64TH HIGHWAY GEOLOGY SYMPOSIUM

SEPTEMBER 9-12, 2013 | NORTH CONWAY GRAND HOTEL | NORTH CONWAY, NEW HAMPSHIRE

PROCEEDINGS

Hosted By:

The New Hampshire Department of Transportation

On Cover - Picture of Mount Washington Auto Road

64^{th} ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM
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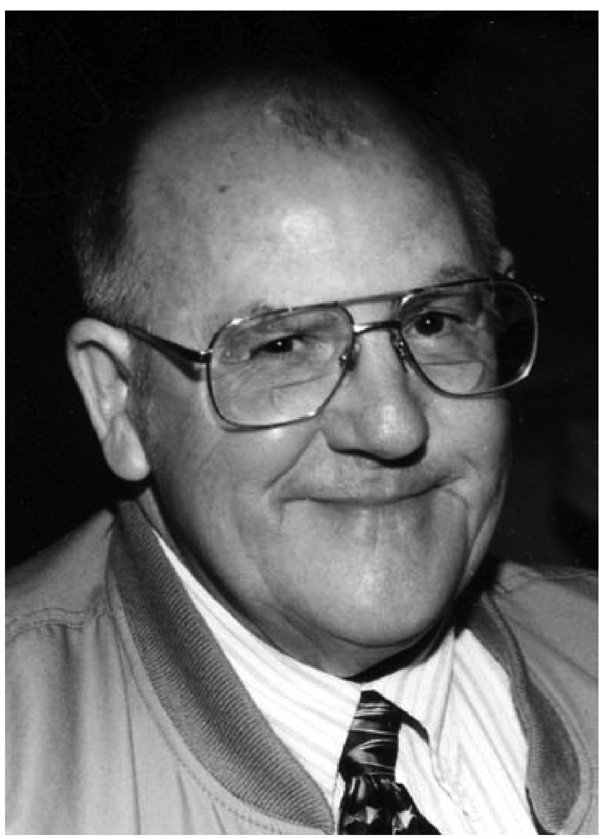
64th Proceedings Volume, Highway Geology Symposium Dedicated to Earl Wright and Bill Lovell

Earl Wright, 1931-2012

Earl was admired and respected by his peers. Earl was a graduate of the University of Kentucky and was employed by the Kentucky Transportation Cabinet, Geotechnical Branch for 31 years (1970-2001) where he served as Chief Geologist. Prior to this he was employed as a geologist in the Petroleum Industry. He is remembered as a professional that always strived to find the most economical solutions for the numerous geological problems encountered in the State of Kentucky. He was a perfect example of how Geologist and Engineers work together to disseminate information of geology and geotechnics to the practice of transportation engineering.

Earl was a registered professional geologist and member of the Kentucky Society of Professional Geologist, Association of Engineering Geologists and National Steering Committee for the Highway Geology Symposium.

Earl attended his first HGS meeting in 1968 which was held in Morgantown, West Virginia. Earl was Secretary for the HGS in 1993, Vice Chairman in 1994 and 1995, and Chairman 1996 thru 1999. He was the recipient of the prestigious HGS Medallion in 1997 and was an Emeritus Member of the National Steering Committee.



64th Proceedings Volume, Highway Geology Symposium Dedicated to Earl Wright and Bill Lovell

C. W. "Bill" Lovell, 1922-2013

Bill Lovell, Professor Emeritus, School of Civil Engineering, Purdue University, died at age 90 at home in West Lafayette, IN on June 15, 2013, with his wife Mary Ellen at his side. Bill was a geotechnical engineer with a keen interest in applied geology. He was an active member of ASTM, ASCE, and TRB, where he was presented with several prestigious awards and a member of the Highway Geology Symposium (HGS). Bill served on the HGS Steering Committee for many years serving as the Vice President before he became an Emeritus Member of the Committee on his retirement from teaching at Purdue in 1993. He served as the Co-Chairman of the 36th Annual Highway Geology Symposium in Clarksville, IN, May 13-15, 1985. He was the recipient of the highest award of the organization, the HGS Medallion award in 1989.

Bill was dedicated to teaching and research participating for 45 years in the geotechnical engineering program at Purdue. During this time he supervised 100+ graduate studies and nearly 50 research theses. His research interests were broad and varied including soft rocks (shale), compaction and compacted properties, soil fabric and pore size distribution, slope stability and erosion, cold weather problems, pavements, and most recently uses of waste materials in geotechnical engineering. His work on the STABL analysis of soil landslides formed the basis for the procedures used widely today for slope stability calculations. Following his mandatory retirement from academic endeavors, Bill taught the COVEY method to Purdue personnel until 2010 when he fully retired from the University.

Beginning in 2003, the C.W. Lovell Distinguished Lecture series was established through a generous contribution by Dr. Lovell to set up this annual invited lectureship. Distinguished geotechnical engineers, many with an international reputation, have been invited to make this annual presentation. The tenth lectureship was held this past fall with Bill in attendance. Many of Bill's former students and his university colleagues have expressed their condolences at Bill's passing. He is survived by two children and his wife, Mary Ellen.

64^{th} ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

North Conway, New Hampshire
September 9^{th} – September 12^{th}, 2013

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HIGHWAY GEOLOGY SYMPOSIUM

History, Organization, and Function

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on March 14, 1950, in Richmond Virginia. Attending the inaugural meeting were representatives from state highway departments (as referred to at that 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, 62 consecutive annual meetings have been held in 32 different states. Between 1950 and 1962, the meetings were 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 from the east to the west. The Annual Symposium has moved to different location as listed on the next page.

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 participants.

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

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, WV
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, TN
37th	1986	Helena, MT	38th	1987	Pittsburg, PA
39th	1988	Park City, UT	40th	1989	Birmingham, AL
41st	1990	Albuquerque, NM	41st	1991	Albany, NY
43rd	1992	Fayetteville AR	44rd	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
57th	2006	Breckinridge, CO	58th	2007	Pocono Manor, PA
59th	2008	Santa Fe, NM	60th	2009	Buffalo, NY
61st	2010	Oklahoma City, OK	62nd	2011	Lexington, KY
63rd	2012	Redding, CA	64th	2013	North Conway, NH
65th	2014	Cheyenne, WY	66th	2015	TBD

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

The field trip is the focus of the meeting. In most cases, the trips cover approximately 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 interests. 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 mine region was visited in Texas; and the Tennessee meeting in 1981 provided stops at several repaired landslide 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 treatments 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 of 876 feet above the water.

In Cody, Wyoming the 1996 field trip visited the Chief Joseph Scenic Highway and the Beartooth Uplift in northwest 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 the Oak Creek Canyon near Sedona and a mining ghost town at Jerrome, Arizona. The Virginia meeting in 1999 visited the “Smart Road” Project that was under construction. This was a joint research project of the Virginia Department of Transportation and Virginia Tech University. The Seattle Washington meeting in 2000 visited the Mount Rainier area. A stop during the Maryland meeting in 2001 was the Sideling Hill road cut for I-68 which displayed a tightly folded syncline in the Allegheny Mountains.

The California field trip in 2002 provided a field demonstration of the effectiveness of rock netting against rock falls along the Pacific Coast Highway. The Kansas City meeting in 2004 visited the Hunt Subtropolis which is said to be the “world’s largest underground business complex”. It was created through the mining of limestone by way of the room and pillar method. The Rocky Point Quarry provided an opportunity to search for fossils at the North Carolina meeting in 2005. The group also visited the US-17 Wilmington Bypass Bridge which was under construction. Among the stops at the Pennsylvania meeting were the Hickory Run Boulder Field, the No.9 Mine and Wash Shanty Museum, and the Lehigh Tunnel.

The New Mexico field trip in 2008 included stops at a soil nailed wall along US-285/84 north of Santa Fe and a road cut through the Bandelier Tuff on highway 502 near Los Alamos where rockfall mesh was used to protect against rockfall. The New York field trip in 2009 visited the Niagara Falls Gorge and the Devil’s Hole Trail. The Oklahoma field trip in 2010 toured through the complex geology of the Arbuckle Mountains in the southern part of the state along with stops at Tucker's Tower and Turner Falls.

In the bluegrass region of Kentucky, the 2011 HGS field trip included stops at Camp Nelson which is the site of the oldest exposed rocks in Kentucky near the Lexington and Kentucky River Fault Zones. Additional stops at the Darby Dan Farm and the Woodford Reserve Distillery illustrated how the local geology has played such a large part in the success of breeding prized Thoroughbred horses and made Kentucky the "Birthplace of Bourbon". In Redding, California, the 2012 field trip included stops at the Whiskeytown Lake, which is one in a series of lakes that provide water and power to northern California. Additional stops included Rocky Point, a roadway construction site containing Naturally Occurring Asbestos (NOA), and Oregon Mountain where the geology and high rainfall amounts have caused Hwy 299 to experience local and global instabilities since first constructed in 1920.

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 papers may be obtained from the Treasurer of the Symposium.

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

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

A number of past members of the national steering committee have been granted Emeritus status. These individuals, usually retired, resigned from the HGS Steering Committee, or are deceased, have made significant contributions to the Highway Geology Symposium. A total of 33 persons have been granted Emeritus status. Thirteen 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 held in Albany, New York were dedicated to Burrell S. Whitlow (1929 - 1990, Virginia).

In 2013 the Proceedings of the 64th HGS held in North Conway, New Hampshire are dedicated to Earl Wright and Bill Lovell.

HIGHWAY GEOLOGY SYMPOSIUM

Emeritus Members of the Steering Committee

Emeritus Status is granted by the Steering Committee

R.F. Baker*
John Baldwin
David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
Richard Cross
A.C. Dodson*
Walter F. Fredericksen
Brandy Gilmore
Robert Goddard
Joseph Gutierrez
Richard Humphries
Charles T. Janik
John Lemish
Bill Lovell*
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Willard McCasland
David Mitchell
Harry Moore
W.T. Parrot*
Paul Price*
David L. Royster*
Bill Sherman
Willard L. Sitz
Mitchell Smith
Steve Sweeney
Sam Thornton
Berke Thompson*
Burrell Whitlow*
W.A. "Bill" Wisner
Earl Wright*
Ed J. Zeigler
Harry Moore

(* 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. Edmondson*	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
Mike Hager	2005
Joseph A. Fischer	2007
Ken Ashton	2008
A. David Martin	2008
Michael Vierling	2009
Richard Cross	2009
John F. Szturo	2010
Christopher Ruppen	2012
Jeff Dean	2012

(*Deceased)

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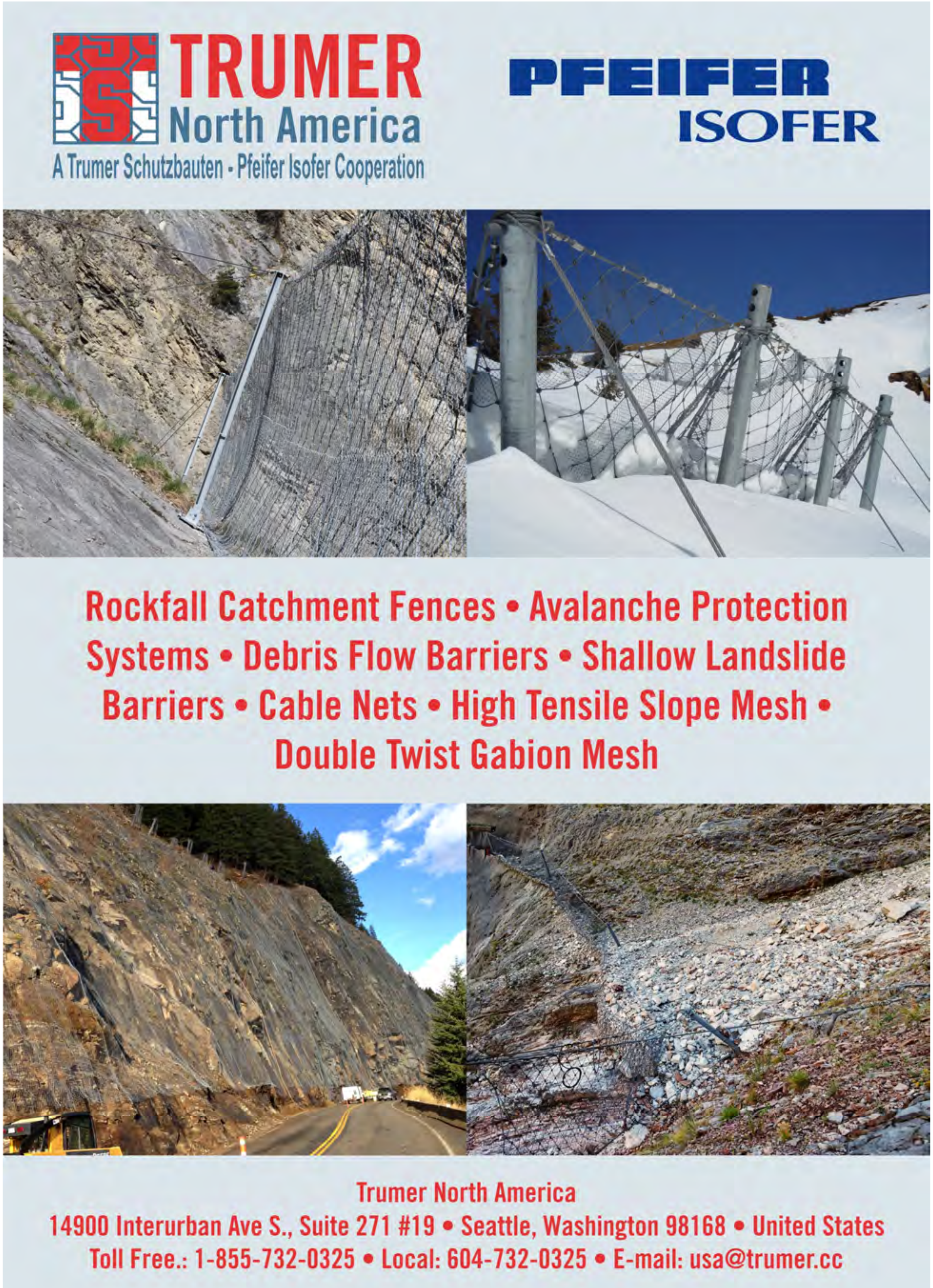
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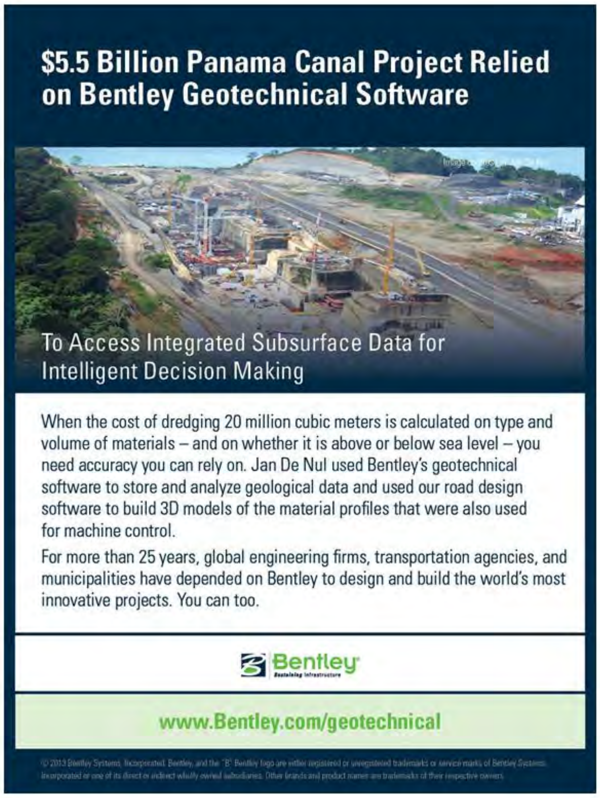

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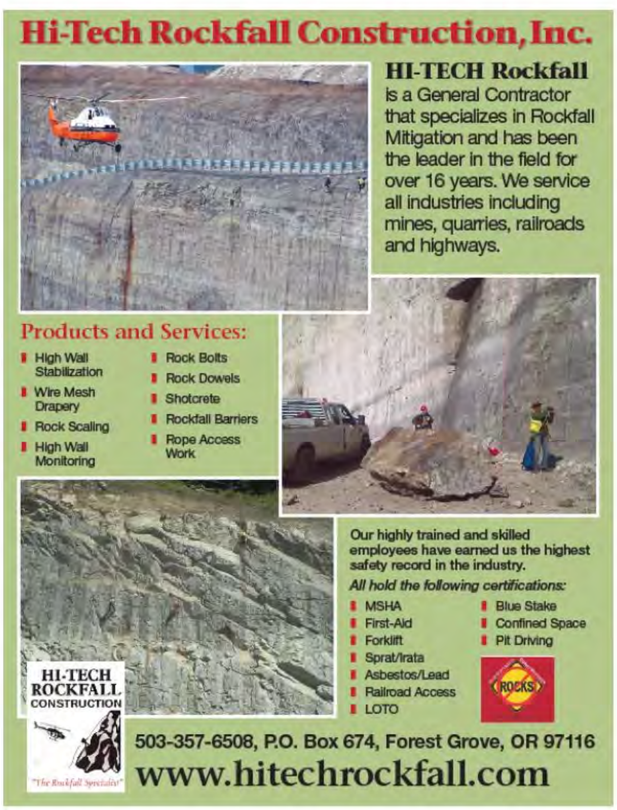

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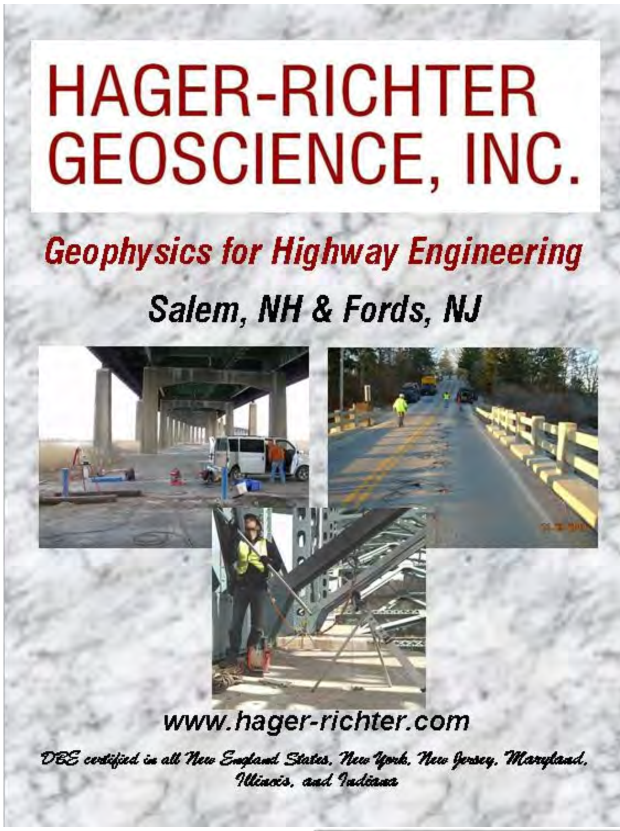
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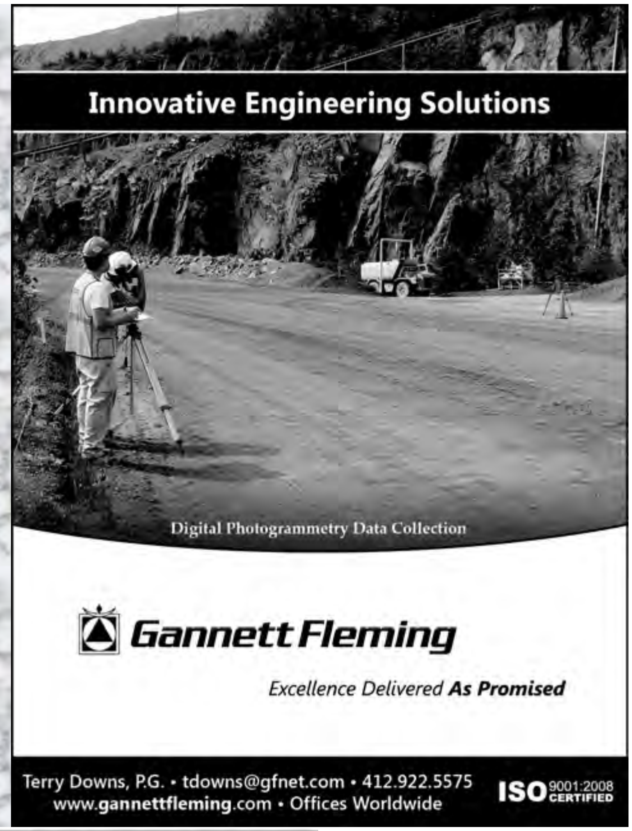
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Lower hemisphere pole plot showing kinematic analysis of the bedrock structure including potential planar, wedge and topping failure modes
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GZA GeoEnvironmental, Inc.
Geotechnical Designer
Output from CRSP, used to assess catchment for representative rockfall sizes

64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM
Exhibitor Display Locations

Booth Assignments:

1. 2014 HGS – Wyoming DOT
2. Williams Form Engineering Corp
3. SIMCO Drilling Equipment
4. Monotube Pile Corporation
5. Foundation Technologies, Inc.
6. Maccaferri, Inc.
7. TenCate Geosynthesies
8. AMS, Inc.
9. Chama Valley
10. GeoStabilization International
11 & 12. Mount Washington Observatory
13. Pacific Blasting
14. Central Mine Equipment Co.
15. Ameritech Slope Constructors, Inc.
16. Hi-Tech Rockfall
17. GZA GeoEnvironmental, Inc.
18 & 19. Trumer North America Inc.
20. Gannett Fleming Inc.
21. Hager-Richter Geoscience Inc.
22. Golder Associates, Inc.
23. Michael Baker Jr., Inc.
24. Zonge International, Inc.
25 & 26. Geobrugg
27. Bentley Systems
28. Geokon, Inc.
29. Association of State Boards of Geology (ASBOG)

64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

EXHIBITORS

Thanks to all participating exhibitors. The exhibit booths are in the Ballroom Foyer, Mt. Jefferson and Mt. Madison ballrooms.

Geobrugg North America, LLC.
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Algodones, New Mexico 87001
Phone: (505) 771 4080
Fax: (505) 771 4081
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Trumer Schutzbauten Canada Ltd
14900 Interurban Ave S, Suite 271 #19
Seattle, WA 98168
Phone: (855) 732-0325
Fax: (604)738-4080
www.trumer.cc

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Ameritech Slope Constructors, Inc.
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Phone: (828)226-6188
Fax: (828) 398-2041
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AMS, Inc.
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American Falls ID 83211
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Grand Junction, CO 81502
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Fax: (970) 245-7737
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www.maccaferri-usa.com

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EXHIBITORS

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North Conway, NH 03860
Phone: (603)-356-2137x225

Pacific Drilling and Blasting Ltd.
3183 Norland Ave
Burnaby British Colombia VSB 3A9
Phone: (604) 291-1255
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64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Agenda

Monday, September 9th, 2013

7:00 AM – 5:00 PM
Highway Geology Symposium: Registration – OPEN

12:30 PM – 4:30 PM
Transportation Research Board: Technical Session
“Site Characterization and Monitoring for Highway Engineering Problems”

6:30 PM – 8:30 PM
Ice Breaker Social
Location: Mt. Jefferson Ballroom
Sponsored by: **Hi-Tech Rockfall**

Tuesday, September 10th, 2013

6:30 AM – 9:00 AM
Continental Breakfast

6:30 AM – 5:00 PM
Highway Geology Symposium: Registration – OPEN

7:30 AM– 8:30 AM
Welcome and Opening Remarks
Krystle Pelham, Engineering Geologist, New Hampshire DOT
Christopher Clement, Commissioner New Hampshire Department of Transportation
Frederick Chormann, State Geologist, New Hampshire Geological Survey

8:00 AM – 5:00 PM
Highway Geology Symposium: Exhibitor Area – OPEN

9:00 AM – 4:00 PM
Highway Geology Symposium Spouse Field Trip
Location: Castle in the Clouds

(continued)

Tuesday, September 10th, 2013 (continued)

8:30 AM – 9:50 AM

Technical Session I Presentations – Rock Slope Case Studies
Moderator: John Szturo

9:50 AM – 10:20 AM

Morning Coffee Break
Location: Mt. Jefferson Ballroom
Sponsored by: **Gannett Fleming, Inc.**

10:20 AM – 12:00 PM

Technical Session II Presentations – Landslides
Moderator: Bob Henthorne

12:00 PM – 1:00 PM

LUNCH
Sponsored by: **Trumer North America, Inc.**
Location: Wylie's Restaurant and Tavern

1:00 PM – 2:40 PM

Technical Session III Presentations – Rockfall
Moderator: Ken Ashton

2:40 PM – 3:00 PM

Afternoon Refreshment Break
Location: Mt. Jefferson Ballroom
Sponsored by: **Gannett Fleming, Inc.**

3:00 PM –4:00 PM

Technical Session III Presentations – Rockfall (continued)
Moderator: Ken Ashton

4:00 PM – 4:20 PM

Field Trip Preview
Brian Fowler

5:00 PM

Conway Scenic Dinner Train
Depart the North Conway Grand to travel to Train Depot for a 5:30 PM Departure
Transportation Sponsored by: **GZA GeoEnvironmental, Inc.**
Drinks sponsored by: **Geokon, Inc.**

Wednesday, September 11th, 2013

6:00 AM – 7:00 AM
Continental Breakfast (to go)

6:30 AM – 7:00 AM
Highway Geology Symposium: Registration – OPEN

7:00 AM – 4:45 PM
Highway Geology Symposium: Field Trip w/Lunch
Field Trip Refreshments Sponsored by: **Golder Associates, Inc.**
Lunch Location: Cannon Mountain Peabody Base Lodge
Lunch Sponsored by: **Geobrugg**

5:30 PM – 6:30 PM
Highway Geology Symposium: Social Hour
Location: Mt. Jefferson Ballroom
Sponsored by: **Zonge International, Inc.**

6:30 PM – 9:30 PM
Highway Geology Symposium: Banquet Dinner
Keynote Speaker: Peter Crane - Mount Washington Observatory
Sponsored by: **Bentley Systems**

Thursday, 2013September 12th, 2013

6:30 AM – 9:00 AM
Continental Breakfast

6:30 AM – 12:00 PM
Highway Geology Symposium: Registration – OPEN

8:00 AM – 12:00 PM
Highway Geology Symposium: Exhibitor Hall – OPEN

7:40 AM – 10:00 AM
Technical Session IV Presentations – Geotechnical and Project Case Studies
Moderator: Krystle Pelham

10:00 AM – 10:15 AM
Morning Coffee Break
Location: Mt. Jefferson Ballroom
Sponsored by: **Gannett Fleming, Inc.**

10:15 AM – 12:35 PM
Technical Session IV Presentations – Geotechnical and Project Case Studies (cont.)
Moderator: Jay Smerekanicz

12:35 PM – 12:45 PM
Closing Remarks
Krystle Pelham

12:45 PM
Adjournment

64^{th} ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

TRANSPORTATION RESEARCH BOARD

TECHNICAL SESSION

Site Characterization and Monitoring for Highway Engineering Problems

North Conway, New Hampshire

Monday, September 9, 2013

Presided by

Benjamin Rivers
FHWA – Resource Center

Sponsored by:

AFP20 - Exploration and Classification of Earth Materials

Co-sponsored by:
AFP10 - Engineering Geology
AFP30 - Soil and Rock Properties

TRANSPORTATION RESEARCH BOARD TECHNICAL SESSION

Site Characterization and Monitoring for Highway Engineering Problems

This session will focus on case studies where excellent site exploration, characterization, data analysis and/or instrumentation monitoring programs have been used to solve engineering geological problems.

