study in this particular unit do not indicate a potential problem. This is important, as ARD potential can not be determined from visual inspection alone and can vary considerably within a given exposure.

4. During the construction phase of the project, it is important to conduct water quality monitoring in the vicinity of the excavated cuts and down stream of any stockpiles of PAG materials, to verify the assessment results. While the assessment methodology should be designed to produce reasonable, yet conservative results, the many variables which inevitably arise when simulating such complex processes, always lead to some uncertainty in the results. A well designed monitoring program is a useful means to overcome such concerns.

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Building the Case for Soft Solutions: Coastal Erosion and the 2004 Hurricane Season in Florida

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Abstract

During the 2004 hurricane season, the state of Florida was impacted by four hurricanes - Charley, Frances, Ivan and Jeanne and one tropical storm – Bonnie. These storms caused widespread damage to infrastructure and left not a single county in the State unaffected. Since the start of record keeping in the 1850's, only Texas in 1886 has also been impacted by 4 hurricanes in a single season. Coastal erosion was extensive in the vicinity of the storms' landfall locations resulting in damage to coastal structures including roads and bridges. Field investigations carried out on the Southwest Gulf Coast as part of the disaster response efforts enabled post-storm visual assessments of the response of coastal erosion protection structures. These investigations indicate that soft solutions to coastal erosion like beach nourishment weathered the storms better than more traditional hard solutions like seawalls and groins.

1.0 Introduction

The 2004 hurricane season spawned five storms that impacted Florida: Bonnie, Charley, Frances, Ivan, and Jeanne. Four of these storms reached hurricane strength on the Saffir-Simpson Hurricane Scale (NOAA, 2004b, 2005a-c) while Bonnie remained a Tropical Storm (NOAA, 2004a). All the storms resulted in erosion of coastal areas in Florida and impacts to infrastructure including seawalls, groins, revetments and areas of beach nourishment. Natural disasters were declared for all the storms, Tropical Storm Bonnie and Hurricane Charley being declared together.

As part of the disaster response to the 2004 Hurricane Season in Florida, the Federal Emergency Management Agency (FEMA) engaged beach erosion experts to assess and report on coastal erosion issues. FEMA's mandate, as part of the Department of Homeland Security, is to "prepare the US for all hazards and effectively manage federal response and recovery efforts following any national incident" (FEMA, 2003). Six beach erosion experts were deployed by FEMA around the state to assess beach erosion impacts: three on the Atlantic Coast to assess the impacts from Hurricanes Frances and Jeanne, two on the Florida Panhandle to assess the impacts from Hurricane Ivan, and one to the Southwest Gulf Coast to assess the impacts from Hurricane Charley and Tropical Storm Bonnie.

Approximately 142 miles of Florida's 825 miles of sandy coastline are managed by the Florida Department of Environmental Protection (FDEP, 2000). All the sections of managed coastline were impacted and thus they were assessed by FEMA specialists. Due to the extensive length of impacted coastline from the 2004 Hurricane Season in Florida, the assessments carried out under

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FEMA's disaster response provides an excellent opportunity to evaluate the performance of coastal erosion protection designs. This opportunity is certainly the case on the Southwest Gulf Coast which felt the impact of all the storms from the 2004 season.

On the Southwest Gulf Coast both hard and soft erosion protection methods were impacted by the storms. This paper will show from observations of coastal erosion that soft approaches to erosion protection (e.g. beach nourishment and dune restoration) weathered the storms better than the more traditional hard approaches (e.g. seawalls, groins and revetments) and that this difference in performance had implications for coastal roads and bridges in the Southwest Gulf Coast.

2.0 The Storms

Tropical Storm Bonnie made landfall near Apalachicola on August 12 with sustained winds of approximately 55 knots. The storm surge was measured as 1.9 feet on Cedar Key (NOAA, 2004a). Following landfall the tropical storm proceeded northeastwards across the Eastern United States. Due to the coincidence of Bonnie with Charley, impacts of the southwest Gulf Coast were generally attributed to Charley.