AGENDA for Monday, September 9th, 2013

Exploration Case Studies: **12:30-2:00 pm**

Test Embankment, Newington Dover 11238-Q, Joseph Blair, New Hampshire Department of Transportation

I-20 Mississippi River Bridge in Vicksburg, Mississippi, Megan Bourgeois and Robert Werner, Ardman and Associates.

Using Sonic Drilling Methods to Speed Installation of Instrumentation and Provide Continuous Sampling for Evaluating a Landslide in Northern Vermont, Jeffrey Lloyd – Golder Associates, Robert Danckert – Coastal Drilling, Pete Ingraham – Golder Associates and Mark Peterson – Golder Associates

New Techniques and Monitoring Technologies: **2:00 –2:30 pm**

New Geophysical Technology for Imaging of Sinkholes in Limestone Foundations using MERIT, David Harro – Geo3Group, Sarah Kruse and H. Kiflu – Department of Geology, University of South Florida

Break: **2:30 – 3:00 pm**

New Techniques and Monitoring Technologies, continued: **3:00 – 4:00 pm**

Remote Sensing and Monitoring Efforts by the Colorado Department of Transportation to Identify Geological Hazards for Improved Asset Management, Bob Group and Ty Ortiz – Colorado Department of Transportation, Mark Vessely – Shannon and Wilson, Fransisco Gomez – University of Missouri, Ken Fergason – AMEC, Fulvio Tonon - Engineering, Measurement and Testing

Real Time Remote Data Acquisition Technology for Landslide/Rockslide Observation and Mitigation – Bill Phillips, Silent Solutions Security

Techniques from Other Project Types with Applications to Highway Construction: **4:00 – 4:30 pm**

Templeton Gap Floodway Levees Investigation and Mitigation of Mine Subsidence, Kannan Hanna – ZAPATA Engineering

Pioneer Mountain –Eddyville Project, George Machan – Landslide Technology

64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

September 11th, 2013 - Banquet Keynote Speaker

Peter Crane, Mount Washington Observatory

Peter Crane is the Curator of the Mount Washington Observatory's Gladys Brooks Memorial Library, whose collections include books, maps, prints, photos, and other historical and scientific material relating to the Observatory, Mount Washington, and the White Mountains.

Peter has lived in the White Mountains for more than thirty-five years, and served the U.S. Forest Service and the Appalachian Mountain Club in various roles before coming to work for the Observatory in 1988. He began his Observatory career as a weather observer atop Mount Washington, and later focused on the Observatory's educational activities based at the North Conway Weather Discovery Center. He served for several years as Director of Programs before transitioning to his current position. Peter did his undergraduate work at Harvard College, and earned his doctoral degree from the University of Pennsylvania. An avid year-round hiker, he is a volunteer trail maintainer for the A.M.C., a member of Androscoggin Valley Search and Rescue, and serves on the board of the New Hampshire Outdoor Council.

Breaking the Ice on Mt. Washington: How Two Nineteenth Century Geologists Left Their Mark on The Mountain

After working together at in the New Hampshire Geological Survey, Charles Hitchcock and Joshua Huntington teamed up for the first occupation of the summit of Mount Washington for scientific purposes. Their plan was to live atop the mountain for a full winter, observing its meteorology. Despite many logistical challenges, they installed a scientific crew atop the mountain in the winter of 1870-71. Huntington was crew leader and Hitchcock was liaison at nearby Dartmouth College. Their successful effort led to reoccupation by the U.S. Army Signal Service from 1871 to 1892, and then by the Mount Washington Observatory from 1932 to the present. Today the Observatory, an independent, membership-supported, non-profit organization, is a vibrant scientific institution and an integral part of the unique public-private cooperative "community" on the summit of Mt. Washington. Its world-wide reputation for documenting the "World's Worst Weather" and for conducting diverse and related research have made it and the Mountain unique in the fields of environmental science and a tribute to Hitchcock and Huntington. The "community" on Mt. Washington has grown from the early 19th Century into the modern group of facilities there today, and it continues to do so to accommodate the ever-growing demand for activities and visits to the summit of Mt. Washington.

TOUR THE WORLD FAMOUS
Mount Washington Observatory

Go behind the scenes of this famous mountaintop weather station and get a first-hand look at what it's like to live and work atop Mt. Washington at 6,288 feet, the highest mountain peak east of the Mississippi and north of the Carolina's. These guided tours allow you to meet and interact with the Observatory's scientists, learn how they monitor the Mountain's unique weather, how they create forecasts, and what specially-designed instrumentation they use to document the Mountain's legendary weather extremes.

Reservations

The Observatory is a working weather station, so advanced reservations are required for tours. Weather and logistics on Mt. Washington permitting, tours for the 64th Highway Geology Symposium can be conducted on Monday, 9/9, Tuesday, 9/10, or Thursday, 9/12 in either the morning or afternoon. Spots on these tours are available at $5.00 per person. A minimum of 8 persons is required for a tour with a maximum of 14 persons on a tour. To reserve space or to form a tour group, please visit the Mount Washington Observatory's Exhibit Booth here at the meeting.

Tour participants are responsible for arriving at the Observatory on Mt. Washington at the designated time for their tour. There are 3 choices available for getting there. From the east side of Mt. Washington, from the "The Glen" just north of the height of land in Pinkham Notch (NH 16), tickets can be purchased to drive personal vehicles up the Mt. Washington Auto Road (allow 1 hour) or to ride up the Road in a chauffer-driven van (allow 30-45 minutes). From the west side of Mt. Washington, from Bretton Woods (US 302), tickets can be purchased to ride the also World-famous Mt. Washington Cog Railway (allow 1 hour). For planning and scheduling purposes, both of these transportation alternatives to the base of the Mountain are located about an hour away from the Hotel. Thus, please allow about 4-5 hours for a round-trip tour.

Directions on Mt. Washington

The Mount Washington Observatory is located inside the Sherman Adams Building on the summit of Mt. Washington. Please check-in at least five minutes before you tour's time at the Mount Washington Museum, located downstairs of the Sherman Adams Building (signs). This is also the location where you will pay for your tour. The Observatory's Museum attendant will inform you of where to meet the Observatory staff member who will lead your tour.

More Information

Further information is available at the Mount Washington Observatory Exhibit Booth here at the meeting.

64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Abstracts & Notes

North Conway, New Hampshire

September 9th – 12th, 2013

Hosted By:

The New Hampshire Department of Transportation

64th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Abstracts & Notes

Tuesday, September 10th, 2013

Tuesday, September 10th, 2013 (continued)

Thursday, September 12th, 2013

Thursday, September 12th, 2013 (continued)

1. Squeezing Between Rock Cuts, The Route 128/I-95 Add-A-Lane Project Dedham/Westwood, Massachusetts

Andrew R. Blaisdell, P.E.
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Christopher L. Snow, P.E.
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ABSTRACT

In the 1950s the new Route 128/I-95 corridor was cut 10 to 50 feet into the granite bedrock. Over time, a lane was added along the shoulder. Now another is being constructed along the median. The result is tall rock cut slopes with narrow catchments. When this tight squeeze became apparent to the owner, GZA was asked to meet with project team and consider alternatives for rock slope stabilization and rockfall catchment.

Initial observations revealed several areas at risk for rock fall into the travelway. A graduated approach was developed with limited field measurements and chart-based catchment evaluations for the lower risk slopes; and more detailed field mapping, kinematic analysis and computer-based catchment evaluation for the higher risk slopes.

Over 1,400 discontinuities were mapped on nine rock exposures covering approximately 4,000 linear feet of cut. The field data were gathered via a GIS-based application on a tablet computer, tied to GPS locations, imported directly to spreadsheets, then directly into software to create stereographic projections, expediting kinematic and CRSP catchment evaluations.

The team focused on unstable planes and wedges, and on irregularities that could serve as launch points for fallen rock. Scaling and local stabilization were recommended to mitigate the launch points and unstable areas along the median. Given the narrow width, permanent barriers were evaluated and recommended along the outside shoulders. Scaling is scheduled to begin in summer 2013. Observations and measurements during scaling will be used by GZA to design localized stabilization measures, and update CRSP analyses of the launch points.

Notes

2. Fort Ann Rockfall and Emergency Repair Contract

Douglas Hadjin
New York State Department of Transportation
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(518)-457-4728
Doug.Hadjin@dot.ny.gov

Matthew Balmer
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ABSTRACT

On the morning of October 15, 2012 a massive rockfall buried both lanes of US Route 4 in Fort Ann, New York, near the Vermont border. After it was reported that a car might be buried under the rock pile, a State Police helicopter equipped with thermal imaging equipment was brought in and it was determined that there was no vehicle present. Cleanup operations commenced and over the next 48 hours, the New York State Department of Transportation's (NYSDOT) emergency contractor removed over 1,700 cubic yards of fallen material. Engineering Geologists from the NYSDOT evaluated the remaining rockslope and determined that the slope was stable enough to temporarily reopen the road. However, they also recommended that the slope be remediated quickly and not be allowed to go through another winter of destabilizing freeze thaw cycles and high groundwater conditions. Due to the tight time frame, it was decided to remediate the slope under the existing emergency contract.

For the first time at NYSDOT, airborne and terrestrial LiDAR (Light Detection And Ranging) were combined with traditional ground survey and photogrammetric mapping to create a DTM (Digital Terrain Model) of the rockslope and the surrounding area. This DTM proved invaluable in the design of the remediation of the slope and for estimating rock removal quantities for the immediate cleanup and new rockslope construction. This paper will discuss the rockfall and the design and construction of the new slope under the emergency contract.

Notes

3. Idaho Transportation Department (IDT): Emergency Rockfall Assessment, US-95, Riggins, ID, December 2012

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ABSTRACT

During the evening of 2 December, 2012, a large rockslide occurred that blocked US-95 at MP 188 about 5 miles south of Riggins, Idaho. No accidents were reported as a result of the rockslide. The next day, Idaho Transportation Department (ITD) noted a large, unstable, 200-ton block, hanging precariously on the face about 180 feet above the road that required immediate assessment.

The rockslide originated from an ultramafic rock massif cropping out about 250 feet above the road. The team employed rappelling techniques to map vertical scanlines and access the unstable block. While on rappel, the team collected physical and engineering characteristics of the rock mass including information on discontinuities. The field data were used to assess the kinematic relationships between the structure of the rock mass and the rockslope face and establish failure mechanisms. During the assessment, a large and deep tension fracture was observed between the critical block and the hanging wall of the main rock mass. The kinematic analysis demonstrated that the critical block and slope were unstable and wedge failures were dominant.

To mitigate the initial, unstable, slope conditions and critical block, a rock scaling contractor was immediately mobilized to the site. After assessment, the team established that the block should be removed by trim blasting. The team developed a trim blasting design that would bring the block down yet preserve the back wall. They worked closely with the blaster-in-charge and contractor to drill, load, and shoot the critical block. The block was removed safely by presplit blasting on 23 December. US-95 highway was then reopened to the public to accommodate the Christmas traffic.

Notes

4. Case Studies on Rockfall Mitigation and Rock Slope Stabilization in California, Tennessee, Virginia, and Vermont

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ABSTRACT

Rockfall constitutes a major hazard along our nation's roadways and a nagging liability to our maintenance and engineering departments. Recurring cleanup and repair costs have stressed dwindling maintenance budgets. The extensive nature of the problem precludes repairing and mitigating every possible site, but new and innovative mitigation technologies and contracting techniques can serve to stretch tight budgets.

There are many methods that can be used to stabilize a rock slope. These include altering the slope geometry, installing drains, adding reinforcement, or a combination of these methods. The challenge for engineers is to design a method that can be installed with little or no impact to the traveling public, is expedited through innovative contracting methods, limits the disturbance to environmentally sensitive areas, and maintains an aesthetically pleasant appearance and appropriate service life.

This presentation covers four case studies that highlight innovative technology and innovative contracting methods for rockfall mitigation. The case studies include a project for the United States Army Corps of Engineers near Chowchilla, CA using design/build/warranty contracting and post-tensioned rock bolts with Maccaferri B600 mesh facing; a project for the Tennessee Department of Transportation near Maryville, TN using rock dowels and Geobrugg's high-capacity Tecco® mesh facing; a design/build project for the Virginia Department of Transportation near Hillsville, VA using a shear dowel array encased in reinforced shotcrete overlying a drilled drainage array; and an emergency design/build rockfall mitigation project for the Vermont Agency of Transportation that used a shear key, scaling, vegetation removal, rock dowels, and both wet and dry mix shotcrete.

Notes

5. Slope Failure Investigation and Remediation using Geosynthetic Reinforced Earth atop Poor Foundation Soil

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ABSTRACT

This Zoar Valley Road site in Erie County, New York, is approximately 925 feet north of, and 250 feet above the elevation of Cattaraugus Creek. The topography between Zoar Valley Road and Cattaraugus Creek exhibits an irregular slope leading from the edge of pavement down to Cattaraugus Creek, including shallow rotational slumps, displaced soils that have slid down the slope, flat benches, and small actively-eroding channels. A relatively flat plateau extends approximately 1,200 feet northeast from the site before the topography begins to rise again. These site conditions confirmed surficial geologic mapping compiled in 1979, suggesting that the Cattaraugus Creek Corridor is rimmed within a morphogenetic region referred to as landslides and slumps.

Design studies revealed that uncontrolled stormwater was the primary cause for the road failure; two culverts discharged onto steep and easily erodible soils adjacent to the road. In addition, the subsurface investigation indicated that elevated pore-water-pressure in a silt layer further exacerbated the failure. The road had, up until that point, been maintained within the right-of-way using unsuitable fill. The remedial design included a Geosynthetic Reinforced Earth Slope (GRES) to reestablish the road near its original grade within the right-of-way, and subsurface drainage improvements to relieve the high pore-water-pressure in the GRES foundation soil. During construction, we used vibrating wire piezometers to monitor the pore-pressures in the foundation soil. Stormwater is conveyed down the slope in a pipe-slope-drain to a non-erosive outlet at an elevation 100 feet below the road.

Notes

6. A Bump in The Road – Remediation of the SR 87 Landslide

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ABSTRACT

An ancient landslide along SR 87 in northern Pennsylvania re-activated in 2011 after the toe of the hillside was eroded due to flooding of the North Branch of the Mehoopany Creek during Hurricane Irene and Tropical Storm Lee. Slope movement damaged the SR 87 roadway creating a large bump, tension cracks, and rough roadway conditions throughout the landslide area. Published literature indicated that the project area was underlain by a glaciolacustrine deposit from the Pleistocene Age. An extensive subsurface exploration program, consisting of 33 borings and 6 test pits, was performed to determine subsurface conditions at the project site. Comprehensive laboratory testing was performed on soil samples collected during the subsurface exploration to estimate engineering properties of the glaciolacustrine material. Laboratory strength tests included direct shear with residual measurements and triaxial shear. Inclinometers were installed in eighteen borings and piezometers were constructed in nine borings. The Pennsylvania Department of Environmental Protection identified an exceptional value wetland within the upper portion of the landslide and preferred that the landslide remediation not affect the wetland. Of the multiple remediation alternatives considered, the selected alternative preserved the exceptional value wetland and included relocation of the creek, construction of a soil berm at the toe, and reconstruction of SR 87 along the roadway's existing alignment. Unique aspects of the project included varying varve orientations and thicknesses within the glaciolacustrine material. Inclinometer and piezometer readings continue to be obtained to monitor the area.

Notes

7. Surface kinematics of the Ferguson rock slide revealed by repeat lidar and GPS measurements, Highway 140, California

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ABSTRACT

High-resolution topographic data, such as that collected using lidar ("light detection and ranging"), allow examination of the complex morphology of landslide masses. When these data are collected repeatedly over temporally significant time intervals (i.e., days to years), the kinematics of slide motion can be extracted. This information can guide assessments of expected future deformation, and in turn assist hazard and risk assessments as well as steer the design of potential mitigation options. Here, we examine the motion of a large (approximately 800,000 m^3) rock block slide reactivation located in northern California. The Ferguson rock slide moved during the particularly wet spring of 2006 in an area of prehistoric instability as evidenced by multiple headscarps in the upper portion of the slope. The landslide is located on one side of the narrow Merced River canyon where both the river, nationally designated as Wild and Scenic, and California State Highway 140 share the canyon bottom. The 2006 reactivation caused a 3-month closure to this section of the highway, which receives about 875,000 vehicle trips per year and serves as the main all-weather entrance to the iconic and heavily visited Yosemite National Park. As of summer 2013, talus from the landslide still blocked the original roadway and traffic used a one-lane temporary road to detour around the closure.

We present surface and cross-section deformation analyses of the landslide surface using a total of four high-resolution terrestrial lidar data sets collected at approximately two-year intervals following the landslide reactivation. We couple these data sets with differential GPS data collected semi-continuously at three locations on the landslide surface during approximately this same time interval (late-2006 to late 2012) to examine patterns of motion within the slide. Our results provide a more complete understanding of the complex interactions between the upper, driving part of the landslide and the conveyor belt pathway that creates and deposits talus on the original roadway and into the river. Overall, we find that rock slide motion is mostly translational, and it moves at higher velocity in its middle and lower areas compared to the upper blocks. However, we also find that overall velocities have decreased over the 6-year period of investigation. This case study illustrates the use of repeat high-resolution topography for guiding hazard assessments related to ongoing motion of large landslides.

Notes

8. Remediation of an Active Landslide within a Prehistoric Landslide – SR 2065 Thompson Run Road, Monroeville, Pennsylvania

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ABSTRACT

The primary goal of the SR 2065 Thompson Run Road landslide remediation project was to stabilize the roadway without triggering movement along potentially healed prehistoric landslide failure surfaces. Groundwater trending through weak claystone and thick colluvial slopes above the active landslide presented formidable remediation design challenges. Treatment limitations extended beyond the site geology to include: slope geometry; existing and required right-of-way constraints; railroad right-of-way access restrictions; and the inability to encroach upon, or alter the course of, the Thompson Run stream. Gannett Fleming was tasked by the Pennsylvania Department of Transportation (PennDOT) with providing permissible treatment alternatives and a preferred conceptual remedial design for solicitation of bids from Design/Build contractors. The preferred conceptual design consisted of a caisson supported slope with reconstructed rip-rap embankment. Permissible treatment alternatives included roadway excavation and replacement with stabilized material or a soil nail slope treatment. Adequate roadway stabilization, site geology, right-of-way concerns, and stream encroachment/course alteration were all addressed by the preferred conceptual design. The caissons were designed to key into the Grafton Sandstone/Shale to provide stability for the active landslide while perforating potential prehistoric landslide failure surfaces. A steepened rip-rap embankment allowed for reconstruction of the roadway embankment slope without encroaching on the stream and provided a material that would resist erosion of the slope toe.

Notes

9. Rope Access for Geotechnical Work

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ABSTRACT

In 1989 the California Department of Transportation developed a rope access-training program that includes a code of safe operating practices and a corresponding training class for rock scalers, construction inspection, rockfall mitigation system maintenance, and geologic investigations. During the last 23 years over 1,600 students have successfully completed the training. The instructors have been trained by the Yosemite Mountaineering School, American Mountain Guide Association (AMGA), and most recently the Professional Climbing Instructors Association (PCIA). The techniques used utilize a combination of industrial and recreational climbing techniques. A manual and video are used during the class but the focus of the class is training on slopes in the field. There are two formal training sites and several back up sites with various slopes configurations ranging from 1 ¼: 1 to vertical. Two classes are available: an entry-level class and a refresher class. Each climber must attend the entry-level class then periodically attend the refresher class throughout their climbing careers. Emphasis is placed on basic skills and equipment for statewide uniformity in technique and equipment. Of the 12 Regional Transportation Districts, all have trained personnel. Nine districts have scaling crews. Three districts have an annual scaling program for slope maintenance.

Notes

10. Rock Slope Protection and Rock Mechanics

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ABSTRACT

Rock slope stabilization requires (1) engineering geology/geotechnical engineering input, (2) selection of suitable protection means and (3) a means for evaluating the nature and level of protection appropriate for the particular project. The interrelationship between these requirements is presented using the example of an existing, typical hard rock slope along the entrance road to a large development. Solutions using conventional analyses and the Geobrugg SPIDER® and RUVOLUM® Online Tools were used to develop a design for rock mechanics problems on a slope.

The programs are online tools available to engineers and designers and where the user will input site conditions based upon field evaluation, be able to select anchor spacing and size and see results that are in an optimized arrangement. The programs are based on Mohr-Coulomb Equilibrium theory and it establishes the relationship between driving and stabilizing forces. The programs use a trial and error method and it is quite easy to change the input parameters. Unfortunately, the programs cannot currently analyze wedge failures. However, as an example of the procedures we will use, a wedge failure analysis that is performed in a conventional manner to provide rock discontinuity strength properties for use in the programs for a suspect, wedge-shaped body within the rock type.

Mapping, analytical and evaluation procedures are straight forward and can be used by any competent geotechnical organization charged with developing appropriate rock slope stabilization. The information collected is critical for the program. The last step in the process is installation and using a qualified and experienced rock slope remediation contractor is the best approach. The contractor should be also able to provide assistance during the project development stage.

Notes

11. Testing of Rockfall Post Foundations in Colorado

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ABSTRACT

The Colorado Department of Transportation (CDOT) recently conducted testing of different types of post foundations used to support rockfall barriers and attenuator systems in Colorado. Current testing of rockfall barrier systems typically does not involve impact testing of the posts but rather impact testing to the center of a net or panel system that transfers a portion of the loading to the post foundation. These transferred loads are a fraction of the load that would be generated from a direct impact to a post.

Based on full scale rock rolling tests in Colorado, in which posts were knocked down during a rock rolling event, it was evident that if the post and foundation system could resist at least one or two direct impacts during a multiple rockfall event, the performance of the rockfall barrier or attenuator system could be greatly increased. Additionally, understanding the failure characteristics of the post foundation system could provide insight into reducing maintenance costs and improving management practices of these systems. To determine the loading conditions and evaluate the effectiveness of various foundation designs under direct post impacts, a pendulum test site was constructed in Colorado to generate at least 220 kJ of impact energy.

The post foundation testing to be discussed in this presentation consisted of 29 direct post impacts. The testing conditions ranged from a rockfall post connected to only a base plate in contact with the ground, to a post that was attached to a 6 foot deep (1.8 m), 36 inch (0.9 m) diameter foundation. The testing also consisted of various combinations of uphill retaining anchors that were instrumented with load cells on the foundations and load cells on the retaining anchors.

Notes

12. Development of a modular brake element for the use in modern rock fall catchment fences

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ABSTRACT

Rockfall catchment fences have a long history, with their beginnings being rooted in rigid structures. Building upon experience, mitigation structures became increasingly more flexible. Eventually the modern rockfall catchment fence was born, consisting of steel posts, continuous bearing ropes that support a flexible net structure and brake elements.

Early brake elements primarily functioned by absorbing energy during an impact through friction. Support cables were lead through steel plates with several holes drilled in them. Another example of early brake elements utilized the deformation of steel to absorb energy by leading support cables through steel tubes in the shape of a ring. As a force was exerted on the cable, it tightened the ring, ultimately pulling it into a knot. A further advancement removed the support ropes from the brake element entirely and relied on the deformation of steel, for example a coil of steel that uncoils as one end is held in position and the opposite is pulled or a strip of steel that is forced through a roller to make a bend at a defined angle (e.g. 180 degrees).

The authors will present a new type of brake element that further relies on the properties of steel to absorb energy, but instead of focusing on friction or the deformation of a profile, it harnesses the controlled failure of steel.

Notes

13. ONR 24810 – A comprehensive guideline for building better rockfall protection structures

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ABSTRACT

The influence of the ETAG 27 Guideline for European Technical Approval of Falling Rock Protection Kits, published in 2008, has been relatively far reaching, including here in North America. ETAG 27 makes it possible to compare products, from different material suppliers, through standardized reporting of testing and material data. However, it does not consider best practices for the implementation or the evaluation of safety and maintenance requirements.

A new document published by Austrian Standards Institute – the Austrian national standards body, similar to ASTM and CSA – goes beyond ETAG 27, though in a much more broad spectrum including stabilization with anchoring and mesh/nets, embankments, and galleries. The document is entitled “ONR 24810, Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance”, published in January of 2013.

Herein, the authors focus on summarizing the parts of the ONR specific to catchment fences beginning with the initial site investigation, which results in the input parameters for the numerical rockfall analysis. The semi-probabilistic verification of the design is then explained by the comparison of the impact parameters, such as energy and bounce height, with the resistance parameters of the catchment fence. Furthermore, helpful design and constructive rules regarding anchor design and fence layout are given. Lastly, maintenance and inspection schedules are presented.

Notes

14. Proof Testing of Cable Anchors in Rockfall Protection Systems

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ABSTRACT

This is a report on the testing procedures for wire rope anchors we see in rockfall mitigation systems across the US. For years the standard had been to incrementally load wire rope anchors just as if they were solid bar anchors.

Wire rope anchors react differently to loading as compared to solid bar anchors. The cable has a certain amount of natural elongation due to their construction that takes place under loading. They are superior to solid bar anchors in rockfall systems as the wire rope anchor has the ability to absorb rock impacts without the possibility of breaking due to shearing. Due to this natural elongation, the incremental loading procedures we typically see in specifications for rockfall barriers and rockfall drapery projects are not necessary and have no relevance in determining the ultimate strength and pullout capacity of the anchor.

We have found that simply just loading the anchor to the capacity required, waiting a couple of minutes for the natural elongation to take place and then reloading the wire rope anchor to the required capacity and holding at that load for a designated period of time proves the capacity of the anchor without the loading and unloading several times that is required during incremental loading procedures.

Notes

15. Rockfall Barrier Behavior on Multiple Rocks Impact

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ABSTRACT

When a barrier is impacted by rocks, multiple components of the barrier are engaged to absorb the energy generated by the falling rocks. More often than not, rockfall events generate multiple rockfalls, that impact the barrier at different intervals. Rockfall barriers are generally very difficult to design considering that most information comes from few case histories, rigorous statistical analysis, and a knowledge of mechanical behavior of the barrier structure. With regard to the behavior of the barrier, the primary information available to the designer is provided by standard testing procedures like the ETAG 27, that defines a Maximum Energy Level (MEL) and Service Energy Level (SEL) capacity for the structure. The main question for the designer is: how does one correctly synthesize the data derived from geological and topographic surveys, the probabilistic analysis of the trajectories, and knowledge of barrier characteristics into the design?

This paper outlines some practical recommendations, that help overcome the main uncertainties affecting design reliability, to foresee and compensate for installation problems, and reduce maintenance costs. The goal is optimization of rockfall barrier designs considering their Service Energy Level (SEL) and Maximum Energy Level (MEL), as well as their behavior in cases where multiple impacts occur. The selection of a Rockfall Fence Kit, designed in accordance with full scale crash tests (ETAG 027), is recommended in order to understand and incorporate the values of loads and deformations acting on and through the fence kit during impacts.

Notes

16. Use of Rockfall Rating Systems in the Design of New Slopes

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ABSTRACT

One typically uses different means to evaluate a highway rock slope depending on whether it exists currently or is in design. For example, the Rockfall Hazard Rating System (RHRS) and derivatives are commonly used to evaluate existing slopes and inform decision makers who are managing rock slope inventories. In contrast, kinematic and limit equilibrium analyses and methods based on observation and probability, such as Ritchey Ditch Criteria, Rockfall Catchment Area Design (RCAD), and the Colorado Rockfall Simulation Program (CRSP), are typically used to provide information for decision making when designing new slopes. Is there good reason for this difference? This paper raises this challenge and proposes that rating systems are not just good for existing inventories; they are good tools for design of new and rehabilitated slopes. Some of the challenges in using a rating system for design are addressed and the importance of distinguishing risk from hazard is highlighted. Finally, the paper demonstrates how rating systems can help us move towards and define a standard of practice for rock slope design in Colorado and other mountainous environments, and it discusses the challenge of establishing and applying an appropriate standard.

Notes

17. The Engineering Geologist and Transportation

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ABSTRACT

Transportation engineering geologists are called on to perform various duties for a public agency or consulting firm. Traditionally, many of these groups were named "Soils and Geology" units and were staffed by personnel with an engineering geology background. The geotechnical branch of civil engineering gained strength during the 1970's and now many of the groups are staffed by both engineering geologists and geotechnical engineers. The tasks and responsibilities between the two professions are sometimes blurred.

The responsibilities of Engineering Geologists within the transportation industry vary as widely as the geology of the 50 States. Their principal responsibilities include exploration and classification of earth materials, geologic mapping, geomorphology, geologic hazard identification, groundwater, geologic processes, rock discontinuity characterization. Problems can arise when engineers with little or no background or education in geology perform these tasks. Many geotechnical engineers have never had a university level course in geology.

Transportation engineering geologists should have a role in the planning or NEPA process, identification of geologic hazards, route selection, bridge foundations, subsurface characterization and location of materials, slopes, especially rock slopes.

Highway engineering involves many aspects of geology. Applying the principals of geology should make for less risk during construction and better, longer lasting, trouble free highways. The tasks an engineering geologist performs in highway engineering should be better defined given the evolution of the practice.

Notes

18. New Design Software for Rockfall Simple Drapery Systems

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ABSTRACT

Rockfall drapery systems are commonly used as simple, fast and economical measures to control rockfall trajectories on very steep slopes. The systems basically consist of a steel mesh attached at the slope crest with a longitudinal cable fixed by means of a suitable number of ground anchors. The effect of this kind of intervention is to control the trajectory of falling rocks, which then fall to the bottom of the slope with a with slower velocity, or are stabilized in place. They can be used on any kind of slope to protect sensitive targets in the mining industry, roads and railways, and inhabited areas.

The falling blocks, typically smaller than 0.6 - 1.0 m in diameter, pile up into a trench (or into a "pocket of mesh") at the bottom. In comparison to other types of rockfall protection measures, the simple drapery is cheaper, and its maintenance is easier. On the other hand, it cannot be considered a remedy for shallow instability because it can only control the trajectories of falling rocks and facilitate their collection at the slope toe.

The design of simple draperies requires the analysis of several factors such as slope features (height, gradient, morphology), the geological and dynamic features (nature of the ground or rock, type of instability, erosion problem, blocks size), the environmental condition (presence of vegetation, aesthetic concerns), the installation problem (access to the slope, safety for the workers, safety for the surrounding areas) and finally the performance required (temporary or permanent intervention, required maintenance, cost). Finally the most problematic design-step is the choice of a suitable mesh, the top longitudinal cable, and the top anchor type. Because of the highly variable nature of rockfall behavior, these structures cannot be standardized - they have to be analyzed and designed for each application.

Maccaferri has developed a new software application (MacRO 2) with a practical tool to define the mesh and the related supporting structure consisting of up-slope cables and anchors.

The software, based on an approach proposed by Muhunthan B. et al. (2005), allows designers to size the top longitudinal cable, the anchors, and select the appropriate mesh drapery and establish for maintenance procedures. Even if the method seems quite simple and rough, it is effective and lets the designer correctly select drape materials and the geometry to be used on the systems. This paper analyzes the conditions for a simple drapery installation, the main steps used for the calculations, and presents a case study at a Mine in the U.S. Nevertheless, even if the software allows for a quick and simple calculation approach, onsite observations are always recommended in order to achieve a good design, with the ultimate goal of protecting property and human lives.

Notes

19. Maud Farm Road Investigation

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ABSTRACT

This paper presents the findings of an extensive site investigation into the causes of embankment settlement over five flexible pipe culverts ranging respectively in 36, 24, 36, 36, and twin 48 inch diameter. The site location is on an east−west County Road 131 near Maud, Oklahoma. The project was developed and designed by the Bureau of Indian Affairs (BIA). The issue here concerned a lawsuit brought by the BIA against the contractor in which the BIA wanted to know if the contractor could be held libel for the embankment settlement. At stake was contact retainer held by the BIA in the sum of $358,000 against the contactor.

The site geology consists of very shallow alluvial soils and/ or residual soil underlain predominately sandstone and sandstone and interbedded shale in the narrow drainways. The embankment was constructed from roadway cut sections containing residual sandy and clayey soils underlain by sandstones and sandstone interbedded with shale. The site landscape is one of shallow rolling hills.

The field investigation consisted of a total of 15 piezocone soundings at the site. Soil properties of the embankment material, the underlying shallow alluvial and/or residual soil, and underlying geology were inferred from the piezocone tip resistance (q_c) and friction ratio (R_f). Three piezocone soundings were made in a staggered pattern at each of the five pipe locations in as close a proximity to the pipe centerline as possible.

The analysis used software for the analysis of buried structures, Cande−2007 Update Release 7/31/2011, Version 1.0.0.7. This software uses a finite element mesh analysis. A detailed analysis revealed that the settlement at each pipe location was due to deformation below the pipe grades. The piezocone tip resistances in the embankment indicated a very stiff material and did not support the BIA claim that the contractor was responsible for the subsidence above the pipe culverts. The analysis showed that the settlements were the result of vertical pressure against a yielding base, a concept borrowed from theoretical soil mechanics.

Notes

20. Emergency Slope Stabilization, Catskill Creek Bridge, New York State Thruway, Catskill, New York

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ABSTRACT

In response to flooding/ scour damage from Tropical Storm Irene in late August 2011, the New York State Thruway Authority and Golder prepared emergency slope mitigation designs for two slope failure areas in the southern embankment beneath the Catskill Creek Bridge on I-87 south of Albany. Regional catastrophic flooding occurred in the region on August 28, 2011 from Tropical Storm Irene. Based on United States Geological Survey (USGS) stream gauging data, the water level in Catskill Creek rose at least 25 feet during flooding from Irene. Following this event, Thruway personnel inspected the bridge foundations, and discovered recent scour of embankment fill, riprap and other soils surrounding the piers north and south of the streambed. The scour included loss of riprap and soils adjacent to the east footing of Pier 3 on the south side of the northbound truss. The scour compromised the pier foundation as well as a large portion of the slope supporting the southeast approach of the northbound structure.

Shortly after discovering the damage, site visits were conducted to initially evaluate the scour damage adjacent to the pier and collect site geologic/geotechnical field data. During one site visit, a larger landslide failure surface was noted, along with tension cracks at the head of the southern bridge approach embankment. To evaluate potential mitigation approaches, the project team reviewed site geology and geotechnical conditions using the original highway/bridge design borings; conducted back-analysis of the failure modes to estimate geotechnical conditions; developed conceptual slope mitigation concepts, inclusive of the Thruway's design for oversize riprap for scour mitigation; developed a soil nail – tensioned mesh system to retain both soil scour areas and the toe of a riprap repaired slope (used only in areas where a stable riprap slope design could not be used to avoid encroaching on the stream channel); developed special provisions; and prepared a design report. Mitigation construction was conducted between November 2011 and May 2012.

Notes

21. PDA and Pile Restrike; A Better Understanding of Pile Resistances

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ABSTRACT

As KDOT continues to move forward using Load and Resistance Factor Design (LRFD) the utilization of high-strain dynamic pile testing is a fundamental step in generating our geotechnical recommendations. By implementing a PDA (Pile Driving Analyzer) during high-strain testing KDOT geologists and engineers have more confidence in the recommended bearing resistances. The goal for KDOT is to better understand pile resistances in various geologic settings to aid in reducing costs, reduce pile sizes and increase the loads needed to meet LRFD standards.

The current practice for PDA testing is to monitor piling to end of initial drive (EOID), and then perform short and long term restrikes. This current testing method has allowed KDOT geology to verify pile design resistances, and short and long term setup gains. Ultimately, KDOT anticipates establishing a new modifier for the ENR formula based upon data collected from PDA's and pile restrikes.

KDOT will utilize that PDA and restrike data in the design phase of future projects, thus taking advantage of the soil setup, reducing pile sizes, increase design recommendations to measured pile capacities, eliminate pile overruns, and expedite pile installation.

Notes

22. Cooperative Geotechnical Designs to Build on Liquefiable and Compressible Soil in Salem, Massachusetts

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ABSTRACT

The Massachusetts Bay Transit Authority (MBTA) is addressing accessibility throughout their facilities. The Commuter Rail station in Salem, Massachusetts is upgrading their facility to improve site accessibility and increase parking capacity. Proposed improvements include a parking garage replacing the existing parking lot, a pedestrian bridge replacing the existing stairway connecting track level with downtown Salem, and a full-length high-level platform. Historical records, a geophysical survey, and an archaeological survey indicate structural remains from an historic train depot are largely intact beneath the surface of the existing lot.

Subsurface explorations encountered fill overlying loose saturated sands above 40 feet of soft, compressible marine clay deposits extending to competent argillite rock at 60-80 feet below grade. Deep foundations bearing on rock were recommended for structural support of the garage, bridge, and platform. Potentially liquefiable sands, the potential for lateral spreading, and a poor seismic site classification exist at the site. Ground improvement techniques were recommended to improve the subsurface soil conditions and limit liquefaction and lateral spread potential. Several value-engineering options were explored, including options to replace traditional deep foundations with drilled displacement columns for garage support, using shallow retaining wall foundations for platform support, and using a slab-on-grade instead of a structural slab. The resulting cooperative designs required additional coordination between the design team to maximize efficiency of the project budget.

Notes

23. Cellular Geosynthetics in Highway Applications

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ABSTRACT

Expanded polystyrene (EPS) is a closed-cell, polymeric ('plastic') foam. It was invented circa 1950 and is now a commodity material that is manufactured worldwide for numerous, diverse commercial applications. In its generic block-molded product form (EPS-block), it is the geofoam material and product of choice as lightweight fill for earthwork construction such as highway embankments on soft ground. It has been used for this geosynthetic-functional application for over 40 years since the first documented project in Norway in 1972. This mature, well-established geotechnology is now widely known and used worldwide, with exponential growth occurring throughout the U.S. and Canada during the past 20 years.

However, there are many other potential functional applications and uses of not only EPS-block geofoam but a broader range of cellular-geosynthetic (geofoam and geocomb) materials and products in highway-related applications that are less well known and used to date. This paper highlights these lesser-known capabilities of cellular geosynthetics that have already been used and proven in practice and may be of interest to geo-professionals involved in transportation-related projects. Also presented in this paper are highlights of new developments related to the well-known and established uses of cellular geosynthetics such as the use of EPS-block geofoam for soft-ground applications.

Particular topics of relevance and interest addressed in this paper include presentations and discussions of:

- results from the latest National Cooperative Highway Research Program (NCHRP)-funded research into broader uses of EPS-block geofoam in slope stabilization, not limited to soft-ground conditions. This research included development of an updated version of the first-of-its-kind material and construction standard developed a decade earlier as part of the original NCHRP-funded research into embankments on soft ground
- reduction of lateral earth pressures behind both new and existing earth-retaining structures of all kinds, e.g. free-standing retaining walls, conventional jointed-bridge abutments, and integral and semi-integral bridge abutments

- compressible inclusions to reduce both vertical and horizontal stresses on structures from expansive soil and rock
- control of seasonal ground freezing beneath pavements and behind earth-retaining structures
- protection of rock and snow sheds from slide and other falling debris
- important issues concerning failures in project applications; manufacturing and construction quality; and material standards and generic construction specifications that have emerged as hot-button issues throughout the U.S. in particular in recent years.

Notes

24. Corridor Management: Capturing Geotechnical Impacts on Highway System Performance

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ABSTRACT

Risk-based transportation asset management plans are required under new performance-driven legislation. Bridges and pavements are required within these plans, and the inclusion of other assets is encouraged. One could argue that the primary assets of a transportation agency are the transportation corridors that have been established to provide means for moving people and goods safely and efficiently. A corridor's performance in this regard is only as good as its weakest link. Therefore, the way an agency can manage an asset, such as a corridor, to a standard for system performance, is to consider its components concurrently, not by individual asset classes. A corridor has embankments, slopes, walls, bridges, and pavements, and considering these geotechnical features separately does not make sense from a system performance perspective. Settlement, slope instability, rockfall, erosion and corrosion are events which can be surprising, or recognized in advance and managed. The corridor concept can bring geotechnical assets into consideration and result in better management for system performance. It also provides a means for rational prioritization that allows for a phased approach to the daunting task of collecting inventory and condition assessment for features that have not previously been managed. Geo-professionals are developing tools and practices for inventorying, assessing performance, predicting life-cycle costs and degradation, and evaluating risk associated with geotechnical features. These tools and practices will contribute to effective corridor management.

Notes

25. Four Times the Effort: Big Blue River Bridge Project

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ABSTRACT

Projects for the Kansas Department of Transportation, Geotech Section are usually pretty straight forward. The geology section has a great working relationship with our design squads. Design changes are usually minor, such as small alignment corrections, right-of-way needs, or minor adjustments to a bridge span. However, one particular project was not that simple, the Big Blue River Bridge replacement and realignment of US-77 highway. The geologic setting is the Flint Hills Region of Kansas with approximately 200 feet of topographical relief and an extensive gypsum mining operation.

This project went through 4 alignment changes. Some of these changes moved the roadway as much as ¾ of a mile, others only a couple hundred feet. The Geotech Section was given 3 months to complete the investigation. After completion of the field work the alignment changed to eliminate an 80 foot rock cut slope. Other alignment shifts were put into place, always after the field work had been started. The final alignment shift was begun by a local landowner. He had a better plan than our design squad.

What started out as a simple project now had consumed 1 year of field time, involved numerous design revisions, had major utility impacts and resulted in alterations to two Kansas highway alignments.

Notes

26. Enhanced Geotechnical Site Investigation of Presumpscot Clay by Cone Penetration Testing

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ABSTRACT

"Presumpscot Formation" designates the spatially variable fine-grained, glacial marine sediments deposited within Maine's coastal and inland area previously submerged by the sea. Subsurface investigations (SI's) of geotechnical design parameters in Presumpscot clay are mainly based on interpretations of data from in situ vane shear tests of variable quality and classification and index testing on disturbed split spoon samples collected at discrete intervals within borings. SI's may be augmented with undisturbed thin-walled tube samples for laboratory determination of more reliable engineering parameters, however cost and sample quality concerns often limit this option. Consequently, engineering characterization of Presumpscot clay relies heavily on empirical correlations and engineering judgment, creating the need to manage uncertainty and risk though greater conservatism and higher safety factors. Improvement in quantity and reliability of SI information is therefore key to reducing uncertainty and enhancing foundation design.

This paper describes an investigation of the use of cone penetration testing (CPT) as a reliable in situ characterization tool for the Presumpscot clay at two proposed bridge replacement sites in Maine. CPT data are compared and correlated with routine geotechnical investigation data (i.e. split spoon and field vane data) as well as to results of index and advanced laboratory strength and compressibility testing on high quality undisturbed thin-walled tube samples. This represents an effort between MaineDOT and University of Maine to innovate site investigation for Presumpscot clay to meet LRFD design requirements and reduce subsurface uncertainties and associated risk and cost to foundation design and construction.

Notes

27. Digital Terrain Modeling Techniques for a Better Subsurface Soil Layers Representation

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ABSTRACT

Geotechnical engineers due to the nature of their work have to deal with large extensions of terrain as they are trying to model the ground, but using a geotechnical perspective. Vast information is available from government agencies in the form of electronic quad maps, satellite images and elevation models. Unfortunately, this information is not accurate enough when details of the terrain are needed.

On the other hand, surveyors provide a wealth of data to optimize the civil aspect of capital projects, such as electronic ground data in the form of elevations, ground features, terrain configuration, etc. that taken for further processing give the engineers the ability to see a graphical representation of the working site in their computer monitors.

The purpose of this paper is to explore available software techniques that could be used to analyze the data given and interacting with a geotechnical database be able to model a better representation of ground and subsurface conditions in our projects. This paper discusses the different methodologies used to take ground information and thereafter create a proper DTM model of the surface conditions. A Geotechnical database needs also to be properly configured in order to interact with the ground information and depending of the amount data collected we can create an accurate representation of the soil layers in an electronic format, rather than creating soil profiles, interpolating between them and manual connecting the soil layers in a graphical borelog profile report.

A case study will be discussed in which after loading data from the web and civil survey, geotechnical boreholes were performed mapping the proximity of a bedrock layer. Then, using civil software and 3D modeling techniques a subsurface ground model was developed and further analyzed to find the areas in which the bedrock layer was closest to the surface. A model of isopach contours was develop and then plotted in a CADD environment.

Notes

28. On-line Geotechnical Database Considerations and Data Sharing

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ABSTRACT

Geotechnical data for a project can come from two primary sources: a general, geologic review of the area as well as site-specific investigations.

There are many ways to provide data to optimize the civil aspect of capital projects. Today's digital data (e.g., elevations, ground features, terrain configuration, LiDAR profiles, satellite pictures) with further processing give engineers the ability to see a graphical representation of the working site in their computer monitors. Many of the sources to get detailed information about the project vicinity are widely available, but not project specific.

Gaining insight into subsurface conditions is done on a project-by-project basis via geophysical methods, cone penetrometer testing (CPT), dilatometer testing (DMT), and standard borehole explorations. Linking information from separate projects in the same area is rarely done. This is, in part, because by traditional work methods, data exchange is not possible as proper software tools are not available.

Organizations utilizing a robust geotechnical database are able to use general project data as well as information from projects in the vicinity to quickly and easily gather valuable information with minimal work time.

This presentation will review two state-supported online geotechnical databases, and review technical components, development methods and system considerations that Minnesota DOT and Virginia DOT have encountered during their on-line database implementation, as well as current capabilities of their systems.

Lessons learned and benefits expected will be reviewed. Future possibilities as technology advances and becomes more accessible to organizations will also be discussed.

Notes

29. Validation of Interferometric Synthetic Aperture Radar as a Tool for Identification of Geohazards and At-Risk Transportation Infrastructure

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ABSTRACT

As part of the USDOT-funded research program RITA-RS-11-H-UVA, "Sinkhole Detection and Bridge/Landslide Monitoring for Transportation Infrastructure by Automated Analysis of Interferometric Synthetic Aperture Radar [InSAR] Images," the authors broadly validated the use of InSAR data as a tool for early detection of geological hazards and failing infrastructure, including sinkhole development, potentially dangerous rock slopes, distressed bridges, rock buttresses, and other geotechnical assets. By bringing the InSAR dataset into a GIS dataframe and correlating the data to published maps of sinkhole locations and karst terranes, the authors were able to correlate average displacement velocities of InSAR data points (scatterers) with respect to their proximity to mapped sinkholes. Additionally, the authors correlated the InSAR signal characteristics with kinematic analysis of rock slopes using point-cloud data generated using digital photogrammetry and LiDAR. Lastly, the displacement time-series of the InSAR scatterers were used to screen for compromised geotechnical assets and infrastructure, and the findings were strongly confirmed by field inspection of distressed bridges and a failing rock buttress. The validation of InSAR data for these purposes thus allows generation of GIS-based geohazard and at-risk infrastructure/asset maps and provides the opportunity to augment or eventually replace a periodic inspection-based infrastructure management system with continuous performance-based system.