Hurricane Charley made landfall on the Southwest Gulf Coast of Florida near Cayo Costa on August 13 as a Category 4 hurricane with sustained winds of approximately 130 knots. The storm surge was measured as 4.2 feet on Estero Island (Fort Myers) and approximately 3.5 feet on the Caloosahatchee River with visual estimates of storm surge reaching 6 to 7 feet on Sanibel and Estero Islands (FDEP, 2004a; NOAA, 2005a). Following landfall the hurricane proceeded inland past Orlando before moving off the northwest coast of the state near Daytona Beach on August 14 with sustained winds of 70 knots.

Hurricane Frances made landfall on the Atlantic Coast of Florida at the south end of Hutchinson Island on September 5 as a Category 2 hurricane with sustained winds of approximately 90 knots. The storm surge was estimated to be 8 feet near Vero Beach and 6 feet around Cocoa Beach. Following landfall the hurricane proceeded northwestward across Florida to emerge as a Tropical Storm in the Gulf of Mexico near New Port Richey with sustained winds of 55 knots. A storm tide of between 4 and 4.5 feet was reported on the Southwest Gulf Coast from Naples, Fort Myers, Port Manatee and Clearwater Beach. Final landfall was made near the mouth of the Aucilla River on September 6 (FDEP, 2004f; NOAA, 2004b).

Hurricane Ivan made landfall just west of Gulf Shores, Alabama on September 16 as a Category 3 hurricane with sustained winds of approximately 105 knots. Due to the width of the eye (40-50 nautical miles) some of the strongest winds affected the Alabama-Florida Panhandle border. The storm surge was estimated to be 10 to 15 feet from Destin on the Florida panhandle westwards into Alabama. A storm surge of 3.5 feet was reported around Tampa Bay indicating the effect of Hurricane Ivan on the Southwest Gulf Coast (FDEP, 2004e; NOAA, 2005c). Following landfall the hurricane proceeded northeastwards across the US.

Hurricane Jeanne made landfall on the Atlantic Coast of Florida at the south end of Hutchinson Island on September 26 as a Category 3 hurricane with sustained winds of approximately 100 knots. The storm surge was measured to be 3.8 feet at Port Canaveral and estimated to be 6 feet between Melbourne and Fort Pierce. Following landfall the hurricane proceeded northwestwards up the Florida peninsula. A storm surge of 3.5 feet was measured at Cedar Key on the Gulf Coast as Hurricane Jeanne moved north (FDEP, 2004f; NOAA, 2005b).

3.0 Cumulative Impacts

On the Southwest Gulf Coast, the coastline was impacted by the passage of all four hurricanes, especially in Lee County where Hurricane Charley made landfall (FDEP, 2004f). The impacts from Tropical Storm Bonnie on the Southwest Gulf Coast on August 12 were somewhat obscured by the subsequent arrival of Hurricane Charley a day later. Due to the tracks followed by the various hurricanes, some beaches on the Southwest Gulf Coast recorded the passage of the hurricanes as separate erosional scarps (Figure 1). However, in most places the individual impacts of the hurricanes could not be ascertained.