Notes

30. Use of Multi-Electrode Electrical Resistivity to Define the Depth of the Landslide and the use of Isolated Tie-Back Plates to Stabilize the Landslide in Steep Terrain

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ABSTRACT

An existing landslide located at the southeast corner of the Telluride Regional Airport has represented an on-going liability for the Airport, the Federal Aviation Administration, and the Colorado Department of Transportation. A catastrophic failure of this landslide occurring in a manner similar to that which occurred at the Airport in 1987 has posed an on-going threat to closing Colorado State Highway 145 which is located below the slide area. The existing landslide was characterized as a series of multiple failed block areas located downhill of the airport runway that have occurred in severely weathered Mancos Shale.

A total of 12 alternatives were evaluated to mitigate the landslide, including the preferred alternatives of either partial or total landslide removal. However, prior development left essentially no place on the airport property that would allow for the disposal of the landslide debris, and the closest off-site disposal area was approximately 40 miles from the site. As a result, in-situ stabilization of the landslide, including a primary system of isolated tie-back anchor plates with strand anchors, and a secondary system of high strength steel mesh and intermediate anchors was selected for the ultimate design to stabilize the landslide in place.

For design purposes, geotechnical characterization of the slide area was accomplished through geological mapping, conventional borehole exploration and geophysics using multi-electrode resistivity (MER). The paper discusses the benefits of using MER and isolated tie-back anchor plates, particularly after discovery of survey error required redesign of the entire stabilization system half way through the project.

Notes

31. "Large" - Scale Seismic Reflection for Infrastructure Projects - Not just for Oil and Gas Anymore

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ABSTRACT

The seismic reflection method is one of best established geophysical techniques taught in introductory geophysical courses. A common misconception of the method is that it is solely a tool for mapping deep geologic structure and stratigraphy. This perception is unfortunately associated with the level of cost and scale required for petroleum exploration.

Modern engineering-scale seismographs (12-48 recording channels) have been used to a varying degree of success in mapping shallow geology with seismic methods. Recent advancements developed for the petroleum industry in instrumentation and data acquisition are being co-opted by the shallow geophysics community with tremendous success. Wireless sensors, very large seismic sources, and professional-level data processing services are now being applied beyond the oil patch and incorporated into small engineering-scale projects.

In this paper, we will show several examples where the utilization of hundreds of recording channels was capable of providing high-resolution geophysical data for a fraction of the exploration costs required only 10 years ago. Project examples include identifying karst features, mapped and unmapped fault structures, and general geologic structure. These examples are completed, ground-truthed engineering projects. Additionally, we present one example where the seismic reflection method was only marginally successful at achieving project goals, as well as a discussion about the drawbacks and limitation of the method.

Finally, as an industry we can safely state that seismic reflection surveys are no longer "just for the big boys" and can provide added benefit to the shallow engineering community.

Notes

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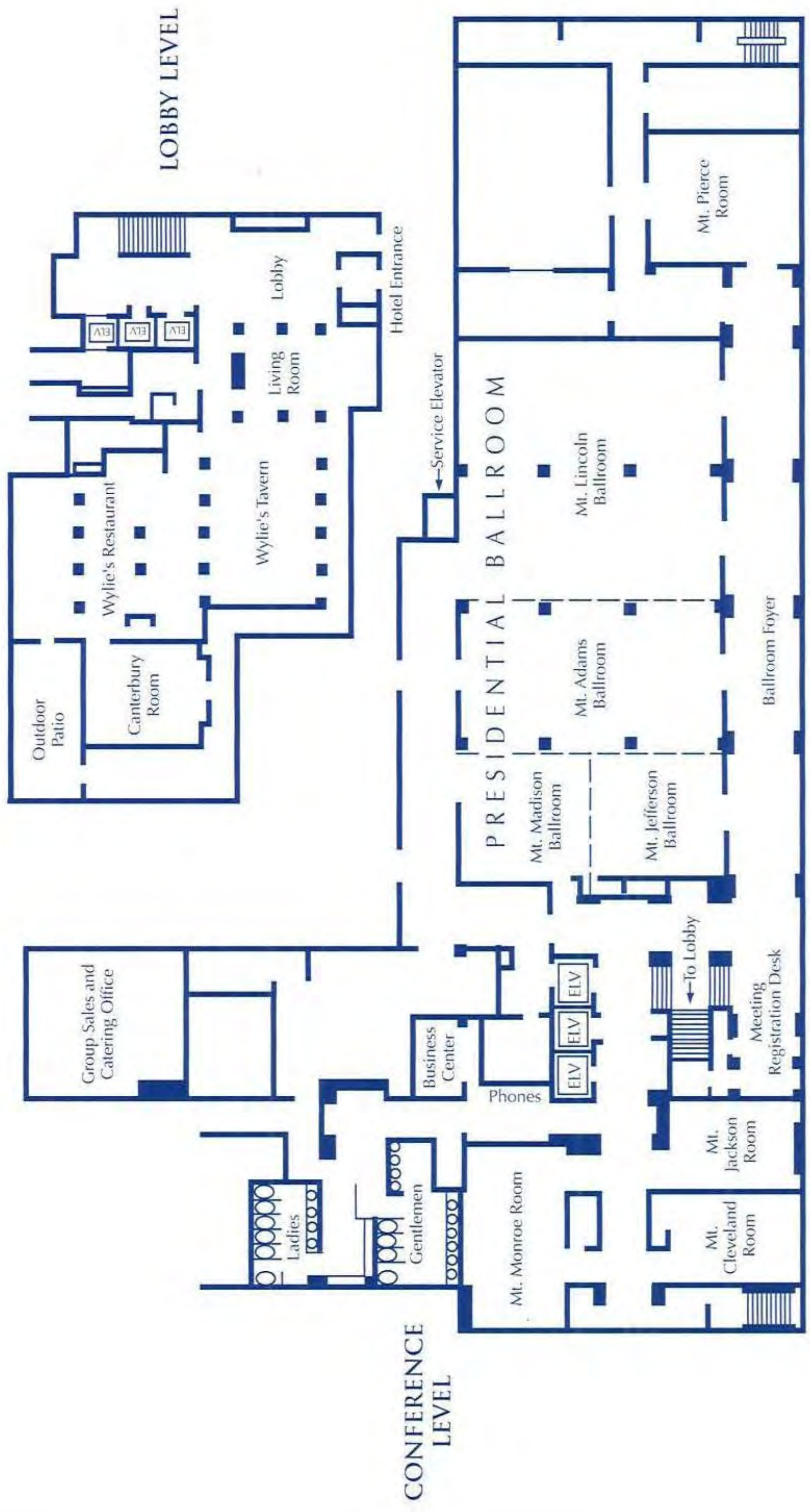
North Conway Grand Hotel Floor Plan



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Squeezing Between Rock Cuts
The Route 128/I-95 Add-A-Lane Project
Dedham/Westwood, Massachusetts

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ABSTRACT

In the 1950s the new Route 128/I-95 corridor was cut 10 to 50 feet into the granite bedrock. Over time, a lane was added along the shoulder. Now another is being constructed along the median. The result is tall rock cut slopes with narrow catchments. When this tight squeeze became apparent to the owner, GZA was asked to meet with project team and consider alternatives for rock slope stabilization and rockfall catchment.

Initial observations revealed several areas at risk for rock fall into the travelway. A graduated approach was developed with limited field measurements and chart-based catchment evaluations for the lower risk slopes; and more detailed field mapping, kinematic analysis and computer-based catchment evaluation for the higher risk slopes.

Over 1,400 discontinuities were mapped on nine rock exposures covering approximately 4,000 linear feet of cut. The field data were gathered via a GIS-based application on a tablet computer, tied to GPS locations, imported directly to spreadsheets, then directly into software to create stereographic projections, expediting kinematic and CRSP catchment evaluations.

The team focused on unstable planes and wedges, and on irregularities that could serve as launch points for fallen rock. Scaling and local stabilization were recommended to mitigate the launch points and unstable areas along the median. Given the narrow width, permanent barriers were evaluated and recommended along the outside shoulders. Scaling is scheduled to begin in summer 2013. Observations and measurements during scaling will be used by GZA to design localized stabilization measures, and update CRSP analyses of the launch points.

INTRODUCTION

The Massachusetts Department of Transportation (MassDOT) is widening a significant portion of Route 128/Interstate 95 in the greater Boston area, a project designated as the Route 128/I-95 Add-A-Lane project. Route 128 was converted to a divided highway in the 1940's, and at that time, the corridor was reconstructed with sufficient width to allow for future widening of the highway. The area of interest, Section IV of the Add-A-Lane project, passes through a circa-1940 rock cut in the towns of Dedham and Westwood, Massachusetts.

Because the right of way area was wide enough to allow construction of the planned widening, additional rock excavation was not needed and scaling and stabilization were not envisioned. During construction of this portion of the widening project, MassDOT and The Louis Berger Group, the highway designer for Section IV, discovered potential concerns regarding stability and rock fall potential for existing slopes.

GZA GeoEnvironmental, Inc. (GZA) was asked to visit the site and observe the existing rock slopes. During the initial site reconnaissance, GZA observed portions of the exposed rock slopes that appeared to be marginally stable based on visual observation. The project team agreed that a field exploration and measurement program was warranted to provide data suitable to evaluate the rock stability.

PROJECT AREA

The area investigated for Section IV of the Add-A-Lane project included nine different rock outcrops along the northbound and southbound barrels of a 1.6-mile portion of the highway. The project limits and rock outcrop locations are indicated on the aerial photograph shown in **Figure 1**.

Figure 1 – Project Location Shown on Aerial Photograph

A total of approximately 4,100 lineal feet of existing rock slope were evaluated for this project, with maximum heights varying from about 18 to 43 feet. The evaluated outcrops are summarized in **Table 1**.

Table 1 - Summary of Investigated Rock Slopes

Outcrop	Location	Approximate Length (ft)	Maximum Height (ft)
Outcrop 1	NB Left Shoulder	980	43
Outcrop 2	NB Right Shoulder	330	29
Outcrop 3	SB Right Shoulder	460	33
Outcrop 4	SB Left Shoulder	490	29
Outcrop 5	SB Right Shoulder	260	22
Outcrop 6	SB Left Shoulder	490	36
Outcrop 7	SB Right Shoulder	530	19
Outcrop 8	SB Left Shoulder	100	18
Outcrop 9	SB Right Shoulder	460	22

Note: Left shoulder consists of the shoulder adjacent to the median for NB and SB barrels.

The nine rock outcrops investigated for this project were all created by blasting methods. At the left slope (median) rock slopes, visible half casts on the bedrock surface suggest that perimeter control was typically used in locations with rock slope heights greater than about 20 feet to provide a 1 horizontal to 4 vertical (1H:4V) rock slope, except near the tops or ends of rock exposures. Occasional mid-slope benches are present at Outcrop 1 that were apparently created during the original rock excavation and possibly enhanced by subsequent rock fall. The rock slope surface appears to have been damaged by blasting operations in some locations, primarily resulting in opening of fractures and/or partial dislocation of portions of the rock mass along existing jointing. In addition, differential weathering appears to have occurred along joint features in several areas.

Photographs representing the range of conditions for the median rock slopes are presented in **Figure 2**.

2a – Outcrop 1, typical mid-slope benches at right

2b – Outcrop 1, without benches

2c – Outcrop 4

2d – Outcrop 6

2e – Outcrop 8

Figure 2 – Representative Left (Median) Shoulder Rock Slope Photographs

The right (outside) shoulder slopes are characterized by irregular slope geometries, with inclinations ranging from past vertical (i.e. overhanging slopes) to moderate angle slopes (40 to 55 degrees), with low angle to horizontal mid slope benches in several areas. The condition of the rock slope surface varies along each of the outcrops. Some areas have a consistent slope and others have several slope changes and mid slope benches. These slopes appear to have been drilled and blasted without perimeter control, resulting in geometry being largely controlled by the predominant local jointing patterns and joint spacing in the rock mass. Differential weathering was observed in several locations. Photographs representing these rock slopes are presented in **Figure 3**.

3a – Outcrop 2

3b – Outcrop 3

3c – Outcrop 5

3d – Outcrop 7

3e – Outcrop 9

Figure 3 – Representative Right (Outside) Shoulder Rock Slope Photographs

PROPOSED HIGHWAY WIDENING

The existing NB and SB barrels of Route 128/I-95 are typically within 10 feet or less laterally from the toe of existing right shoulder (outside) rock slopes, while the plan distance between left shoulder (median) outcrops and the existing roadway varies from about 20 to 40 feet. The widening is being constructed along the median side of the existing roadways to utilize the currently available space, pushing the travelway towards the higher rock cuts and reducing the catchment width.

Prior to recognition of rock slope stability and rock fall hazards as a consideration, left shoulder catchment geometries were typically designed in accordance with MassDOT standards. Existing right shoulder geometries do not provide a depressed catchment between the rock slope and the roadway and are very narrow, typically on the order of 5 feet. In the original proposal, the right shoulders were not to be modified.

SCOPE OF SERVICES

The goal of the geotechnical and geological assessment was to consider rock slope stability and rock fall hazards and provide recommendations to mitigate areas that were considered to be of risk to the traveling public. To that end, GZA proposed a phased approach to the evaluations that included limited evaluations at some locations, and a more complete evaluation at more critical locations. The combination of significant height, reduced catchment width and visible, partially dislodged blocks and wedges at Outcrop 1 justified a full evaluation at that location, which was proposed to include the following items:

a. Obtain and review rockfall and related maintenance documents from MassDOT;
b. Qualitatively review photographs and mapped bedrock geology;
c. Assess proposed/existing rock slope and catchment geometry via plan review, on-site observation, and possible additional survey, if needed;
d. Field-characterize continuity & spacing (potential block size) using field measurements and scaled photographs;
e. Preliminarily evaluate catchment using Ritchie (1963) rockfall ditch design chart;
f. Perform detailed field mapping of joints and faults (include identification of visually at-risk blocks);
g. Kinematically analyze field mapping data using DIPS/SWEDGE;
h. Assess stabilization/protection system alternatives;
i. Perform detailed catchment evaluation using Colorado Rockfall Simulation Program (CRSP); and
j. Complete final design of stabilization and enhanced catchment or rockfall mitigation measures, if needed.

Given their slope geometries and visible characteristics, it was apparent that Outcrops 2 through 9 were less prone to rock fall hazards related to kinematic instability than Outcrop 1. Therefore a reduced scope was proposed at those locations. **Table 2** summarizes the proposed scope for outcrops 1 through 9. The task items are identified by letter as previously described.

Table 2 – Preliminary Scope Summary

LOCATION		APPROX. HEIGHT (FT)	VISIBLE EXISTING ISSUES	PROPOSED EVALUATION
1. SB 253+00 RT	Median shoulder Rt 128	20 - 50	Scaling needed for loose blocks/large slabs High slope and narrow catchment (may be adequate) Toppling north end Slab sliding - wet in few spots	a,b,c,d,e,f,g,h,i,j
2. SB 253+40 LT	Outside shoulder SB on ramp	15 - 25	V. narrow catchment (on-ramp slow speed traffic)	a,b,c,d,e
3. NB 131+00 RT	Outside shoulder Rt 128	10 - 25	V. narrow & shallow catchment	a,b,c,d,e
4. NB 131+00 LT	Median shoulder Rt 128	15 - 30		a,b,c,d,e
5. NB 139+20 RT	Outside shoulder Rt 128	10 - 15	Flat, narrow catchment	a,b,c,d,e
6. NB 142+00 LT	Median shoulder Rt 128	25 - 35	Overshot at top w/ several 3-8' blocks loose	a,b,c,d,e
7. NB 142+00 RT	Outside shoulder Rt 128	10 - 15	Flat, narrow catchment	a,b,c,d,e
8. NB 146+00 LT	Median shoulder Rt 128	10 - 20		a,b,c,d,e
9. NB 146+00 RT	Outside shoulder Rt 128	15 - 25	Flat, narrow catchment	a,b,c,d,e

The MassDOT / Louis Berger Group / GZA team discussed the proposed scope of services in light of the project schedule and goals of providing stable, low maintenance rock slopes that reduce risk to the traveling public to the fullest extent possible. A significant emphasis was placed on minimizing the duration of lane closures in the heavily travelled corridor. Recognizing the tight schedule and desire to minimize disruption to traffic, it was agreed that GZA would complete a full evaluation (items a through j) for each rock slope during the initial mobilization.

REGIONAL BEDROCK GEOLOGY

According to the available bedrock geologic maps (*3*), the regional bedrock is Gabbro-Basalt (Gg) and Alkali-Feldspar Granite, Granite, Quartz Monzonite, and Granodiorite (Gf) of the Westwood Granite Formation. The Gabbro-Basalt is described as "very much altered, probably originally gabbro and (or) basalt." This unit has been complexly intruded by the alkali-feldspar granite, resulting in variable degrees of thermal alteration of the rocks.

The Alkali-Feldspar Granite, Granite, Quartz Monzonite, and Granodiorite unit (Gf) is described as "fine-grained to very coarse-grained; pink, pink and light green, and light gray rocks containing plagioclase feldspar, pink perthitic orthoclase, large glassy quartz crystals, and biotite.

The geologic map does not indicate the presence of faults in the immediate vicinity.

The observed bedrock in the project area consists primarily of medium to coarse-grained, grey to dark grey gabbro and fine- to coarse-grained, light gray to light pink-gray, granite and granodiorite. These rocks are slightly weathered and hard to very hard. The gabbro and granite are seen to intrude exposed regions of basalt in several places with high angle to vertical contacts. The basalt is a grey to dark grey, slightly weathered, hard to very hard rock.

FIELD MAPPING

Geologic field mapping was undertaken to provide data for evaluating the stability of existing rock cuts. The field mapping effort was conducted by a two-person crew of GZA engineers between July 10 and October 20, 2012, working during low-traffic windows and on weekends. GZA made a total of 1,478 direct measurements of bedrock joints and features exposed at the ground surface and accessible with a ladder and a bucket truck with a 70-foot-long extension arm.

Field mapping included assessment of rock type, dip, dip direction, spacing, persistence, roughness, aperture, filling and seepage. The characteristics of each feature were recorded digitally using a tablet computer and a Geographic Information Systems (GIS) application. The application was tailored to project specific requirements for the mapping effort. The field data were entered into the tablet computer, and the results were tabulated in a spreadsheet format that served as input for the stereographic projection and kinematic analysis software. By eliminating the usual step of transcribing field data, the accuracy was improved.

Each feature was assigned an identification number, and marked on the rock face with a paint spot. The approximate locations of the mapped features were annotated on photographs of the rock slope, an example of which is presented in **Figure 5**.

Figure 5 – Photographs Annotated with Mapped Features

The field mapping effort also included field surveying of slope and catchment sections at places where the existing conditions appeared to differ from the plans and sections. These areas included irregular, stepped, high angle geometries that are typically not well captured by traditional aerial and optical survey. These issues most frequently occurred in the right shoulder where controlled blasting techniques did not appear to have been used in the original drilling and blasting operations. A total of 15 representative cross section geometries were developed for the five outside shoulder slopes.

EVALUATION METHODOLOGY

A consistent analytical approach was applied to the evaluation of stability for each rock cut. The analysis included three general steps: develop a model for typical bedrock structure, conduct a stability assessment, and complete rock fall catchment evaluations. Details of the evaluations are discussed below.

Methodology for Evaluating Bedrock Structure

The structural data developed from field bedrock mapping was analyzed to identify the significant sets of discontinuities for use in stability evaluations. The process involved converting the numerical dip and dip direction data from each discontinuity into the unique pole representing the plane of that discontinuity. The poles were then plotted on a lower hemisphere pole plot for each outcrop using the analytical software DIPS Version 5.1 by Rocscience. The poles were grouped by photograph number to allow more direct comparison of photographic documentation with pole plots and tabulated data. The density of poles was contoured and plotted to assess the central tendencies and orientations of the most frequent discontinuities. Based on our evaluation of these plots, the discontinuities were grouped into representative joint sets for stability evaluations. In general, the primary joint sets were consistent across the exposure.

The great circles and the cut face orientations were plotted for use in graphical evaluation of rock slope stability following the methodology described by Hoek and Bray (*1*). The condition of the mapped joint sets were typically smooth to rough, stepped to planar, tight to open, and predominantly dry, without infilling.

Stability Assessment Methodology

The overall stability of the proposed rock cut slopes is governed by: (1) the orientation of the rock discontinuities (joints) with respect to each other and the rock cut face; (2) the persistence of the joints; (3) the cut slope angle; and (4) the shearing resistance along the joints. Rock slope stability analyses focused on three primary modes of potential instability: (1) two dimensional plane instability, (2) three-dimensional wedge instability, and (3) toppling instability (*1*).

Plane Instability

Plane instability can occur when rock discontinuities are oriented parallel or sub-parallel (within 20 degrees of the cut face), and dip into the excavation at angles greater than the available friction angle along the discontinuities. Based on our experience, we estimated the available friction along the discontinuities to be 30 degrees, in the absence of a significant contribution from roughness. If the great circle representing a joint set is parallel to and in front of (at a flatter slope angle than) the great circle representing the cut slope, the joint set can daylight in the cut face, and if the dip angle exceeds the available friction angle, there is a potential for two-dimensional plane instability.

Wedge Instability

Three-dimensional wedges can form at the intersection of two or more discontinuities and the cut face. A wedge may be unstable if the line of intersection of the discontinuity planes is perpendicular or sub-perpendicular to the cut face and dips toward the excavation depending on the friction angle and orientation of the discontinuities and other factors. This situation is kinematically possible if the intersection of any two great circles representing joint sets occurs in front of (at a flatter angle than) the great circle representing the cut slope, and the plunge of the intersection is steeper than the available friction angle.

Toppling Instability

Toppling instability can occur when elongated blocks form along closely spaced steep to near vertical discontinuities that dip into the cut face and are intersected by near horizontal joints.

Rockfall Catchment Evaluation Methodology

Unstable blocks have the potential to develop in existing rock cuts, which could fall out if they are not stabilized or scaled periodically. With passage of time, weathering, water, root growth and freeze-thaw cycling will tend to enhance and widen joints in the rock cut slopes, and will result in loosened bedrock blocks. Even if visibly unstable bedrock blocks are identified and scaled or stabilized during construction, rockfall from the cuts is likely over time. To mitigate this risk, GZA performed catchment evaluations for all the rock slopes in the project, even those where kinematic instability was not predicted by the evaluations.

Chart-Based Catchment Solutions

GZA reviewed the rockfall catchment potential for more uniform rock slope geometries using two chart-based methods: Ritchie rockfall ditch design chart (*5*) and the Oregon DOT Rockfall Catchment Design Guide (*4*).

Ritchie (*5*) provides a minimum catchment depth and width based on the slope height and inclination assuming the catchment has a flat base adjacent to the rock slope and a steep upslope at the outside of the catchment, near the shoulder. The Oregon DOT design charts are for 40-foot-high, 1H:4V rock cut slopes and provide recommended catchment widths for either flat or uniformly sloped catchments with a low point at the rock face. These methods were used for initial assessment of rock fall catchment. The limitations are that these are approximate solutions, and only valid to the extent that catchment geometries, slope angles, and heights are representative of the actual conditions.

The chart-based solutions were developed for uniform slopes, and are not able to characterize irregular catchment geometry or multiple slope inclinations. Slope geometries with low-angle mid-slope benches tend to serve as kickers, causing the rockfall to gain horizontal momentum and travel further laterally than those with a consistent slope angle. Where this type of irregular slope geometry, or unusually narrow catchment was observed, the limitations of the chart-based solutions were considered, and more detailed analyses were warranted.

Computer Simulated Rockfall

To better account for varied rock slope heights, irregular slope geometries, and specific catchment geometries, GZA used computer simulated rockfall analyses to evaluate if the proposed geometry provided a sufficient rockfall catchment. The potential for falling rocks to enter the travelway was evaluated using the Colorado Rockfall Simulation Program, Version 4.0 (CRSP). CRSP employs user defined slope and ditch geometries and a series of input parameters to simulate the rockfall behavior for a given slope. Rockfall parameters include the size and shape of the rocks that compose a rockfall event, surface roughness, and the coefficients of friction and restitution of the slope and catchment.

Typical ranges for the coefficients of friction and restitution were selected based on guidance provided in the CRSP manual (*2*). The coefficient of restitution has been found to have the largest impact on rockfall modeling. Considering that the hard rock at the site and seasonally frozen ground reduce effectiveness of the catchment, restitution coefficients in the upper range of values were selected for soil (0.9) and rock (0.95). The surface roughness was modeled as 4 inches, which accounts for typical irregularities in the rock face, catchment area, and/or falling rock surfaces.

The output, from the CRSP analysis, estimates the percentage of the modeled rocks that will be contained in the catchment area for a given slope geometry, rock size, and slope height. The typical criterion, for acceptable rockfall catchment design (used by Ohio DOT and others), is that at least 95 percent of the modeled rockfall is contained in the catchment and does not go past the edge of pavement; in this case, the outside edge of the paved shoulder.

Catchment and slope geometry were typically based on measured average slope inclinations and slope heights shown on the project cross sections for more uniform, median shoulder rock slopes. Cross sections measured by GZA during field mapping were the basis for modeling the outside shoulder rock slopes. Typical section locations were selected for analysis at the higher points along the outcrops and where slope and/or catchment geometry was irregular or extreme.

Multiple block sizes were modeled using CRSP for each rock slope. At the median (left) rock slopes there was not enough fallen rock nor were there well defined negative spaces left by fallen rock to allow direct estimation of potential rock fall sizes and distribution of sizes. In order to model the size and distribution of these potential blocks, GZA calculated the mean, mean minus one standard deviation, and mean plus one standard deviation based on joint spacing for the three joint sets most likely to result in falling blocks. Three block sizes were developed for each outcrop. The distribution was estimated to be roughly normal by assigning 60 percent of the blocks to the mean-size, and 20 percent each to the larger-size (mean plus one standard deviation) and smaller-size (mean minus one standard deviation) blocks.

At the outside shoulder rock cuts, the stepped surface was controlled by the primary joints and the persistent stepped nature allowed direct field-measurement of the negative spaces where blocks had fallen out. Compilation of these data along with the joint spacing data previously described showed that there were typically 2 or 3 predominant rock block sizes, and at these locations the distribution appeared to be equal between the block sizes. Where 3 sizes were present, each was assigned 33 percent of the overall distribution. Where there were 2 block sizes, each was assigned 50 percent of the distribution.

Estimating the overall catchment reliability was broken into three steps as follows:

1. Evaluate block size and block size distribution based upon direct measurement of fallen blocks or the negative spaces remaining in the outcrop, where possible. Augment this data with a statistical evaluation of the field measured joint spacing and assume a normal distribution of block sizes where there are insufficient direct observations. Estimate the weighting based on the assumed or observed distribution of block sizes.

2. Run CRSP to model 500 blocks of each block size falling from the upper one-third of the rock slope and calculate the percentage retained for the individual block sizes;

3. Take a weighted average of the CRSP results for each block size. This average accounts for the distribution of sizes and consequently estimates overall retention reliability for the catchment.

FINDINGS AND RECOMMENDATIONS

The results of our evaluations are summarized in the tables and narratives below.

Outcrop 1 – Kinematic Analysis

A total of 379 joint observations were plotted for analysis. Based on our evaluation of these plots, the discontinuities were grouped into four joint sets. The orientations of the joint sets (JS1 through JS4) are summarized in the following table, along with the range in existing cut face orientations (C1S and C1N).

Table 3 – Summary of Discontinuity Data, Outcrop 1

Joint Set / Cut face	Dip Direction (degrees)		Dip (degrees)	
	Range	Central Tendency	Range	Central Tendency
JS1	252-279 72-99*	265	68-90	80
JS2	325-357 145-177*	340	76-90	87
JS3	232-257	244	31-53	43
JS4	270-333	303	10-30	19
C1S	--	291	--	76
C1N	--	303	--	76

Note: * The second range defines the portion of the joint set that crosses the vertical plane. Refer to Figure D-2 for graphical representation.

The lower hemisphere pole plot and contour plot showing central tendency poles and great circles of identified joint sets developed for Outcrop 1 are presented in **Figure 6**.

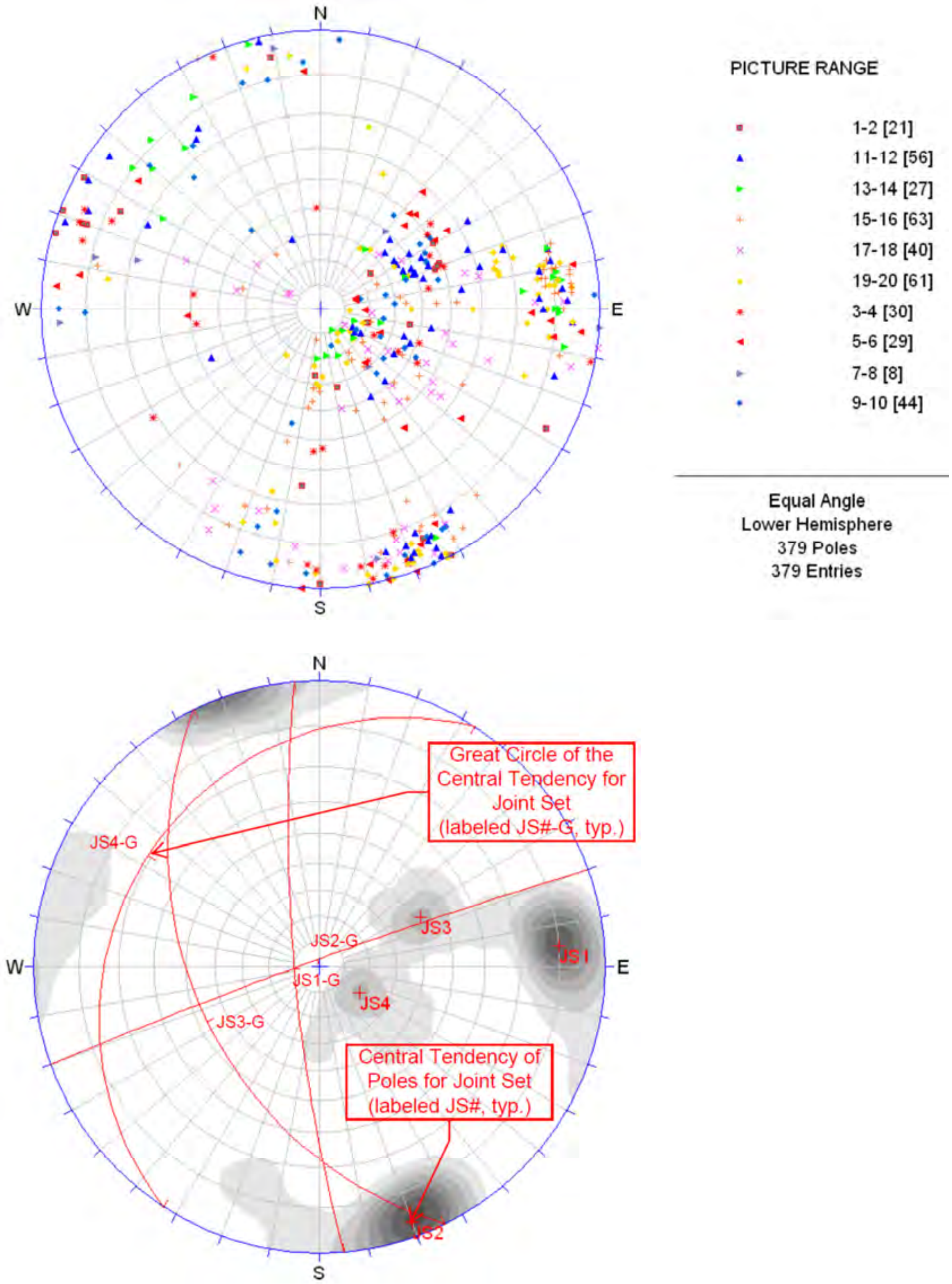


Figure 6 – Lower Hemisphere Pole Plot and Contour Plot, Outcrop 1

The planes representing the central tendencies and the cut face orientations were plotted for use in graphical evaluation of rock slope stability. The plot used for kinematic stability evaluation is presented in **Figure 7**, and the results are summarized in **Table 4**.

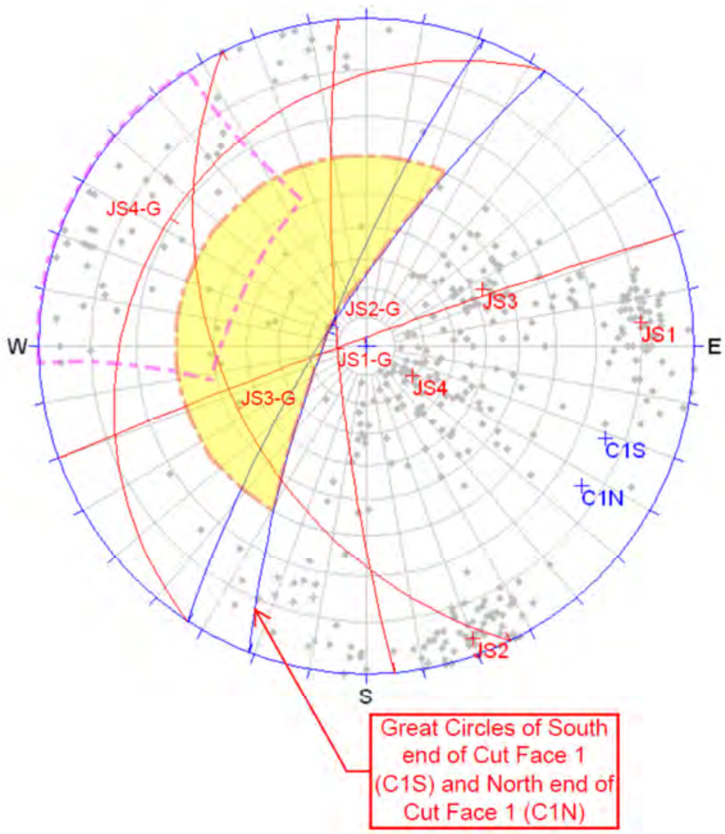


Figure 7 –Kinematic Analysis Plot, Outcrop 1

Table 4 – Summary of Kinematic Analyses, Outcrop 1				
Instability Type	Joint Set/s	Dip Angle / Plunge of Intersection (deg)	Rotation Relative to Cut Slope (deg)	Possible
Planar	JS3	43	44 +	no
	JS4	19	0	no
Wedge	JS3/JS4	19	9	no
	JS2/JS3*	42	39	yes
	JS4/JS1	13	50	no
Toppling	JS1 (outliers)	65 - 90	many < 20	no

Evaluation of the three major instability types showed that JS4 was rotated too far from the exposed face to be considered susceptible to planar failure; although parallel, JS4 dips at too flat an angle to be considered susceptible to planar failure; the plunge of the JS3/JS4 and JS4/JS1 wedges are too flat to be considered susceptible to wedge failure; the combination of JS2/JS3 is considered susceptible to wedge failure; and the outliers of JS1 do represent a kinematically possible scenario for toppling failure. It is noted that the joints intersecting JS1 are spaced similarly to JS1, and would most likely form relatively equal-sided blocks, that would be more prone to weathering and seasonal fall out than to toppling.

In summary, our evaluations indicate that the identified joint sets are typically not conducive to large-scale instability by two-dimensional plane or toppling modes in the existing rock cut face. The wedge formed by JS2/JS3 forms a potential failure mode that warrants further evaluation. Review of the mapping data and photographs identified several locations where the JS2/JS3 combination is present and at risk. These areas have been flagged for either scaling or stabilization. In addition, the presence of parallel planes and wedges at slightly flatter angles increases the likelihood of rock fall over time due to ice wedging, seepage forces, root growth, and other weathering, which has created several portions of the rock slope that appear marginally stable. Therefore, catchment analyses were warranted at Outcrop 1.

Outcrop 1 – Catchment Analysis

The results of the Outcrop 1 catchment evaluation using CRSP are summarized in **Table 5**.

Table 5 – CRSP Catchment Evaluation, Outcrop 1

Slope Condition	Representative Rock Diameter (feet)	Percent Retained	Statistical Basis	Estimated Percentage of Overall Rockfall
Uniform 1H:4V Rock Slope; with 8H:1V Catchment Slope	1.25	98	σ - 1	20
	3	96	σ	60
	5	90	σ + 1	20
	Weighted Average	96		100
1H:4V Rock Slope with 2-foot wide mid-slope bench; with 8H:1V Catchment Slope	1.25	97	σ - 1	20
	3	75	σ	60
	5	43	σ + 1	20
	Weighted Average	73		100

At locations with a continuous 1H:4V slope and a 8H:1V catchment slope with low point at the toe of the rock slope, the design criteria are met. At least 95 percent of the modeled rock fall stays outside of the travelway. However introduction of a 2-foot-wide mid-slope bench caused a 23 percent reduction in the percentage contained, rendering the catchment inadequate to achieve the desired retention of 95 percent.

The analyses showed that even for the uniform slope, the retention of the largest diameter (5-foot) blocks was less than 95 percent. However since blocks of this size represent about 20 percent of the potential rockfall in the evaluation, the catchment reliability is maintained.

Outcrop 1 – Recommendations

Considering the presence of significant loosened rock over the surface of Outcrop 1 and the potential that these could be masking larger wedge or planar instabilities, aggressive hand-scaling of the entire slope was recommended. After scaling, observation of the remaining rock slope face is planned to allow assessment of any remaining areas of potential instability that may require stabilization. It was also recommended that the catchment be reevaluated at the completion of scaling, as the slope geometry may be more or less advantageous at the completion of that work. Additional details of the proposed future work are discussed later herein.

Outcrops 4, 6 and 8 (Median)

Kinematic analyses were conducted for the remaining median rock slopes, Outcrops 4, 6 and 8, using the same methods as described above. The results suggested that planar instability was not kinematically possible, and the only wedge instabilities identified were very thin slivers, with one side nearly parallel to the rock slope. In addition, the joint sets that would form wedges were typically located in different portions of the rock cuts; therefore, kinematically possible wedges were not observed.

The catchment for the median outcrops was analyzed with similar block sizes as Outcrop 1 based on the statistical approach to block size estimation discussed previously, but none of these cuts had significant mid-slope benches, and they were all shorter than Outcrop 1 with a greater catchment width to slope height ratio. The calculated retention was greater than 98 percent for all slope heights and block sizes at these slope locations.

Limited scaling was recommended at these outcrops to remove specific blocks that appeared unstable, primarily in the upper portions of the slopes that are judged to have been disturbed due to limited overburden during original blasting. Otherwise, the existing slopes were considered stable.

Outcrops 2, 3, 5, 7 and 9 (Outside Shoulder) – Kinematic Analysis

Between 44 and 145 joint observations were plotted for each outside shoulder exposure. Based on our evaluation of these plots, the discontinuities were grouped into 3 to 4 joint sets for each outcrop. At Outcrops 2 and 7 additional joint sets were identified at shoulder locations that were not found in the median. These were added to the kinematic analyses. In general, joint orientations were similar to those encountered for the other outcrops on the project.

The possible kinematic instability modes, encountered on the shoulder outcrop slopes, are listed in the following table. Toppling is excluded from the table, because the bedrock structure is likely to release blocks that are within the sizes used for the catchment evaluation.

Table 6 – Summary of Kinematic Analyses, Outside Shoulder Outcrops

Rock Slope	Plane Instability	Wedge Instability	Comments
Outcrop 2	no	no	
Outcrop 3	no	yes	Two possible wedge instabilities with plunge line dip of 33-64 degrees. Joint sets forming potential wedges generally in different areas of rock cut; at-risk wedges were not identified.
Outcrop 5	no	no	
Outcrop 7	yes	yes	Plane instability with joint set dip of 39 degrees. Average joint set spacing of 3 feet anticipated to limit released block size. Two possible wedge instabilities with plunge line dip of 30-65 degrees. Joint sets forming potential wedges generally in different areas of rock cut; at-risk wedges were not identified.
Outcrop 9	no	yes	Possible wedge instabilities with plunge line dip of 67 degrees. At-risk intersecting joint sets were not identified.

In summary, our evaluations indicate that the identified joint sets are typically not conducive to large-scale instability by two-dimensional plane, three-dimensional wedge, or toppling modes in the existing rock cut face. The presence of parallel planes and wedges at slightly flatter angles increases the likelihood of rock fall over time due to ice wedging, seepage forces, root growth and other weathering, and there is minimal catchment width and no catchment depth available at the outside shoulder rock cuts. Therefore catchment analyses were warranted at Outcrops 2, 3, 5, 7 and 9.

Outcrops 2, 3, 5, 7 and 9 – Catchment Analysis

The irregular geometry of the existing rock cuts combined with minimal or no depth in the catchment area suggested that chart-based catchment solutions would have minimal value. The joint set spacing was evaluated statistically as described for previous cuts. However, unlike the median rock slopes, the stepped, joint-controlled nature of the exposed faces expedited direct measurement of potential rock fall block size, and our observations suggested that the potential blocks were more evenly distributed across the sizes considered. Therefore, representative block sizes ranging from 2 to 5 feet were developed based on the direct measurements and equal probabilities were assigned to each block size.

Due to the irregular geometry, GZA elected to field survey cross sections that were considered representative the existing slopes. At each outcrop, 2 to 4 locations were selected to represent of the range of exposed face conditions. CRSP was used to evaluate the catchment for each block size and geometry. Initial assessments indicated that catchment effectiveness was inadequate at most outside shoulder rock exposures. This was driven primarily by the narrow width and limited depth of the catchment. The presence of traveling vehicles, in close proximity to the rock slopes, made excavation to widen the catchment an unfavorable option, since blasting would be required. Therefore, a rockfall barrier was evaluated at the edge of the shoulder to effectively increase the depth of the catchment.

For each shoulder rock cut, two of the cross sections modeled in CRSP along with the calculated retention of the catchment are presented below. Each section was modeled with and without a barrier. The analyses showed that only two of the exposure sections had suitable catchment without a barrier. These locations had nearly continuous, near-vertical faces extended from the top to the bottom of the rock slope, as shown in Sections 2-2 and 9-1 below (with section designation terminology corresponding to Outcrop 2 and 9 sections, respectively).

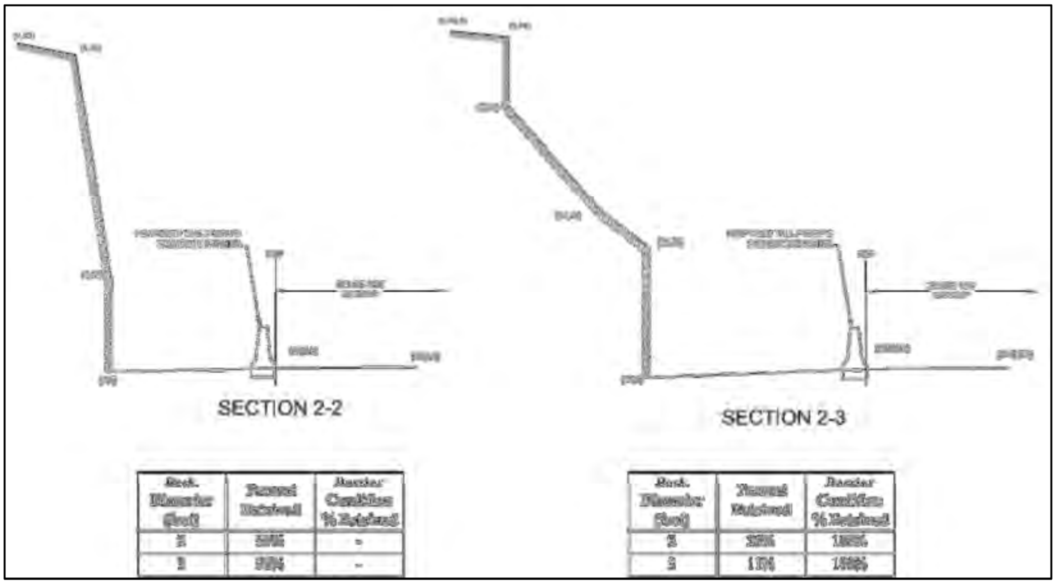


Figure 8 – Outcrop 2 CRSP Sections

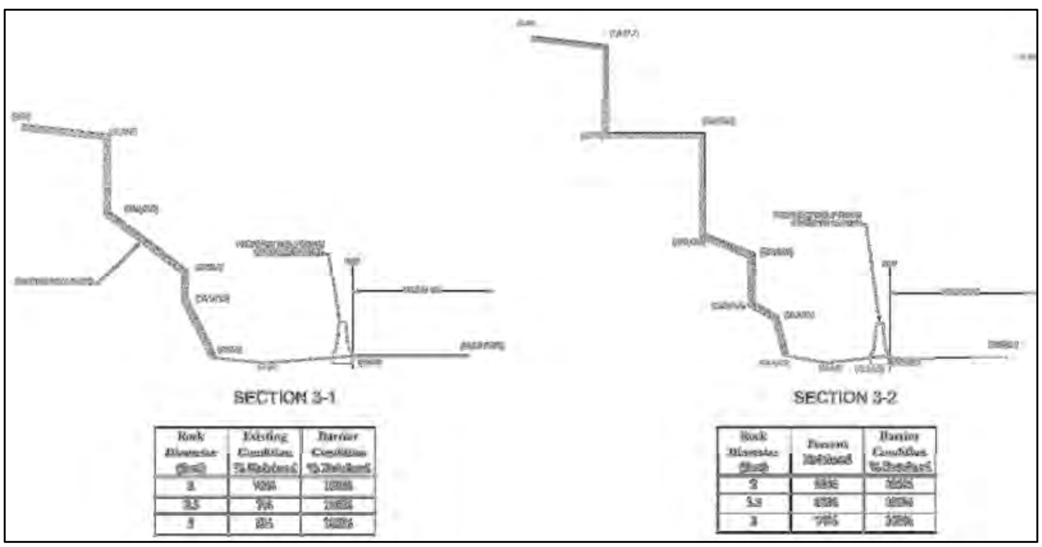
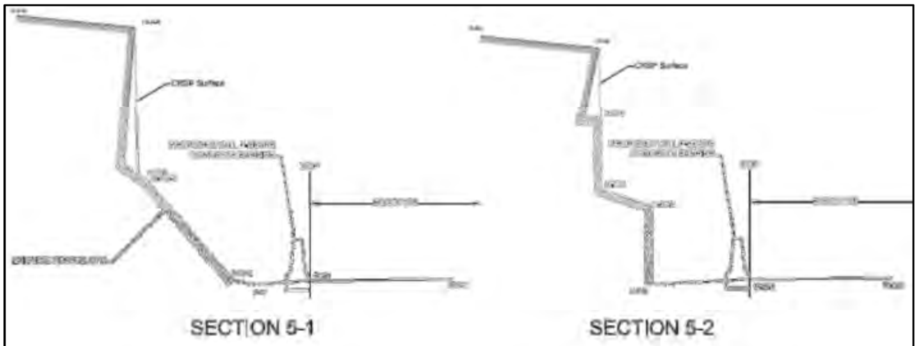


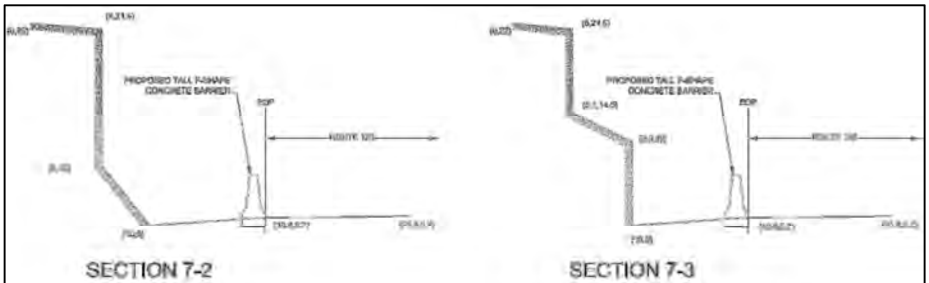
Figure 9 – Outcrop 3 CRSP Sections



Rock Diameter (Feet)	Existing Condition % Retained	Barrier Condition % Retained
1	42%	100%
2	6%	100%
3	0%	100%

Rock Diameter (Feet)	Percent Retained	Barrier Condition % Retained
1	100%	100%
2	58%	100%
3	32%	100%

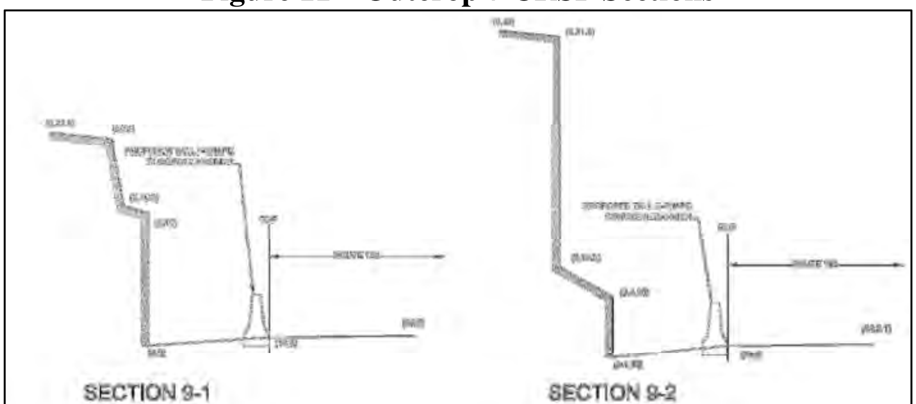
Figure 10 – Outcrop 5 CRSP Sections



Rock Diameter (Feet)	Percent Retained	Barrier Condition % Retained
2	70%	100%
3	23%	100%
5	0%	100%

Rock Diameter (Feet)	Percent Retained	Barrier Condition % Retained
2	95%	100%
3	78%	100%
5	52%	100%

Figure 11 – Outcrop 7 CRSP Sections



Rock Diameter (Feet)	Existing Condition % Retained	Barrier Condition % Retained
2	95%	-
3	100%	-
5	100%	-

Rock Diameter (Feet)	Percent Retained	Barrier Condition % Retained
2	85%	100%
3	64%	100%
5	11%	100%

Figure 12 – Outcrop 9 CRSP Sections

The results show that each rock cut included one or more cross sections with unsuitable catchment.