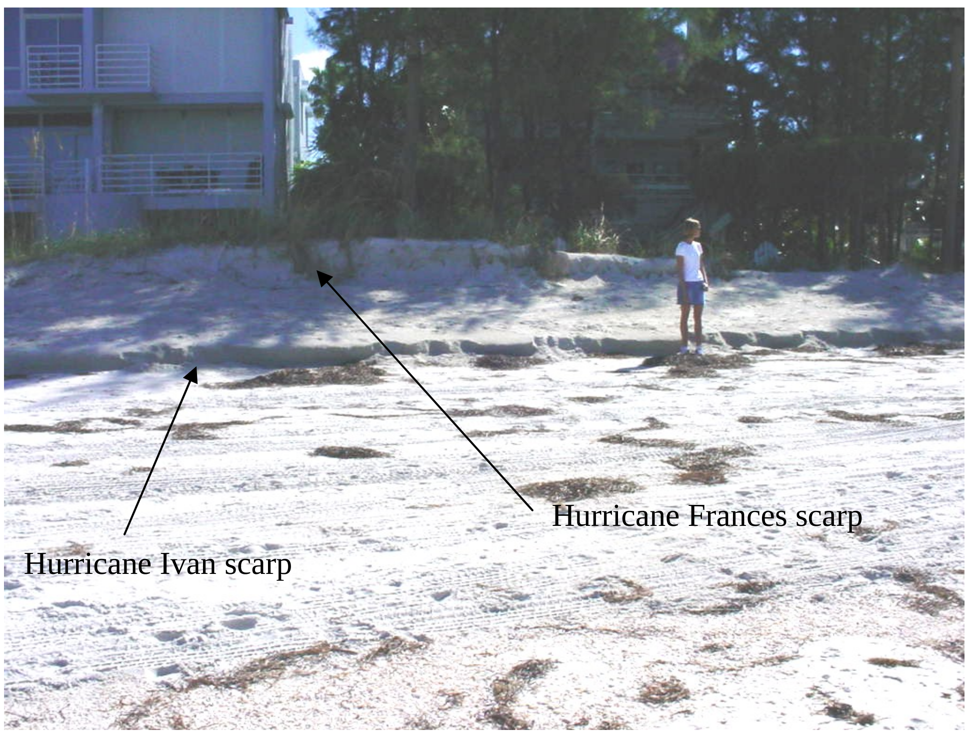

Figure 1: Storm Scarps on Treasure Island, Pinellas County, FL.

Regardless of whether individual impacts from the later hurricanes could be identified, the overall impacts were generally cumulative since the coastline had already been affected by the passage of Hurricane Charley. Assessments of coastal erosion impacts in the field had to be undertaken by considering the cumulative effects as a single occurrence. Rarely were intermediary data available to enable the discrete impacts of each storm to be ascribed in turn (e.g., CEC & CTC, 2004; CTC, 2004; CPE, 2004). This meant that the evaluation of coastal erosion protection structures from a design point of view on the Southwest Gulf Coast needed to establish the sort of design event represented by the combined impact of the four hurricanes.

4.0 Estimated Design Event Recurrence Interval

Florida is only the second state in US recorded history to have been impacted by 4 hurricanes in a single season. The previous occurrence was in 1886 when Texas was impacted by 4 storms (NOAA, 2000). Records of Hurricane occurrences in the US go back to around the 1850's. Given that records of an area being impacted by 4 hurricanes in a single season has only occurred twice in approximately 150 years, it is reasonable to consider that the likelihood of recurrence of the 2004 hurricane season is at least of the order of once every 75 to 100 years.

From this perspective it may be suggested that the cumulative impacts of the four hurricanes on the Florida Coast was equivalent to the impact of a single storm with a recurrence interval of at least 75 years and more likely of the order of 100 years. Thus, from a design perspective, the preliminary observations of erosion and impacts to coastal erosion protection structures were made assuming the coastline had been affected by a 100-year event.

5.0 Coastal Management Practices

A non-bedrock coastline is a dynamic feature on the landscape and not a static one. The coastline shifts landward or seaward in response to the supply and availability of sediment to be moved and changes in the wave regime, particularly as a result of storm activity. This changeable nature of the coast presents unique challenges to locating, designing, constructing and maintaining infrastructure like roads and bridges. These structures, once built, represent a fixed line in space relative to a shoreline whose position naturally changes over time. These changes are typically viewed as problems to be solved.

When sedimentation occurs in places where navigation needs to be maintained, dredging is often undertaken to keep the water depths in a range that suit human purposes but might not be in equilibrium with the natural environment. When erosion occurs in areas where infrastructure has been built, erosion protection designs are developed to protect the structure. How these problems are addressed often affects adjacent infrastructure or property since the coastal equilibrium is organized at a scale larger than a single property or section of shoreline. The approach that is chosen ultimately influences the success of the design and the need for ongoing maintenance.

Coastal management practices designed to provide erosion protection have traditionally relied on hard engineered approaches including seawalls, groins and revetments. The design philosophy has been to build a non-erodible barrier that will stop erosion and provide the desired protection. The arresting of erosion by non-erodible barriers has included seawalls and shore-parallel revetments (e.g. riprap and wood bulkheads) designed to protect land situated landward of the high water mark. It has also included shore-perpendicular features like groins designed to trap sediment being carried alongshore by longshore drift.