Several alternatives were initially considered to improve the catchment, netting, rock fall fences, and permanent concrete barriers. Netting and rock fall fences were both feasible, but considering that the primary rock fall source would be environmental, the cost of these systems was not justified. The selected alternative was a 42-inch-high, permanent concrete barrier (tall F-shape barriers per MassDOT standard specifications), which would provide effective depth to the catchment. As shown on the sections above, the addition of the barrier resulted in 100 percent retention of the fallen rock block sizes. This barrier was used as the standard barrier throughout other portions of the project, and as such did not require development of location-specific design details. Although this option will reduce accessibility to clear the catchment of fallen rock, rock fall of significant volume is not anticipated at any of these locations, so clearing the catchment zone would be infrequent.

Additional Recommendations

Each rock slope contained specific areas representing potential instabilities due to environmental forces combined with instability modes that are nearly kinematically feasible and/or rock slope irregularities that were likely associated with blasting activities. Varying degrees of scaling and/or stabilization were recommended for each slope, ranging from aggressive, full-slope scaling for Outcrop 1 to removal of only a few isolated rock fragments on the other median rock slopes. Annotated photographs were included in the engineering reports to identify areas that were recommended for scaling.

For all of the existing slopes, it was recommended to remove bushes and trees within 10 feet of the top of the slope to limit the potential for rock fragments to be loosened by root growth.

FUTURE WORK

The next phase of the project is planned for early summer 2013 and will consist of scaling and stabilization of Outcrop 1. An experienced scaling and stabilization contractor met with GZA, MassDOT and the Louis Berger group in winter 2012/2013 to provide their observations relative to the existing slope and scaling. The specialty contractor proposed scaling of the entire rock slope, primarily with hand-operated tooling (no blasting), combined with visual assessment of local instabilities that are more appropriate to be stabilized in place.

GZA plans to monitor the scaling work and assess the resulting rock face for potentially unstable features. Where appropriate, GZA and the specialty contractor will collaboratively assess local instabilities, and develop appropriate stabilization solutions (i.e., dowels, shotcrete, drains, mesh). GZA will analyze and provide design recommendations and details for the specific stabilizations, and the specialty contractor will evaluate and propose materials and methods. A submittal, detailing each proposed stabilization, will be developed for review and acceptance by GZA and MassDOT prior to implementing the work.

GZA's geologic observation and field mapping will be full time during the Contractor's scaling and stabilization work. GZA personnel will field characterize the continuity and spacing (potential block size) of features exposed after scaling using field measurements and visually identify at-risk blocks, wedges or slabs; and make direct measurements of bedrock joints and features by foot, ladder or rappelling over the slope using ropes and a harness. During and following scaling activities, the portions of the rock mass identified as potentially at-risk for instability will be further assessed.

The need for stabilization will be evaluated and developed as follows:

- Reduction and graphical presentation of rock discontinuity data in area to be stabilized;
- Kinematic analysis of data in potentially at-risk area using DIPS/SWEDGE;
- Development of preferred stabilization system alternative; and
- Kinematic analysis of area using SWEDGE including preferred stabilization system showing suitable factor of safety.

In the event that the geometry of the rock slope is significantly altered by scaling, the slope geometry will be resurveyed and a supplemental CRSP catchment evaluation will be completed.

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Fort Ann Rockfall and Emergency Repair Contract

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ABSTRACT

On the morning of October 15, 2012, a massive rockfall buried both lanes of US Route 4 in Fort Ann, New York, near the Vermont border. After it was reported that a car might be buried under the rock pile, a State Police helicopter equipped with thermal imaging equipment was brought in, and it was determined that there was no vehicle present. Cleanup operations commenced and over the next 48 hours, the New York State Department of Transportation's (NYSDOT) emergency contractor removed over 1,700 cubic yards of fallen material. Engineering Geologists from the NYSDOT evaluated the remaining rockslope and determined that the slope was stable enough to temporarily reopen the road. However, they also recommended that the slope be remediated quickly and not be allowed to go through another winter of destabilizing freeze-thaw cycles and high groundwater conditions. Due to the tight time frame, it was decided to remediate the slope under the existing emergency contract.

For the first time at NYSDOT, airborne and terrestrial LiDAR (Light Detection And Ranging) were combined with traditional ground survey and photogrammetric mapping to create a DTM (Digital Terrain Model) of the rockslope and the surrounding area. This DTM proved invaluable in the design of the remediation of the slope and for estimating rock removal quantities for the immediate cleanup and new rockslope construction. This paper will discuss the rockfall and the design and construction of the new slope under the emergency contract.

BACKGROUND

US Route 4 travels along the east side of the Hudson River from East Greenbush (east of Albany), north through Troy, for approximately 80 miles to Whitehall, NY where it turns east into Vermont. Fort Ann, NY is approximately 8 miles south of Whitehall. This is on the east edge of Adirondack Park. The bedrock at this location is the Hague Gneiss,a Middle Proterozoic sillimanite-biotite-garnet-potassium feldspar-plagioclase-quartz gneiss (Fisher 1984) that was extensively faulted during the Ordivician Taconic orogeny. The rockslopes along the southbound (west) side of this stretch of road have a history of rockfalls. Some factors that contribute to this are: the rockslopes are long and high, they contain parallel remnant bedding failure planes that dip towards the road, and they are highly jointed and brittle.

Figure 1 - Location map

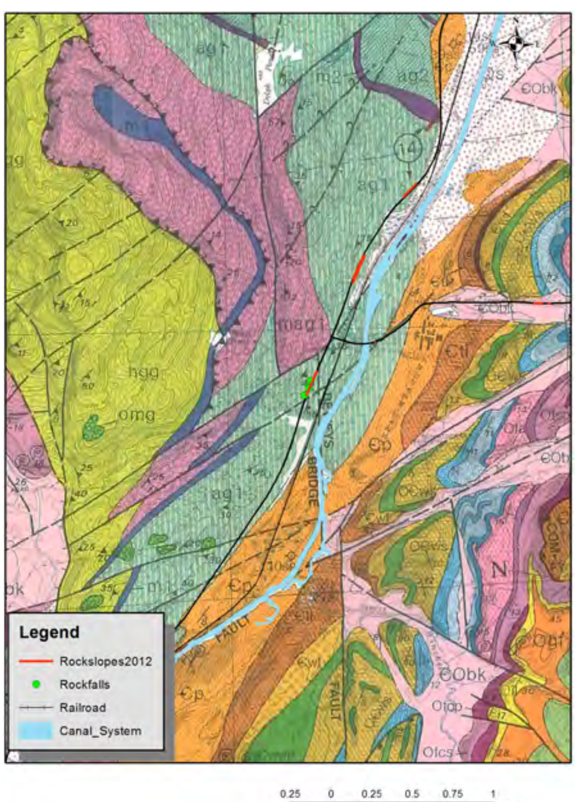

Figure 2 - Bedrock Geology Map (modified from Fisher 1984)

The rockcut at the rockfall location was completed in 1979 at a designed 3 vertical on 1 horizontal slope using pre-split blasting methods. This created a maximum 55 foot high rockslope with an average 12 foot wide by 4 foot deep ditch with a guiderail for a total length of almost 900 linear feet (LF). An unknown quantity of ¾ inch diameter, mechanically anchored, rock bolts were installed following the blasting to reinforce the rockslope.

ROCKSLOPE HISTORY

In response to a rockfall fatality along the New York State Thruway in 1988, NYSDOT began a rockslope inventory and rating system. This rockslope was first rated in 1988 as Site Number 2021 and had been re-evaluated in 1997 using the revised rating procedure described in NSYDOT's Geotechnical Engineering Manual 15. The rockslope received a Relative Risk score of 42 which was the second highest rating for Washington County and was 79th out of 598 slopes in DOT Region 1.

There were two previously reported minor rockfalls during this period, the last one occurring in July, 2010. This consisted of a total of 18 cubic yards of rock contained by the ditch at three locations along the slope. At one of these rockfall sites, the rock failed around a rock bolt at the toe of the slope. Following this rockfall, the slope was inspected by NYSDOT geologists. It was determined that additional small rockfalls would occur, but that these falls would also be contained by the ditch. The recommendation was that the ditch be cleaned of rockfall to contain any future falls. Following the rockfall, geologists marked the slope with spray paint on both sides of the failure plane as a crude method to determine if the entire slope face was moving. There was no obvious evidence of a progressive failure occurring at the slope.

Figure 3 – Rockslope Site No. 2021 – photo of slope before major rockfall

MAJOR ROCKFALL

On October 15, 2012, at 9:19 AM, an incident report was received at the DOT main offices in Albany, NY by the Regional Traffic Maintenance Coordinator. It stated that a major rockfall had occurred on US Route 4 near Comstock, in the Town of Fort Ann, at Reference

Marker 4 1803 1263. The road was closed to traffic, the rockslope appeared unstable, and it was possible that an SUV was buried under the debris. Emergency Responders and DOT personnel were at the site.

Two geologists from the main office Geotechnical Engineering Bureau (GEB) were immediately sent to the site to assist in the emergency situation that was unfolding. Meanwhile, using the NYSDOT Rockslope GIS layer, the slope was located and identified, and the appropriate work history files for the rockslope were gathered. A comprehensive search was begun to uncover any other information and pictures that were available. The road history was researched, the as built plans were located electronically on our SHARP (State Highway As-built Record Plans) database, and the PDF file of the Contract plans were reviewed.

When the geologists arrived at the rockfall site, the area was closed to non-emergency personnel, and a detour was already in place. Several television and newspaper mobile units were just off site. Before any rock was removed, the State Police had a helicopter with an infrared camera scan the entire area looking for a heat source indicating a buried vehicle or people. Fortunately, no one was found under the rockpile. The Emergency Contractor, Reale Construction Inc., of Ticonderoga, NY, was then directed to immediately begin removing the rock in the road under the direction of the emergency contract Engineer in Charge (EIC) and the Engineering Geologists.

Figure 4 - Rockfall photo from State Police Helicopter 10/15/12, 10 AM

Figure 5 - Rockfall ground view

Figure 6 - Rockfall from slope across road

EMERGENCY CONTRACT

The Emergency Contractor had mobilized earthwork equipment, including a backhoe to load two rock trucks and a hoe ram to break the large rocks. Conveniently, there was a quarry directly across from the rockslope that was willing to receive the rock as it was removed from the road. Geologists remained on the site to direct the rock removal in the roadway and to direct the scaling of loose rock remaining on the slope. The DOT press spokesperson addressed the media while the Regional Director directed the Contractor to work around the clock to reopen the road as soon as safely possible. A detour was set up to reroute the traffic around the area. Unfortunately, this was a 22 mile detour on a major truck route to and from Vermont, making it a high priority to reopen the roadway.

Many of the boulders in the roadway and on the rockslope were as large as cars. Entire trees were uprooted and dangling toward the road. With persistent hoe-ramming, all these boulders were reduced to a manageable size without the need for blasting. Due to the flat 30 degree failure surface and the pile of debris, a backhoe was able to climb up on the slope to remove loose rock from the rockslope. Work slowed down overnight, but steady progress was made.

Within two days the slope was cleared of all loose rock and debris and the ditch had been reestablished to contain small material that could ravel. A total of approximately 1,755 cubic yards of rock and debris was removed from the rockslope and deposited onto the floor of the nearby quarry. This quantity of rock removed was determined using the airborne LiDAR survey and applying a bulking factor to the rockpile. The roadway was severely damaged from the rockfall, and the construction equipment and the road surface needed to be patched with asphalt.

Before the roadway was opened to the public the geologists had to assess the stability of the slope. Since one large failure had occurred on the dominant failure plane, and this failure plane continued for another 50 feet toward the south before dipping below the road, a monitoring plan was put in place. The GEB's Instrumentation Unit engineer secured six reflectors to the slope face to detect any movement of the large block directly adjacent to the failure. The reflectors were installed with epoxy and were surveyed in with a theodolite. Displacement gauges were also installed directly on the failure plane and adjacent block. Personnel from the GEB inspected these detectors three times a week.

Fortunately there were no detectable movements of the slope following the emergency cleanup, and both lanes of the road were reopened on October 18th, only three days after the rockfall. Even though the slope was being monitored and showed no movement, geologists recommended that remediation of the slope south of the failure begin before the winter and more freeze-thaw cycles.

Figure 7 - Road reopened following rockfall cleanup

SLOPE DESIGN

It was evident that an accurate survey of the slope needed to be done in order to determine an appropriate remediation of the slope. Immediately following the rockfall, NSYDOT geologists consulted the NYSDOT Photogrammetry Section to find out what would be the best way to quickly get an accurate survey of the slope. The decision was made to conduct an airborne and terrestrial LiDAR survey with control points from a traditional ground survey. Fortunately, the Department maintains an emergency standby agreement with an airborne LiDAR vendor, and the Department leases two terrestrial LiDAR scanners. A meeting was held two days after the rockfall to set up all these surveys. Airborne LiDAR was needed to get an accurate survey of the backslope since the area was heavily wooded, and the slope dropped in elevation behind the rockface to an old roadbed that could be used for equipment access. The Photogrammetry Section would combine and process the results of all the surveys into a useable digital terrain model (DTM) for designing slope stabilization treatments.

On October 24th, just six days after the rockfall, Photogrammetry delivered the first combined point cloud, cleaned up and referenced to real world coordinates. In addition to the point cloud, Photogrammetry produced a DTM for use in Bentley Microstation V8i®, and a Leica TruView® image for use in 3D viewing and rudimentary measuring of the slope. The LiDAR point cloud was analyzed in Split FX®, a point cloud processing software designed specifically for doing geotechnical analysis of terrestrial LiDAR point clouds.

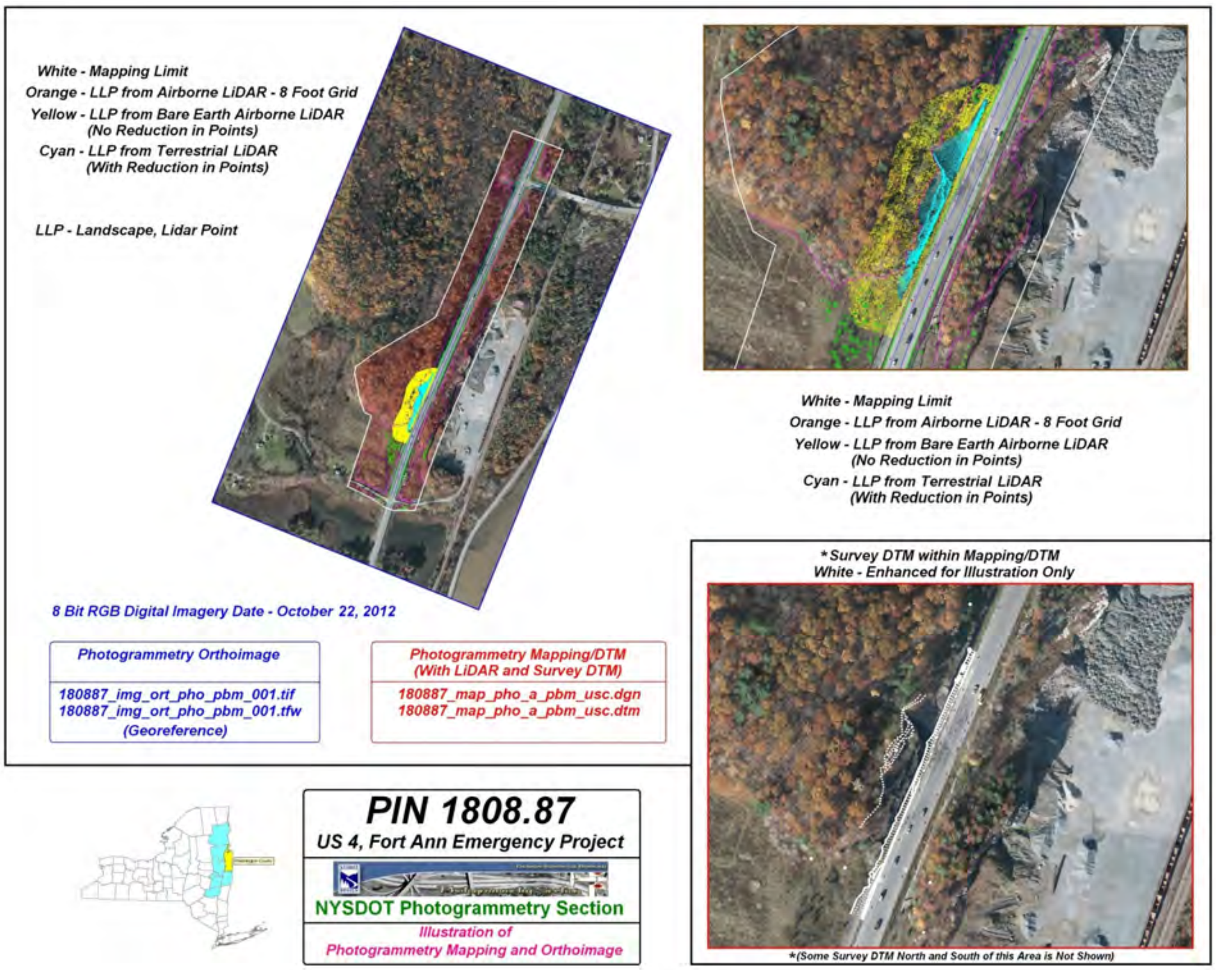

Figure 8 - Cover sheet of Photogrammetry deliverables

Analysis of the point cloud using the Split FX® software revealed that the surface of the failure plane was at an average dip of 30º and dip direction of 126º. Other fracture surfaces were found throughout the remainder of the rockslope that were oriented nearly parallel with this failure plane. Using Split FX ®software to automate the delineation of fracture surfaces and joint sets, these surfaces were plotted on a stereonet. Measurements were taken of block sizes and slope heights at various locations along the slope. Using the TruView® software, these measurements were drawn directly on the 3D photograph to better visualize the geologic conditions.

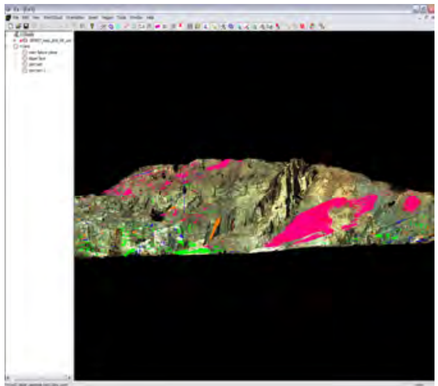

Figure 9 - Split FX® Point Cloud Screenshot

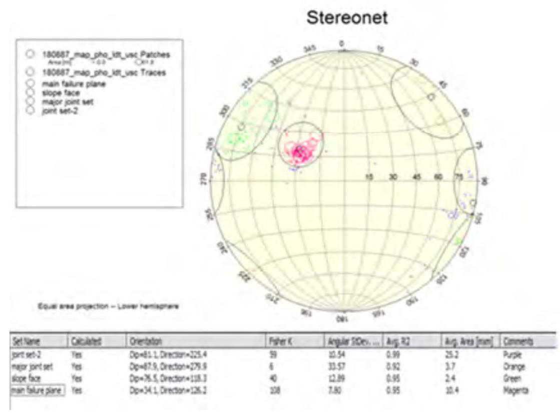

Figure 10 - Split FX® Stereonet Screenshot

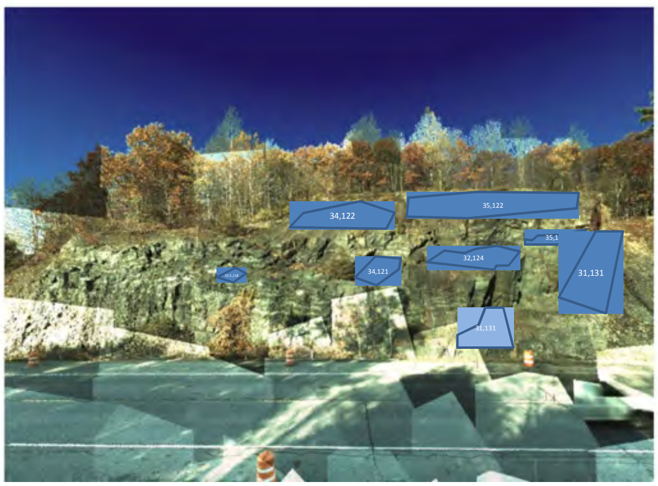

Figure 11 - Lieca TruView® Screenshot

ROCK BOLTING

A pattern of rock bolts was considered as a possible solution for remediation. Based upon slope measurements and using the ROCKPACK III® plane failure analysis program, it was determined that a bolting pattern of three rows of 15-20 foot long rockbolts would be needed for a total of 1,440 LF of bolts. Using an estimate of $150/LF of bolts, the cost for this work alone was $216,000. Bolting the rockslope would necessitate closing a lane of traffic for a large crane needed to install the bolts. The estimated duration of this work was over a month; however, based upon experience with prior projects, the time needed to perform the drilling and bolting could take much longer and extend into the winter season, affecting not only the normal traffic loads, but also the skiers using the highway to travel to Vermont.

SLOPE RECUT

A full recut of the rockslope was the preferred remediation treatment if the cost and duration of construction would be similar to rock bolting. The failure surface of approximately 30 degrees was projected through the slope DTM using InRoads® in Bentley Microstation V8i®. The toe of slope was held to be at the existing ditch and the new slope surface was projected to the top of the slope, paralleling the failure planes. This surface was considered the pay line for the new slope. Fortunately, this plane daylighted in front of an old logging road behind the slope. A blasting subcontractor could set up drill rigs along this abandoned road and drill on a 30 degree slope angle to match the failure surface angle. Based upon the model and projected 30 degree surface, it was estimated that 10,815 cubic yards of rock would need to be blasted. The total cost for the blasting and excavation work at the agreed upon Emergency Contract bid price of $20 per cubic yard was estimated to be $216,300.

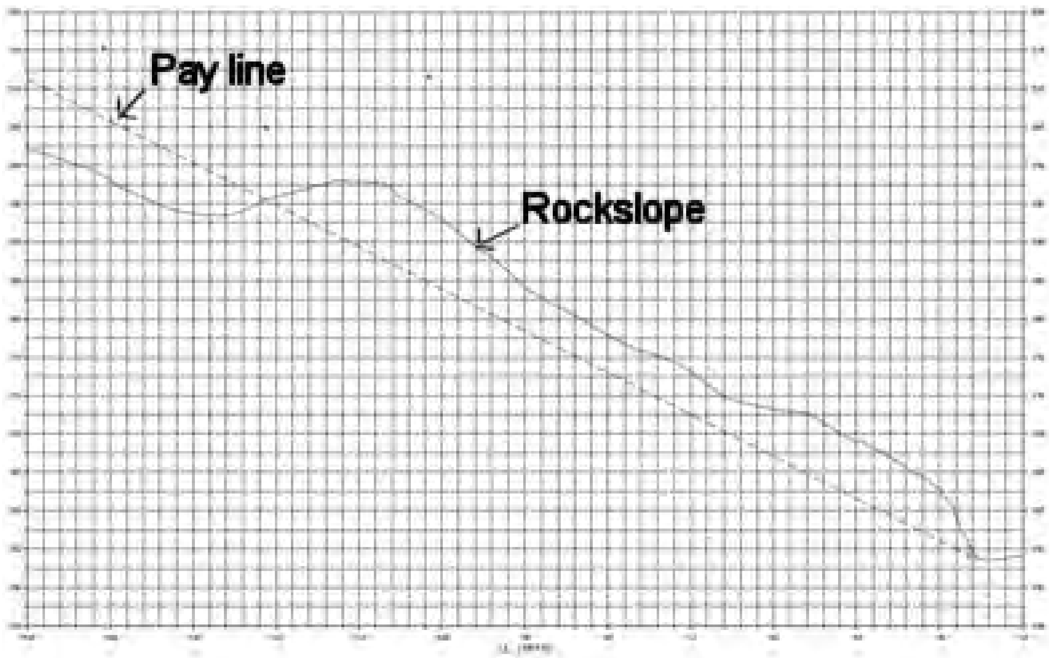

Figure 12 - Cross section of recut slope design

Hybrid designs of bolting and recutting part of the rockcut were also considered in design. Due to the relatively inexpensive bid price for the rock excavation work and time estimate for performing the work, blasting was the treatment that was selected by the Region.