More recently, softer engineered approaches have been developed to provide erosion protection for shorelines. These soft approaches include beach nourishment and dune restoration and stabilization. In beach nourishment, sand is placed on the coastline to provide a “working” fill that the storm waves can rework (Figure 2). In dune restoration and stabilization, both sand and vegetation is placed on the shoreline to build up the seaward dune field to provide a long-term sediment supply to the shoreline and arrest shoreline recession.

Figure 2: Beach Nourishment on Treasure Island, Pinellas County, FL (FDEP, 2004c).

The design philosophy of these softer engineered solutions is to build a structure out of erodible sediments such that they will be transported by waves and thus absorb the energy that previously had been driving coastal erosion. The design typically includes both a design fill component and a sacrificial fill component that is replenished on a periodic basis specified at the design phase. The design fill component is constructed along the shoreward edge of the renourishment zone to a size that is expected to withstand the design storm, its width and height being dictated by the estimated magnitude of erosion that will occur during the design storm. The sacrificial fill component is built seaward of the design fill component to provide sufficient volume of sediment to accommodate erosion of the shoreline at the average annual rate of erosion and still retain the design fill until the next scheduled renourishment period.

Observations of the performance of a variety of hard and soft structures were made on the Southwest Gulf Coast of Florida.

6.0 Observations from the Southwest Gulf Coast of Florida

6.1 Siesta Key – Seawalls and Dune Restoration

Observations of coastal erosion in Siesta Key in Sarasota County are illustrative of the relative behaviour of hard and soft solutions. Vehicle access along the key is made possible in places by a road located along the terrestrial margin of the foreshore that has been protected in places by a seawall (Figure 3) and a dune restoration project (Figure 4) situated less than a mile apart. In both cases sand overwash was observed landward of the structures.

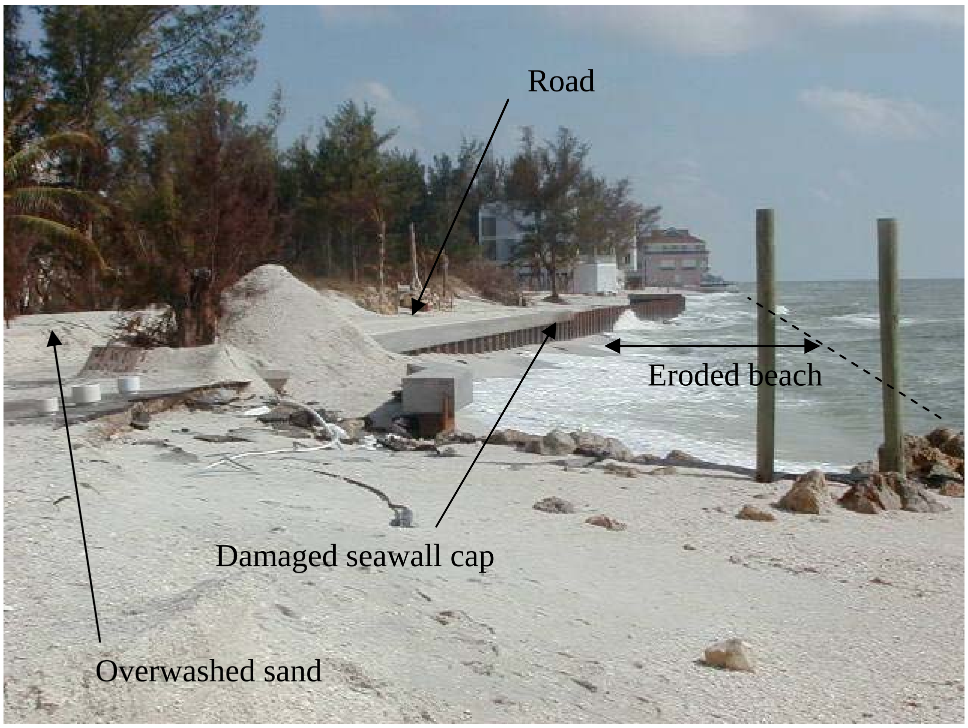

Figure 3: Seawall damage and eroded beach, Siesta Key, Sarasota County, FL.
(Photo Courtesy of FDEP, 2004d)

The seawall was observed to have lost its concrete cap and the road it protected had been undermined behind the sheet metal seawall. Additionally, the beach in front of the seawall was observed to have been eroded to the point of being absent, thereby allowing larger waves to reach the seawall than if a beach were present. Where dune restoration had been undertaken, the dune was observed to have been eroded but sufficient remnant remained to protect the road and the parking lot used for beach access. Although there had been overwash of sand, the road was not observed to have been undermined at this location.