SLOPE CONSTRUCTION

The Emergency Contractor had selected two blasting subcontractors for bids, and geologists met with them in the field to propose blasting the slope. Both subcontractors submitted bids and designs for the slope, proposing to drill vertical holes and blast the rockslope in two lifts instead of drilling holes on a 30 degree angle and shooting the slope in a single lift. The blasting contractors weren't confident that they could accurately drill and maintain the 30 degree slope angle for the full length of the drill holes. Initially, one of the subcontractors proposed subdrilling and loading through the pay line to achieve proper breakage. NYSDOT geologists made the decision that no subdrilling would be allowed. Any rock remaining above the pay line following the blast would be mechanically removed to the pay line. Because drillers would need to set up on the rockslope above the failure plane, a monitoring plan was put in place by the Contractor to ensure the safety of the workers. The Contractor would drill vertical holes at the top of the slope through the failure plane and monitor for movement along the failure plane with downhole inclinometers.

The decision was made to fully close the road for the duration of the blasting and rock removal. An official detour was established to begin on the first day of blasting and continue for two weeks. Based upon the bids and blast plan submittals Maine Drilling and Blasting of Gardiner, Maine was selected as the blasting subcontractor. A pre-blasting meeting was held on November 20th with the designated Project Blaster, Project EIC, Geologists, Emergency Contractor, Emergency Services, and all other interested parties. A representative from the Canadian Pacific and Delaware &Hudson Railroad was one of these parties as the railroad has a live passenger and freight line behind the quarry and within 650 feet of the slope. The Railroad agreed to provide a flagman during the blasts.

Soon after the pre-blasting meeting the contractor was given the DTM model of the rockslope and cross sections to use. After reviewing the sections and rethinking the blasting plan with the DOT Geologists, the blasting subcontractor decided to drill on a 30 degree angle and stay five to six feet above the pay line with the back row of holes. An additional two rows of drill holes were drilled in front and above the back row of holes. The old road behind the slope proved to be a good access road and drilling platform. Clearing and grubbing and construction of the access road began on November 26th. Some fill was required to ramp up and reach some of the drilling locations, and the drillers and rigs needed to be tied off.

Figure 13 - Drilling at 30º angles

Drilling for the first 120 LF of pre-split slope began on November 26th and the first blast was set for November 30th. Prior to blasting the Railroad representative was notified of the blasting schedule and twice asked to remove two maintenance cars from the area, but the Railroad's response was that they could not be moved in time for the scheduled shot. The contractor attempted to protect the railroad cars by parking construction equipment in front of the railroad cars. Sticks of dynamite (2 inch x16 inch) were loaded as primers, and bulk emulsions were pumped to within six feet of the top of each hole. The blast was delayed in a "V" pattern, with the opening to the side of the cut to reduce the chance of flyrock leaving the Right-of-Way (ROW). Each hole was detonated on a separate delay.

Results of the first blast were mixed, there was good breakage and drill alignment and a smooth final rock surface along the failure plane; however, some of the flyrock reached the rails and there was minor damage to the rail cars. This required a new blast plan submittal to the DOT to address flyrock control. It was agreed that blasting mats would be used and the top row of drill holes would be eliminated. The second large shot was detonated on December 6th without incident.

A total of four additional smaller blasts were successfully detonated between December 6th and December 11th to remove high rock and fragment large boulders. No additional flyrock left the ROW. Final scaling of the slope to remove the rock to the pay line was accomplished using a backhoe and some hand scaling and washing of the slope with a high pressure water hose. The ditch was excavated to its original design depth of four feet and the overburden at the top of the slope was graded and seeded.

FINISHED SLOPE

The finished slope was accepted for payment by the NYSDOT Regional Construction Engineer after inspection by NYSDOT Geologists on December 11th. This was less than two and a half weeks after the start of the construction of the new slope and less than two months after the rockfall. The road was repaved and was fully reopened to traffic on Dec 12th. The final excavation quantity was approximately 15,000 cubic yards, and the final slope was very close to the design pay line following existing fracture surfaces. The existing slope was then divided into two rockslopes in the database, the existing slope north of the rockfall, and the newly created slope from the rockfall to its southern limit. The new slope was rerated as a 3.2 Relative Risk, dropping it from second highest rated slope in Washington County to the thirtieth highest.

COSTS

The project was conducted under the Regional emergency contract. The Emergency Contractor had bid only $20 per cubic yard for rock blasting and excavation. The contractor disputed the bid price, as this was not the typical emergency blasting project that the bid was based upon. The Region agreed to pay the blasting sub-contractor's bill for the rock blasting. The total cost for the entire emergency cleanup and reconstruction of the rockslope was less than $871,000 dollars, and it took two months to fully reopen the road.

Figure 14 - Finished Rockslope

SUMMARY

This major rockfall caused a large reaction from both the media and the Emergency Responders. The Emergency Contractor was immediately dispatched and worked continuously to open the roadway to one lane of traffic. DOT geologists responded immediately, providing support to the contractor and Region during the rockfall cleanup. Following the rockfall cleanup, Geology worked with NYSDOT's Survey, Photogrammetry and Instrumentation units to quickly and accurately survey the area and model the rockslope to determine an intermediate response following the rockfall and to design a permanent remediation of the rockslope.

This project incorporated the first use at NYSDOT of Split FX® and Leica TruView ® LiDAR software design tools utilizing both terrestrial and airborne Lidar. The combined LiDAR point cloud was used to determine existing slope conditions, design a remediation treatment and to accurately determine quantities of rock removed during the emergency rockfall cleanup and final remediation. Based upon the success of this project, the use of terrestrial and airborne LiDAR will definitely be incorporated along with these software packages for future NYSDOT rockslope design projects.

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ABSTRACT

During the evening of 2 December, 2012, a large rockslide occurred that blocked US-95 at MP 188 about 5 miles south of Riggins, Idaho. No accidents were reported as a result of the rockslide. The next day, Idaho Transportation Department (ITD) noted a large, unstable, 200-ton block, hanging precariously on the face about 180 feet above the road that required immediate assessment.

The rockslide originated from an ultramafic rock massif cropping out about 250 feet above the road. The team employed rappelling techniques to map vertical scanlines and access the unstable block. While on rappel, the team collected physical and engineering characteristics of the rock mass including information on discontinuities. The field data were used to assess the kinematic relationships between the structure of the rock mass and the rockslope face and establish failure mechanisms. During the assessment, a large and deep tension fracture was observed between the critical block and the hanging wall of the main rock mass. The kinematic analysis demonstrated that the critical block and slope were unstable and wedge failures were dominant.

To mitigate the initial, unstable, slope conditions and critical block, a rock scaling contractor was immediately mobilized to the site. After assessment, the team established that the block should be removed by trim blasting. The team developed a trim blasting design that would bring the block down yet preserve the back wall. They worked closely with the blaster-in-charge and contractor to drill, load, and shoot the critical block. The block was removed safely by presplit blasting on 23 December. US-95 highway was then reopened to the public to accommodate the Christmas traffic.

INTRODUCTION

US-95 is the only north–south highway in Idaho. It stretches along the western border from Oregon to British Columbia over 538 miles. North of New Meadows, the highway descends about 2000 feet, along a steep precipitous canyon, paralleling the Little Salmon River. The highway has been vulnerable to rock slides, soil slides, and debris flows throughout the canyon for years. During the evening of 2 December, 2012, a large rockslide occurred that blocked both lanes of US-95 at Mile Post (MP) 188, about 5 miles south of Riggins, Idaho (Figure 1). According to ITD officials, the last rockfall of this magnitude, to occur in this area, was about four to five years prior to this incident. No accidents were reported as a result of the rockslide. ITD maintenance crews reopened one lane to traffic by midmorning the next day (Figure 2). However, after the rockslide, ITD maintenance personnel noticed a large, unstable, 200-ton block hanging precariously 180 feet up on the face that required immediate assessment, referred to as Block A.

Figure 1: Location of the US-95 rockslide, about 5 miles south of Riggins, CA, at MP 188.

Jacobs Associates (JA) was immediately contacted by Brian Bannan, district geologist for ITD, on 5 December, 2012 to assess the problem. We arrived at the rockslide site on 10 December, 2012. Our team consisted of Brian Bannan from ITD and Jamie Schick and Bill Gates from Jacobs Associates. Because of the exposed rock face and access difficulties, rope access rappelling techniques were used to map the rock slope and access and evaluate the unstable Block A.

Figure 2: Clean-up of several 100 yards of rock debris on US-95 on 3 December 2012. Note maintenance individual next to boulder for scale in right image. Large boulders had to be blasted (photographs supplied by ITD maintenance personnel).

ENGINEERING GEOLOGY

The rockslope parallels the east shoulder of US-95 at MP 188. The foot of the slope is about 650 feet long and strikes to the north. The rock massif is approximately 250 feet high above the shoulder of the highway (Figure 3). The rock mass is composed of foliated Mesozoic metamorphic ultramafic rocks *(1)* consisting of "Greenstone" schist, serpentine, and zones of talc and asbestos. According to the USGS map compiled by Lund *(1)*, foliations dip about 60 degrees to the southeast.

We subdivided the rock mass into four zones based on obvious geologic structures. The upper and middle portions of the rock mass are divided by an oblique shear zone that climbs from south to north at about 30 degrees and ranges from about 100 feet to 30 feet below the brow of the slope. The middle portion of the rock mass rests on an apparent subhorizontal shear zone and pegmatite dike that is about 50 feet above the shoulder of the road.

The rock mass was further subdivided into Blocks A, B, C, and D (Figure 4). In the photograph, Block A is about 70 feet below the brow of the rockslope. Block B forms the hanging wall behind Block A and is bounded on top by a 3 to 4footwide subhorizontal tension fracture between the block and the upper rock mass (Figure 5). A large 5 to 7foot tension fracture separates Block A from the hanging wall of Block B (Figure 6). A separate tension fracture apparently dips to the northeast below Block B and may coincide with the foliation (Figure 6). This tension fracture extends below Block B for at least 40 feet. Blocks A, B, and D rest on Block C, which together are part of a large wedge-shaped block (Figure 4).

The shape of Block A resembled a pyramid (Figure 7), formed by intersecting joints and the rock face. The base measured roughly 10 feet x 13 feet x 20 feet. It was about 30 feet high at the apex. The base of the main block rested on the tectonic shear zone, which consisted of breccia with a matrix of very weak clayey talc. The block was surrounded by loose and unstable cobbles and boulders up to 4 feet in diameter. We assessed the overall strength of the intact rock with a geologic hammer. Based on hammer blows, the rock mass consists of very strong rock (>14, 500 psi). Assuming the rock mass has a unit weight of about 160 pcf, the weight of Block A was about 200 tons.

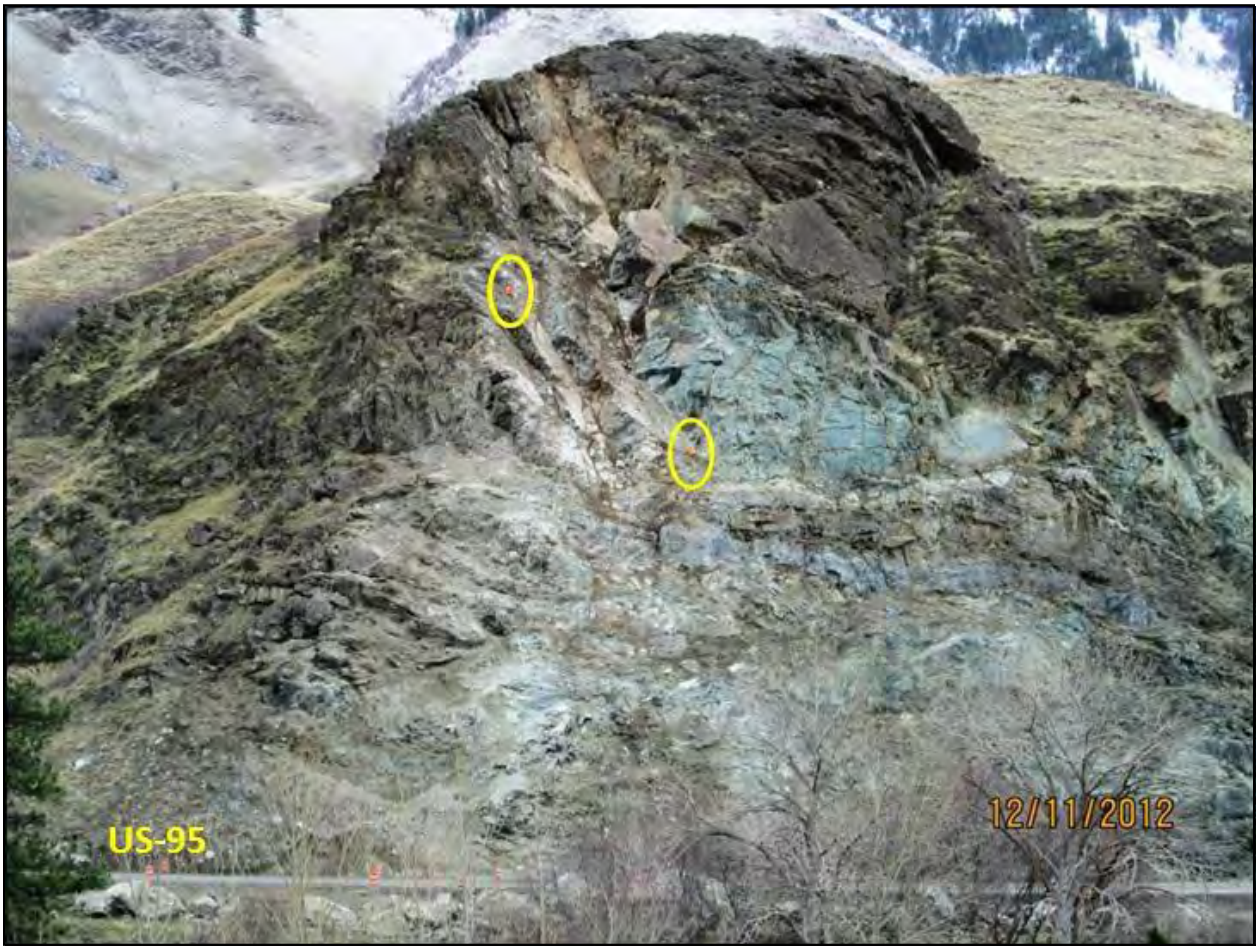

Figure 3: Rockslope source area for the rockslide above US-95. The rockslope is about 250 feet high; rock climbers surrounded by yellow circles provide scale. The rock mass consists of “Greenstone” schist, serpentine, and zones of talc and asbestos.

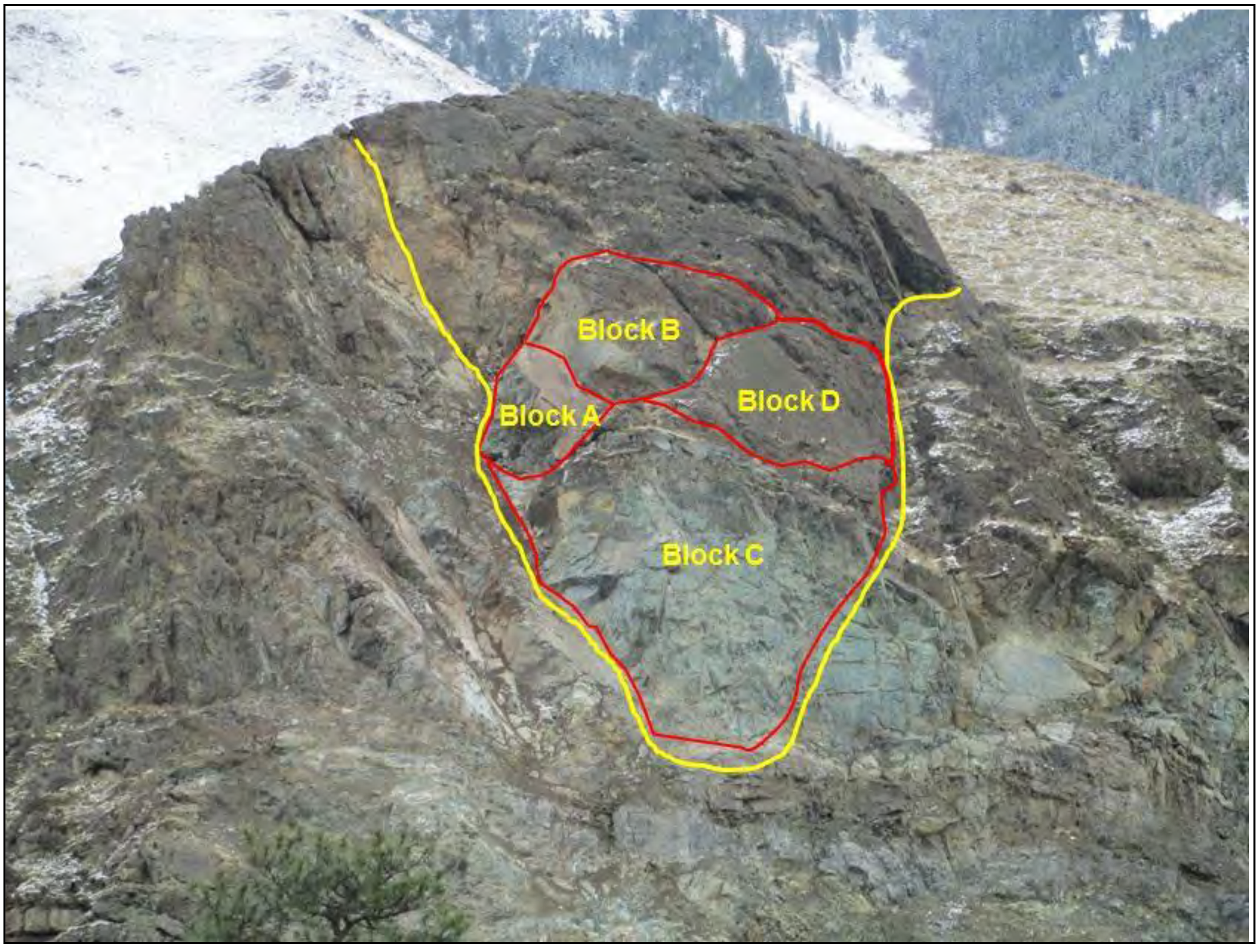

Figure 4: Rock mass subdivided into blocks for reference. Block A was identified as the most unstable and required immediate assessment.

Figure 5: Original tension fractures that have dilated as a result of the rockslide.

Figure 6: Tension fracture between Block A and Block B, hanging wall. Note fracture continuing below Block B.

Figure 7: View of Block A from the north. The arrow is pointing to a tectonic shear that is likely associated with the ledge. The block is approximately 30 feet high and 20 feet wide at the base. Intact rock is very strong and massive. Jamie Schick is for scale.

PROBLEM ASSESSMENT: BLOCK A

The focus of our assessment was Block A because of the severe threat to the highway. We accessed Block A using roped rappelling techniques. While accessing Block A, we collected geomechanical information on the major discontinuities and overall rock mass along the vertical scan lines created from the rappels. Information on the discontinuities was used to evaluate the kinematic relationships between the structure of the rock mass and the rockslope face. Figure 8 is a stereonet developed from RocScience® Dips V6 that displays the rock structure and kinematic relationship of the discontinuities to the rock face.

The rock mass is cut by at least two major joint sets that form prominent wedges plunging west to the road (Figures 4, 8, and 9). Joint Set 1 dips to the southwest at about 50 degrees and serves as the apparent failure plane for the recent failure. Joint Set 2 steeply dips to the northwest at about 80 degrees and forms the hanging wall of the wedges. The face of Joint Set 1 is scarred with recent slickensides from the rockslide that failed to the west. The joint face consists of very slick serpentine. Base friction of the joint face was estimated at 25 degrees based on tilt tests of clasts with similar joint faces and lithology.

As a result of the rockslide, Block A separated and rotated from Block B, moving to the open face where the supporting rock mass was removed by the rockslide that occurred on 2 December, 2012. Consequently, Block A was very unstable. Rotation of the block created a large tension facture by dilation between Blocks A and B (Figures 5 and 6). During our assessment, Block A appeared to be toppling towards the highway. In addition, because of the rotation, it appeared to be sliding in bearing failure through the basal shear zone (Figure 7) and rock mass.

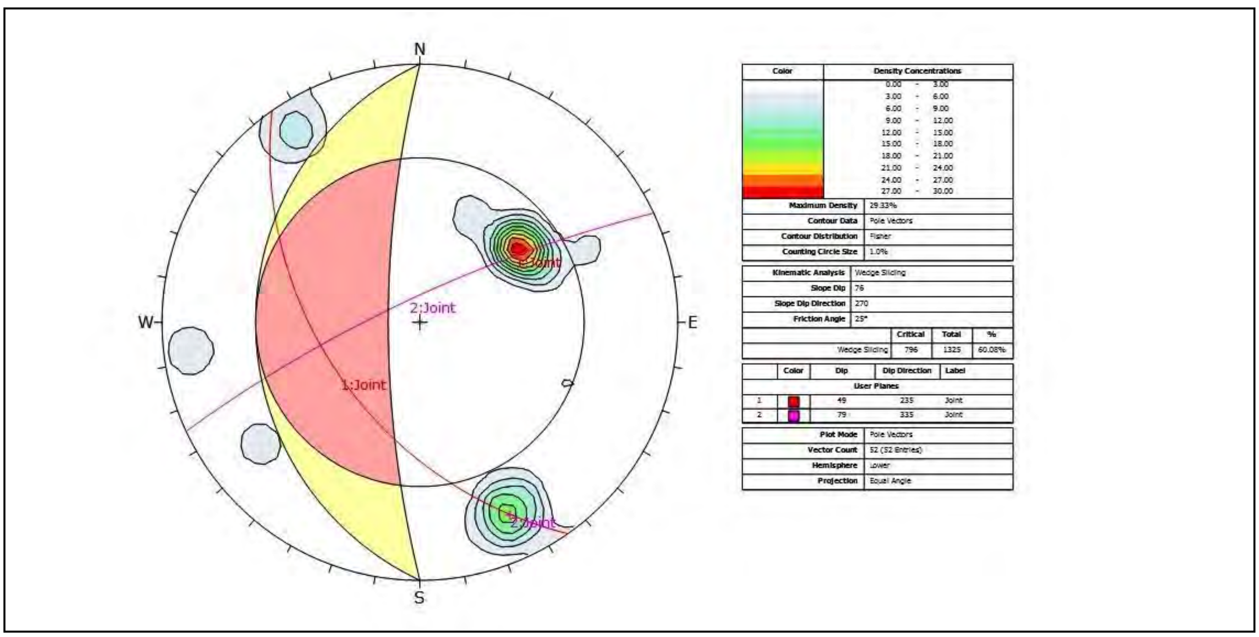

Figure 8: Stereonet displaying the kinematic relationship of the discontinuities and the rock face that created the rockslide. The rockslide failed as multiple wedge failures.

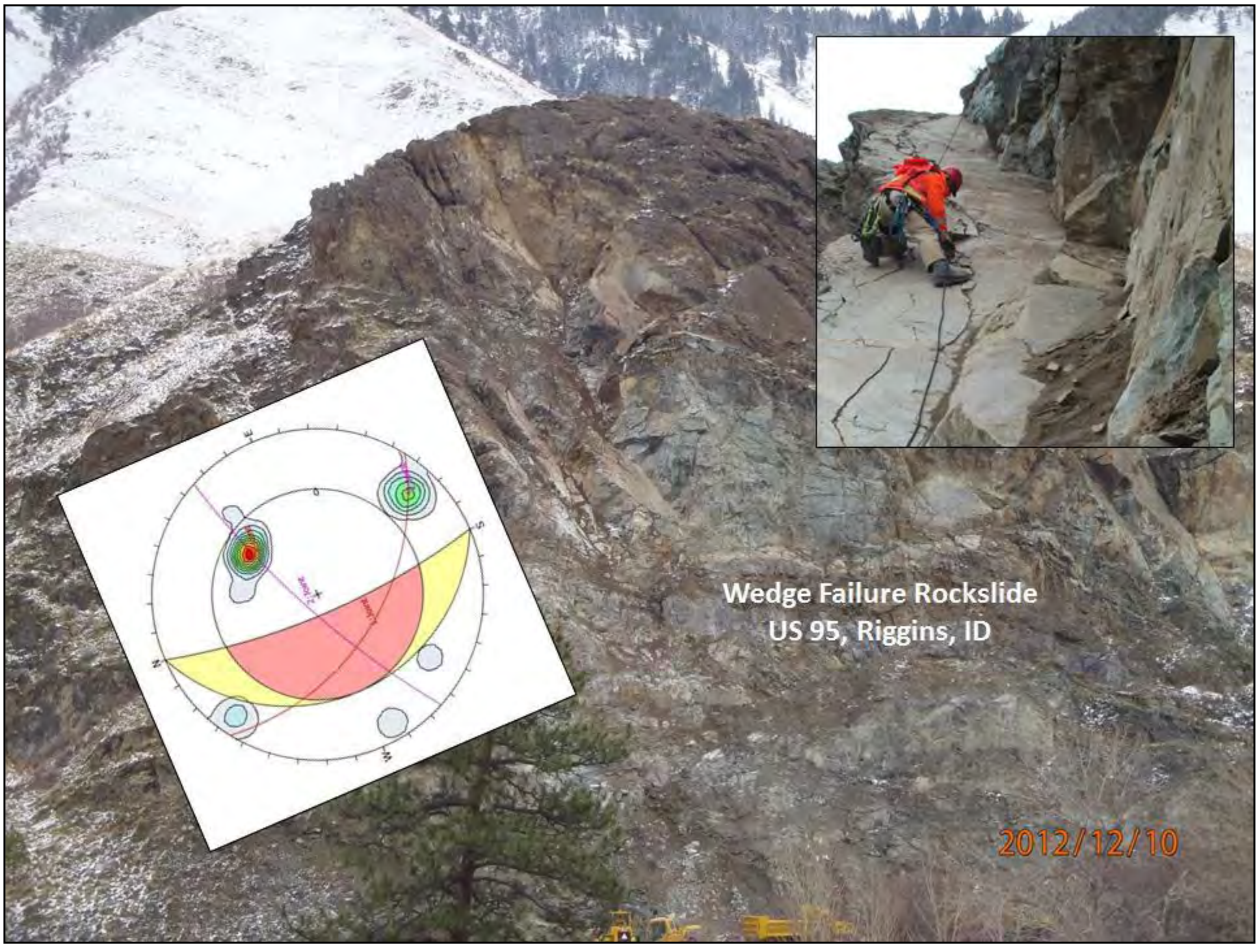

Figure 9: Mapping and kinematic analysis of the rockslope (via stereonet, Dips V6). The steep rock face above the highway dips about 75 degrees to the west.