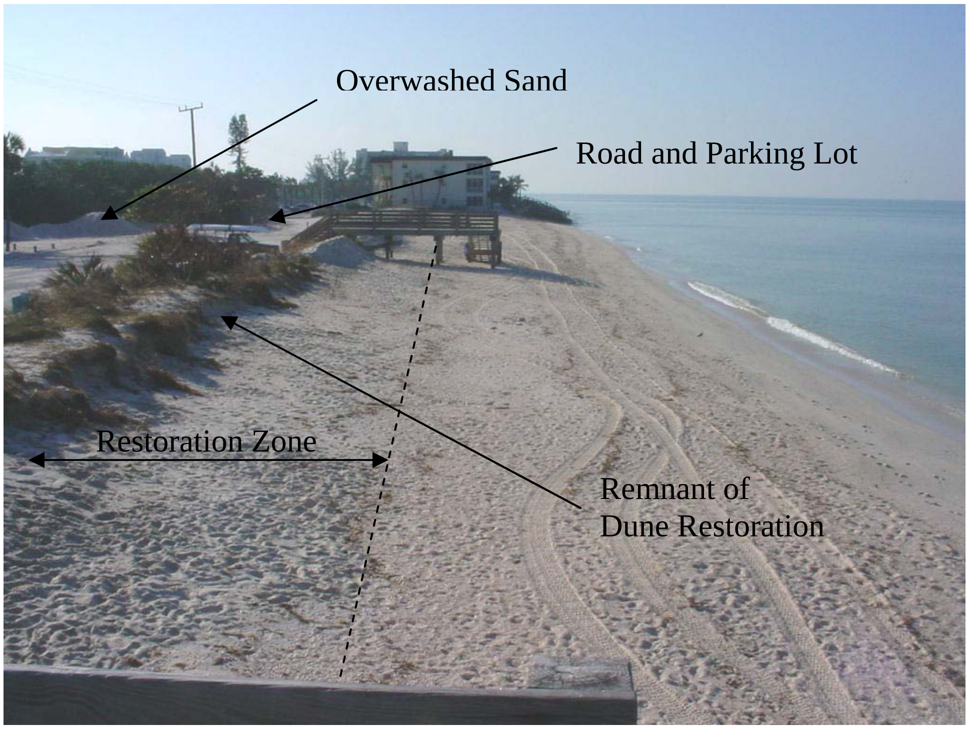

Figure 4: Eroded dune restoration and beach, Siesta Key, Sarasota County, FL.

6.2 Longboat Key and Venice Beach - Hard Structures and Beach Nourishment

Longboat Key is a beach nourishment project maintained by the Town of Longboat Key in co-operation with the FDEP. A renourishment project was scheduled for 2004/2005 (FDEP, 2003) indicating that the renourished beach was reaching the end of the design life for the sacrificial fill component. This meant that the impacts from the 2004 hurricane season impacted the design fill portion of the nourishment project since the sacrificial fill component had all but been removed. In some locations on Longboat Key, hard structures have been built to protect existing infrastructure.

Figure 5 illustrates a portion of the beach of Longboat Key where both beach nourishment and hard structures (seawalls and groins) have been built. Although the shoreline had been nourished historically, the groins constructed at this location were observed to have affected the longshore drift redistribution of sediment along the beach. This effect was observed to have led to obstruction of sediment delivery between the two groins resulting in erosion of the beach and more direct wave attack on the seawall historically constructed as protection for the buildings. Observations made along the twelve mile length of Longboat Key indicated that the locations where hard structures had been constructed to resist erosion in addition to the beach nourishment program, the shoreline was more heavily eroded than immediately adjacent sections of coastline.

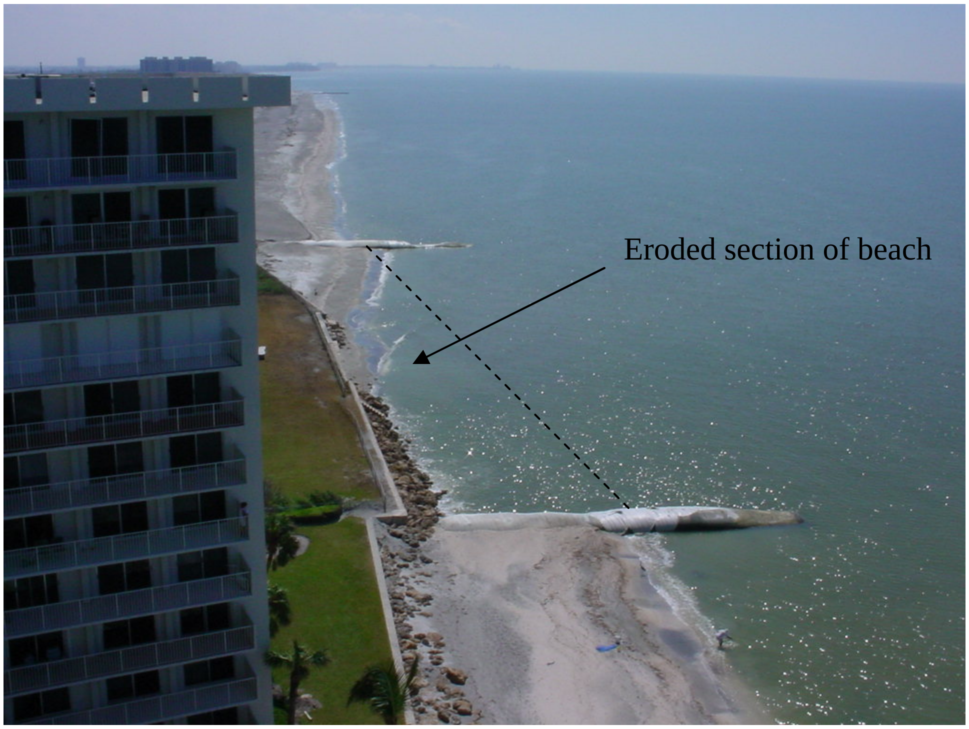

Figure 5: Seawall and groin effects on beach nourishment

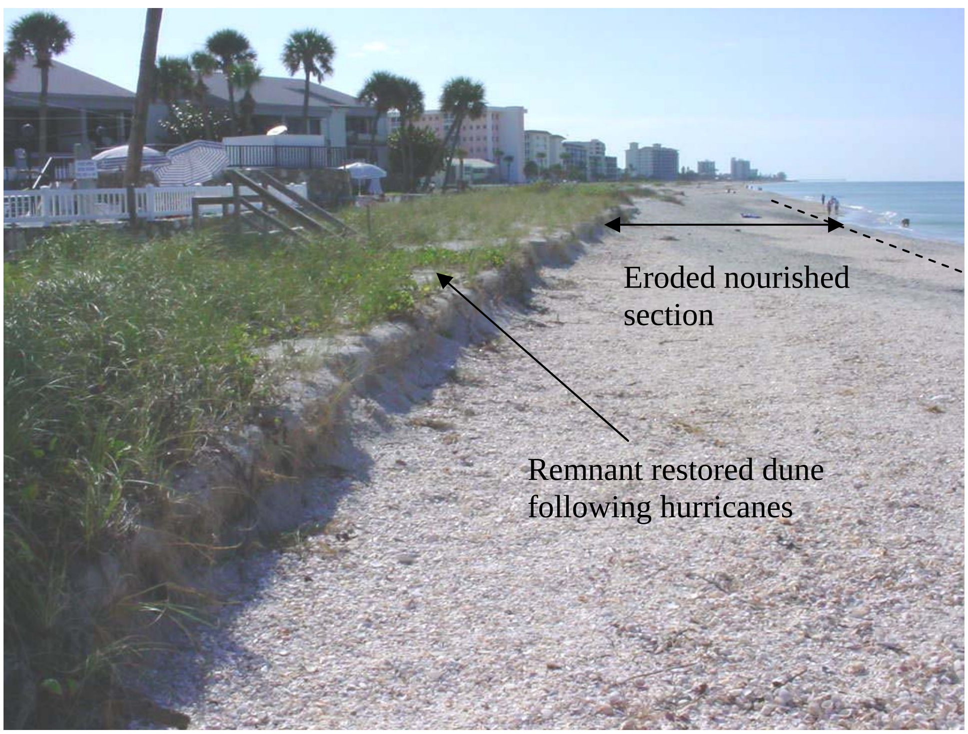

Figure 6: Storm impacted beach renourishment, Venice Beach, Sarasota County, FL.

Venice Beach is a beach renourishment project maintained by the US Army Corps of Engineers. A renourishment project was scheduled for 2004/2005 (FDEP, 2003) indicating that, like Longboat Key, the renourished beach was reaching the end of the design life for the sacrificial fill. Although impacted by all the storms, the design fill was not completely eroded (Figure 6) indicating that it accommodated the storm-generated waves and associated storm surge from the 2004 hurricane season well.

6.3 Lovers Key Bridge and Pavilion

A beach renourishment project was underway on Lovers Key in Lee County as Hurricane Charley approached Florida. Public access to the beach is provided by a concrete one that leads to a wooden pavilion on the shoreline and a wooden one leading to the shoreline. The concrete bridge and wooden pavilion had been protected by the renourished beach prior to the arrival of the hurricanes. The wooden bridge had not been protected by the nourished beach. The wooden bridge (Figure 7) was observed to have been displaced from its moorings and severely undermined and damaged by the storm waves. This bridge was still closed to all traffic in November 2004, approximately 2 months after the hurricanes impacted Florida. The wooden pavilion bridge protected by the beach renourishment project (Figure 8) was observed to have been well protected.

Figure 7: Wooden bridge damaged by hurricanes, Lovers Key, Lee County, FL.
(Photo courtesy of Rob Neal, Lee County)

Figure 8: Protected wooden pavilion at Lovers Key, Lee County, FL.
(Photo courtesy of Rob Neal, Lee County)

6.4 Siesta Key and Captiva Island Beach Roads

Both Siesta Key and Captiva Island have roads built right along the shoreline (Figures 9 and 10). Beach Road on Siesta Key (Figure 9) was observed to have been protected by a rubble armour layer and a pair of groins. The impact of the storm-generated waves was observed to have undermined the road to a width of approximately 10 feet. Sand was observed to have been overwashed onto the lawns of properties on the landward side of the road.

The main road on Captiva Island (Figure 10) is used as part of the hurricane evacuation route from the island and is therefore a critical artery. This road has been protected by beach nourishment structures since the 1980's (CPE, 2004). The nourished beach includes both sections of restored dune and a beach fill. Scheduled for nourishment in 2004/2005 (FDEP, 2003) the beach nourishment was reaching the end of the design life for the sacrificial fill.

Although the eye of Hurricane Charley passed close by and crossed directly over North Captiva Island only a few miles to the north, the restored dune and remaining beach fill was observed to have protected the road. The road was not observed the have been undermined or eroded to the point that vehicle traffic was limited. Some overwash of sand into the properties landward of the road was observed. Compared with the observations of road damage from Siesta Key the beach renourishment project appeared to have provided adequate erosion protection on Captiva Island.

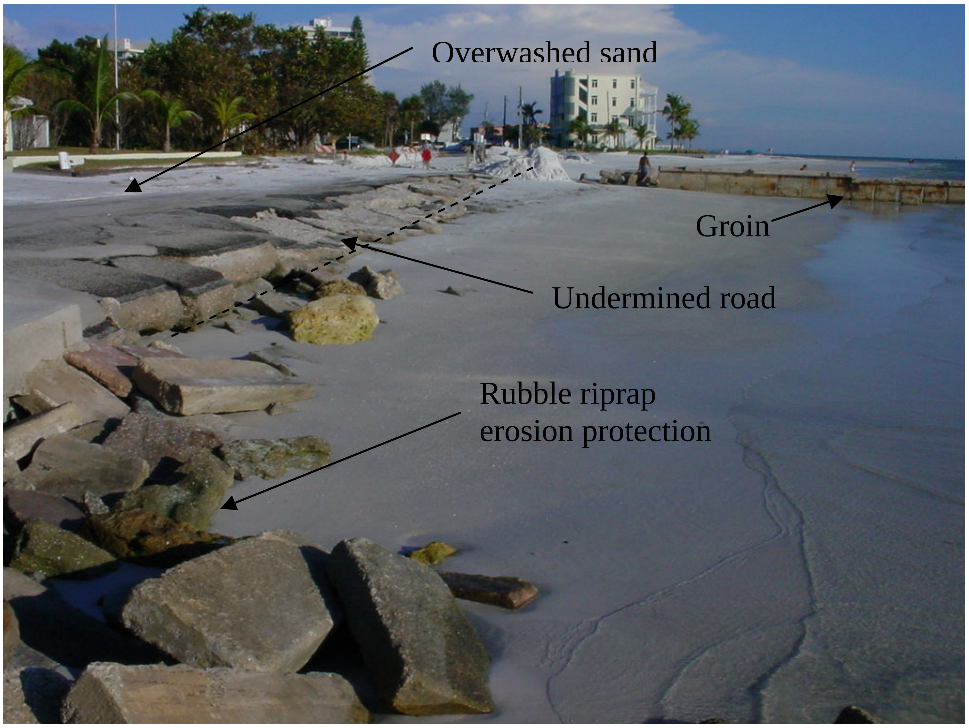

Figure 9: Eroded road on Siesta Key, Sarasota County, FL.

Figure 10: Road and restored dune, Captiva Island, Lee County, FL
(Photo courtesy of FDEP, 2004b)

7.0 Conclusion

Assessments of coastal erosion and performance of erosion protection structures were made throughout Florida as part of the disaster response effort following the 2004 hurricane season. Field observations from the Southwest Gulf Coast enabled assessment of shoreline segments that had been impacted by all of the storms to hit Florida during the 2004 season.

Based on the observations made in the field, the soft solutions appear to have weathered the storms better than the hard solutions. These observations enhance the case for building beach renourishment projects on the coast as a means of controlling erosion. Although hard engineered erosion protection solutions may ultimately be necessary, beach nourishment and dune restoration should be considered everywhere practicable.

8.0 Acknowledgements

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Geophysical Applications for Bridge Design, North Carolina Outer Banks: Results of Marine Seismic and Resistivity Investigations in the Pamlico, Sound

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Abstract

The North Carolina Department of Transportation evaluated the potential construction of a 17 mile long back barrier bridge that would be the replacement bridge for the existing Bonner Bridge at Oregon Inlet, North Carolina. Here, the NC DOT was interesting in evaluating existing subsurface conditions that may impact the installation of bridge footings along the proposed pathway of the bridge. As such, numerous borings were completed along the established bridge pathway. Significant variation in the boring description indicated that the overall geology might be complex and affected by paleo-channel structures.

As a result, the NC DOT contracted ECS, Limited and their specialist team to propose and complete several types of geophysical investigations to better identify potential paleo-channel features that may affect the design of the bridge. The team consisted of Engineers and geologist from ECS, Geo Solutions and East Carolina University. The East Carolina team brought with them approximately 200 miles of raw sub-bottom profiling data that was previously collected in the general vicinity of the proposed bridge pathway.

The fieldwork was completed during the late winter of 2004 and had to be planed in accordance with the known temperamental weather of the North Carolina Outer Banks.

The team conducted the investigation in two phases:

Phase I consisted of the completion of approximately 25 miles of sub bottom profiles that collected records to a depth of approximately 300 feet. The results of these data and the previously collected information by East Carolina University formed the basis for identifying the presence of numerous intertwined paleo-channel features in the sub surface. These channels have been relatively dated and exhibit at least three periods of distinct channel incision in the area underlying the Pamlico Sound.

Phase II of this investigation consisted of the completion of a marine resistivity investigation in the environs of Oregon Inlet. The purpose of this investigation was to evaluate the presence of channel structures utilizing marine resistivity techniques. The results of this phase of study indicated that the saltwaters of the sound and interstitial waters of the shallow sediments adversely affect marine resistivity results.