As displayed on Figure 9, Joint Sets 1 and 2 intersect to form a wedge that plunges west and daylights the steep rock cut face above the highway. The stereonet has been rotated to mirror the rock face. The pink crescent shape represents the critical zone on the rock face; the back portion is the steep rock face, and the outer frontier is the base friction angle of 25^0. The stereonet is oriented west (bottom of page) to represent the rock slope. Note the intersecting joints forming the wedge daylight the rock face, and the plunge is steeper than the friction angle. Therefore, the blocks are kinematically unstable and may fail as multiple wedge rockslides, as occurred on 2 December, 2012 with Block A.

PROBLEM MITIGATION

Our analysis and investigation revealed that Block A was critically unstable and needed to be removed from the slope. In addition, because of the rockslide, there were numerous unstable rocks and debris that required scaling. Initially we assumed that we might be able to remove Block A with the aid of pneumatic matt jacks. However, our investigation revealed that we had a space of 5 to 7 feet between the foot wall (Block A) and the hanging wall (Block B), and therefore it would have been impractical to insert sufficient mat jack to lever the block off of the slope. Thus, our recommendation to ITD was to safety-scale the loose rock around Blocks A and B sufficient to access Block A. Once the loose rock was removed, our plan was to remove Block A by controlled blasting to improve the stability of the back wall using either cushion blasting or presplit blasting techniques.

Based on our recommendation, ITD immediately contacted Midwest Rockfall Contractors (MRC) to mobilize to the site and implement the recommendations. MRC scaled and removed approximately 100 yards of rock before it was able to access Block A.

Figure 10: Midwest Rockfall Contractors

At the same time that MRC was scaling the rock, we evaluated Block A for controlled blasting using cushion (trim) or presplit (preshear) blasting techniques. The principle behind controlled blasting is that closely spaced holes are drilled in a line on the final face and loaded with an explosive charge that is smaller than the diameter of the borehole. The air gap between the charge and the borehole provides a cushion that diminishes the explosive shock wave that penetrates and crushes the rock face. At the same time as the charge is detonated, it creates a shear between the shot holes, and this creates a back wall. The main difference between the two techniques is the back line of holes are fired first for a presplit blast and fired last for a cushion blast. The cushion blast may lead to overbreak, depending on the geologic conditions. Because our goal was to achieve a stable back wall with the hanging wall of Block B, we elected to remove Block A by presplit blasting. Figure 11 displays the blast design layout.

Figure 11: Presplit blasting design for Block A. Blast design was modified from a cushion blast because we wanted to ensure that after the blast we could achieve a clean and stable back wall.

Figure 12: Drilling blast holes behind Block A (left); and loaded blast holes (right). Extensometer constructed from telescoping PVC is visible under the "O" in the left photograph, refer to arrow.

Because of the very strong and hard rock, drilling of the blast holes was difficult and time consuming and it took about 16 hours to drill eight boreholes. The contractor attempted to space the boreholes between 2.0 feet and 2.5 feet. Spacing was controlled by the drilling conditions. To monitor the stability of the block during drilling, MWR installed a "poor man's extensometer," constructed with telescoping PVC pipe across the tension fracture between Blocks A and B (Figure 12). As MWR personnel were drilling the blast holes, they observed at least one inch of movement of across the tension fracture.

Once the blasting holes were drilled and loaded, the blast occurred around noon on 23 December (Figure 13). The presplit blast worked as designed and cleaved the block from the face, leaving a relatively clean back wall (Figure 14). The debris slid down the rock face, creating a manageable muck pile at the base of the slope. The ITD quickly moved the debris muck pile to the edge of the road (Figure 15). The road was then reopened for the 2012 Christmas traffic.

Figure 13: Blasting removal of Block A at 1230 hours 23 December 2012.

Figure 14: Photographs comparing removal of Block A. Reference block is circled in yellow.

CONCLUSIONS

A large rockslide occurred on US-95, failing as multiple wedge failures. The failure planes are composed of very low strength serpentine and talc with a friction value of about 25 degrees. Block A was very unstable, was moving slowly, and appeared to have sheared through the oblique tectonic shear zone. Failure mechanisms for Block A included toppling and wedge failures.

Blocks B, C, and D are kinematically unstable, as suggested by the recently dilated tension. The blocks require further investigation. In the meantime, ITD is planning to monitor the face using LIDAR.

Figure 15: Final cleanup of muck pile on US-95 after blasting Block A.

REFERENCE:

1. Lund, Karen, Geologic Map of the Western Part of the Payette National Forest, West Central Idaho, US Geological Survey, 2004.

Case Studies on Rockfall Mitigation and Rock Slope Stabilization in California, Tennessee, Virginia, and Vermont

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ABSTRACT

Rockfall constitutes a major hazard along our nation's roadways and a nagging liability to our maintenance and engineering departments. Recurring cleanup and repair costs have stressed dwindling maintenance budgets. The extensive nature of the problem precludes repairing and mitigating every possible site, but new and innovative mitigation technologies and contracting techniques can serve to stretch tight budgets.

There are many methods that can be used to stabilize a rock slope. These include altering the slope geometry, installing drains, adding reinforcement, or a combination of these methods. The challenge for engineers is to design a method that can be installed with little or no impact to the traveling public, is expedited through innovative contracting methods, limits the disturbance to environmentally sensitive areas, and maintains an aesthetically pleasant appearance and appropriate service life.

This presentation covers four case studies that highlight innovative technology and innovative contracting methods for rockfall mitigation. The case studies include a project for the United States Army Corps of Engineers near Chowchilla, CA using design/build/warranty contracting and post-tensioned rock bolts with Maccaferri B600 mesh facing; a project for the Tennessee Department of Transportation near Maryville, TN using rock dowels and Geobrugg's high-capacity Tecco® mesh facing; a design/build project for the Virginia Department of Transportation near Hillsville, VA using a shear dowel array encased in reinforced shotcrete overlying a drilled drainage array; and an emergency design/build rockfall mitigation project for the Vermont Agency of Transportation that used a shear key, scaling, vegetation removal, rock dowels, and both wet and dry mix shotcrete.

INTRODUCTION

The key issues in rockfall hazard management are public safety, cost, and effectiveness/reliability of control measures. The evolution of engineering ingenuity, equipment capabilities, and available technologies allowed each of the following case study projects to address those issues while providing stabilization systems that are effective and can also be installed with little or no impact to the traveling public. Three of the four projects were design/build projects at the outset of construction, and the fourth presented an opportunity for a value engineered alternate section that increased the service life of the system.

State Route 73 Rockfall Mitigation near Marysville, TN (TDOT)

The state of Tennessee has a large number of potentially unstable rock slopes adjacent to major transportation routes. These slopes are generally highway rock cuts with raveling or toppling concerns and rock slopes experiencing differential weathering.

State Route 73 (E Lamar Alexander Pkwy) is a highly traveled two-lane road that runs along the Little River and connects the towns of Walland and Townsend. The Tennessee Department of Transportation (TDOT) Region 1 office identified a 650 linear feet section of roadway that posed a high hazard for rockfall, creating a significant risk to the safety of the traveling public. As a result, TDOT advertised a bid letting in January of 2012 to provide rockfall mitigation of the slope adjacent to the roadway. The near vertical cut slope had a height exceeding 80 vertical feet with a construction work area width of only 18 feet and a requirement to maintain one lane of traffic at all times. TDOT also mandated that the asphalt surface be preserved and undamaged during the scaling and rock bolting operations. Soil Nail Launcher, Inc. (which changed its name to GeoStabilization International (GSI) in 2013) was the successful rockfall mitigation subcontractor on the project.

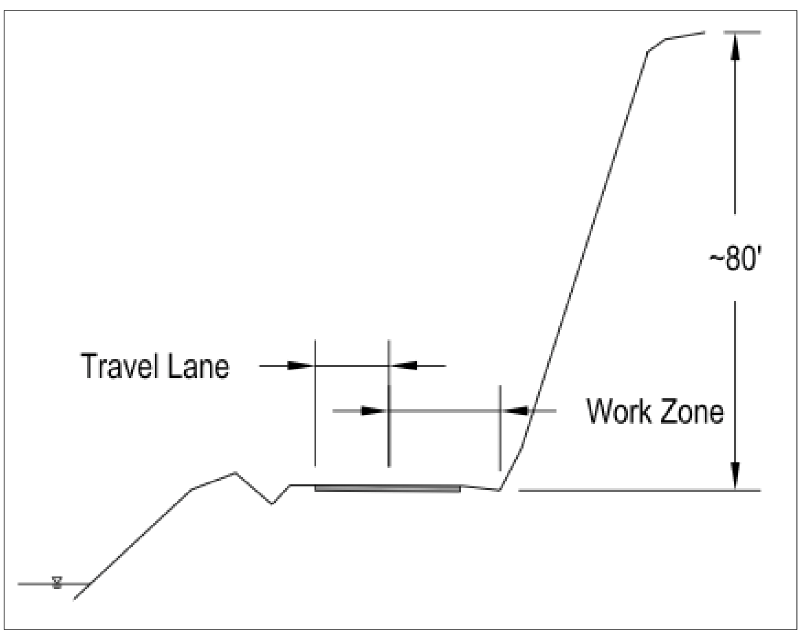

Figure 1 – Slope Cross Section

Jersey barriers were installed between the travel lane and work zone with temporary traffic signals placed on each end of the construction zone to alternate one-way traffic along the 11 feet wide travel lane. Traffic was alternated every few minutes, 24 hours a day.

To preserve the asphalt condition within the work zone, a 12-inch layer of 3/8-inch pea-gravel was placed over the asphalt within the work zone. Scaled material was allowed to accumulate over the pea-gravel and reshaped to create a working platform up to 10 feet above the roadway elevation. This bench created a wider catchment platform for scaled material, decreased the heights the material was allowed to fall, and brought the working area to within reach of smaller more versatile equipment. This increased the project safety and production rates for both scaling and rock bolting operations.

Figure 2 – Work Zone

Figure 3 – Working Platform

Upon completion of the scaling operation, 150 ksi galvanized No. 8 all-thread rock anchors were installed on a 10 foot staggered pattern with embedment lengths up to 27 feet. This was completed by both specialty drilling equipment staged along the working bench and from a crane basket drill capable of working from cranes capable of staging within the 18 feet wide construction area.

Figure 4 – Crane Basket Drilling

The face was draped with a Geobrugg TECCO® mesh with galvanized TECCO® spike plates. Additional spot bolting was also accomplished to ensure the mesh had uniform anchorage and unique blocks and jointing were anchored.

Figure 5 – Mesh Facing

Figure 5 – Shotcrete Facing

During construction, locations of highly weathered sandstone and soil seams were discovered. TDOT and GSI® engineers jointly formulated a plan to cover these areas with steel reinforced shotcrete anchored to the slope with soil nails and rock dowels.

Eastman Lake Rockfall Mitigation near Chowchilla, CA (US Army Corps of Engineers)

Eastman Lake Park is approximately 48 miles north east of Fresno, California in Madera County and is a United States Army Corp of Engineers (USACE) reservoir providing irrigation and flood prevention for local agribusiness. During the early 1970's the hillside along the southern side of the park was cut to its existing grade of 65 degrees (0.5:1) to construct an access road.

The exposed rock on this slope is systematically jointed and has very blocky characteristics. In general the rock blocks are little to moderately weathered, strong to very strong and moderately hard to very hard. Weathering in the rock mass is concentrated along discontinuity planes, where the rock is generally highly weathered to locally completely decomposed. The rock mass is generally closely to moderately fractured, with mean joint spacing of about 3 to 10 feet. The dominant rock structures were characterized by 4 systematic joint sets and the mean orientations of each of these joint sets are presented in Table 1.

Table 1 - Mean Discontinuity Orientations

Discontinuity Set	Mean Orientation (dip, dip direction)	Comments
J1	69, 58	Foliation
J2	79, 300	Joint Set
J3	48, 143	Joint Set
J4	21, 9	Joint Set

The 90-ft tall rock cut slope exhibited episodic failures since its construction and generally occurred following periods of substantial rainfall (Figure 6). Some failures had been large enough to block road passage and raise concerns regarding both impending rock fall hazards and the long term stability of the cut slope.

Figure 6 – Wedge Failure in 2005 Following Substantial Rainfall (Photo Courtesy USACE)

The rock cut slope consists of 980-linear feet of weathered schist and sandstone broken into a lower, middle and upper benches. The lower bench runs the entire 980-linear feet and the middle and upper benches run along 280 linear feet. A concrete v-ditch bench separates the lower and middle section. The average vertical height of each bench is 30 feet. The lower and middle benches are inclined at an average angle of 65 degrees and the upper bench is inclined at an average angle of 45 degrees (Figure 7). Total estimated area of all three benches is 50,000 square feet.

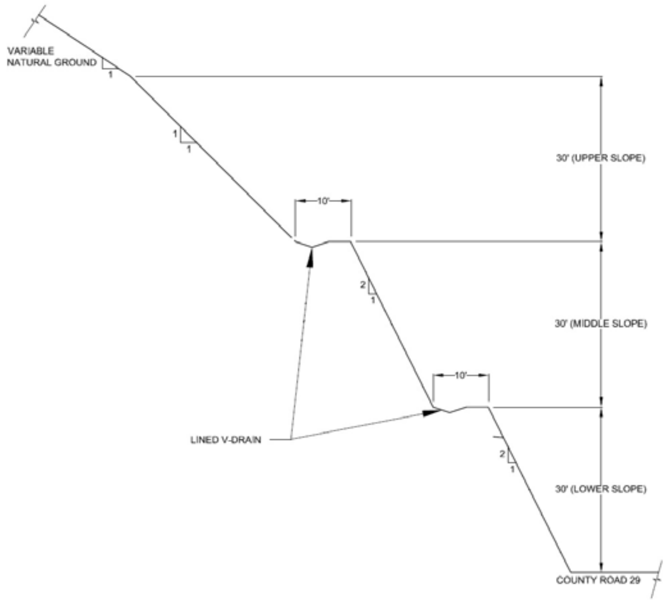

Figure 7 – Slope Cross Section

Following heavy rains during the 1996-97 winter season portions of the slope failed. The USACE immediately commissioned a crew to clear debris from the road and scale the slope. In August of 1997 a consultant was asked to prepare a slope evaluation study to include remediation recommendations. Phase one recommendations called for scaling of all loose rock blocks and talus debris and phase two recommendations included rock bolting and surface protection, adding a debris ditch and barrier, mass grading or abandonment of existing alignment.

Upon receipt of the consultant's recommendations, the USACE sought out a specialty contractor to scale the rock cut slope. Following the scaling effort the slope continued to exhibit failure and in 2005 produced a large wedge failure that completely blocked the access road (Figure 8). The material was removed and no other remediation measures were employed at the time. During the 2010-11 winter season park officials reported that several large rock blocks up to 30-ft in height and 20-ft in width had fallen on the road and were concerned about the apparent dilatation of some of the rock blocks within the slope.

Figure 8 – Failure in 2005 Showing Rock Blocking Access Road (Photo Courtesy USACE)

The rock fall during the 2010-11 winter season coupled with the dilation of rock blocks within the slope prompted the USACE to decide on a remediation plan and proceed with implementation. Cost and time directed the USACE to choose rock bolting and surface protection.

Landslide Solutions, Inc. (which merged with GeoStabilization International in 2013) worked in conjunction with the USACE to optimize the USACE design to minimize time and cost. The friction strength of the rock blocks were not tested, however the average joint friction angle was calculated to be between 20 to 30 degrees. Considering the sensitivity of required bolt spacing to discontinuity strength, bolt orientation and work load, a staggered square bolt pattern with 7-ft spacing between bolts in a single row was recommended. The consultant's report originally assumed a bolt working load of 42 kips with bolts installed horizontally to inclined upward 10 degrees into the hill slope. The USACE chose to use rock dowels with a bond breaker as opposed to "spin-lock" type mechanical rock bolts because of concern that the rock was not hard enough to properly lock the bolts in place. A rock dowel offers a longer grout to rock bond is often able to insure a stronger overall bond in soft rock when compared to mechanical rock bolts. In the time between the consultant's report date and 2011 rock block failures suggested that the original 10-ft and 14-ft recommended bolt length would need to be lengthened to 20-ft and 24-ft. The additional length was determined by size of failed blocks. The working load of each dowel was increased to 59.3 kips and the dowels were installed at a 10 degree sub-horizontal angle to minimize safety concerns and allow for the use of cement grout. A double twist wire mesh cable system with a tensile strength equal or greater than 15,000 lbf/ft and 5,500 lbf/ft transverse strength was selected as the surface treatment.

Construction started in May 2011 with a scaling operation then followed by installation of the rock dowels and Maccaferri's B600 double twist wire mesh cable system (Figure 9 & 10). To date the USACE reports no further dilation or failure of the rock slope.

Figure 9 – Scaling and Rock Dowel Installation in Progress

Figure 10 – Rock Dowel Post Tensioning and Double Twist Wire Mesh Cable System.

Interstate 89 near Burlington, VT (VTrans)

The Vermont Agency of Transportation (VTrans) was notified of a rockfall event on Interstate 89, approximately 15 miles southeast of Burlington, VT during the first week of April 2013. Interstate 89 serves as a critical corridor to the region's economy and a commuter route between Burlington and Montpelier. Individual blocks of rock were weathering out from the crest of two through cuts (four exposed rock cuts) causing some rocks to launch off the mid slope face resulting in rocks landing directly on the highway. The cuts composed of Schist and Metawacke in the vicinity of the rockfall have near vertical jointing with intrusions. The rock is highly fractured along the joints, particularly near the crest of the slope where the rock appears stacked similar to a deck of cards. Root jacking and differential weathering has caused areas to become unstable as well. With multiple launch features along the 100 to 120 ft high rock cuts, significant hazards were posed to the travelling public as depicted in Figure 11.

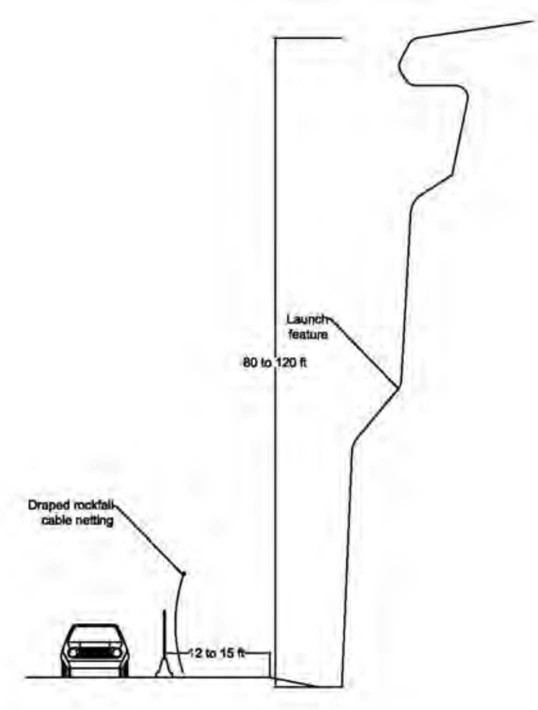

Figure 11– Cross Section of North Cut

GeoStabilization International was asked by VTrans to mobilize to the site and mitigate the rockfall issues through an emergency design-build contract. GSI®'s in-house engineers and rockfall specialists met with VTrans onsite shortly after the rockfall event to determine the best approach. GSI® and VTrans worked together to design a tiered approach to mitigate the issues on the slope: remove vegetation from the slope including trees within 20 feet of the brow; perform light to moderate scaling; install shear key buttresses; install rock dowels; and shotcrete with wet and dry processes. These mitigation techniques were designed and laid out to limit the disturbance to the travelling public.

Figure 12– Scaling Operations with Air Bags

Scaling and tree/vegetation removal required coordination with VTrans traffic control and the sheriff's office. Figure 12 shows a sizeable unstable rock removed with an air bag. A temporary rockfall mesh was draped from excavators at roadway level to reduce the potential for rockfall debris into the adjacent traffic lane. Rolling road blocks were used intermittently to mitigate high risk operations. Many rock features were scaled off the slope; others posed too great of risk to remove and were secured to the slope with shear key buttresses and rock dowels.

In combination with the scaling operation, GSI® placed a thin layer of fiber reinforced gunite (dry shotcrete) to provide temporary stabilization of the rock features. Following gunite placement, GSI® engineers and rockfall technicians designed and constructed buttresses to provide support of cantilevered rock sections near the crest of the slope. Wet shotcrete was used to provide structural facing over the unstable rock masses. Maccaferri's Wirand FS7 steel wire fibers were used in the wet and dry shotcrete. The fiber dosage rate was designed to provide a similar reinforcement of 4x4 W2.9xW2.9 welded wire mesh. Spot rock doweling was installed through the shotcrete areas to secure rock masses to stable areas. Rock doweling was performed using multiple wagon drills and plugger drills. Rock dowels consisted of galvanized #8, Grade 75 ksi, all thread bar, installed in depths varying between 10 and 25 feet.

The mitigation operations were performed safely and within the constraints of the site all while allowing the I-89 southbound lane to remain open to the travelling public.

Figure 13– Drilling Rock Dowels with a Wagon Drill

Figure 14– Shotcreted and Doweled Cut Face

State Route 765 near Hillsville, VA (VDOT)

GeoStabilization International engineers reviewed this site after massive sandstone blocks failed onto the roadway, closing the entire road. Two remediation plans were prepared for VDOT's review: a Geosynthetically Confined Soil retaining wall that would act as a buttress to future failure of the rock slope; and a traditional rock bolting array of the upper unstable rock mass.

Figure 15– Slope After Failure

However, both of those options required both men and equipment to be placed directly below the unstable rock mass. Due to the proximity to Big Reed Island Creek, alternate access options were deemed too costly for repair of this very low volume road. Fisher and Strickler Rock Engineering, LLC was invited to view the site and determined that a shear buttress may be one option to stabilize the unstable rock mass without placing men and equipment in unnecessary danger, and may also fit within VDOTs budget constraints.

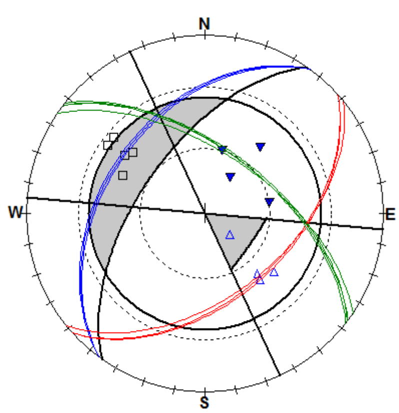

Figure 16– Stereonet Representation of Site (Courtesy Fisher and Strickler)

GSI® engineers then designed the buttress using 2-inch diameter high capacity hollow bars with a yield capacity of 153 kips, installed approximately 4 to 5 feet into the stable rock mass, approximately three feet on center. 75 ksi epoxy coated No. 8 all-thread bars were installed inside the 2-inch tubes to increase the shear capacity and longevity of the system.

Figure 17– Buttress Construction Showing Drainboard and Reinforcing Steel

Approximately three feet of the shear dowels projected into the reinforced concrete portion of the buttress. The reinforced shear key buttress was constructed with 4000 psi shotcrete and multiple layers of 4x4 W4.0xW4.0 welded wire fabric. Drainage was provided behind the buttress to prevent hydrostatic buildup and drilled drains were installed throughout the slope. The shotcrete was finished with a stain similar to the appearance of the native sandstone.

Figure 18– Shear Buttress after Staining

Slope Failure Investigation and Remediation using Geosynthetic Reinforced Earth atop Poor Foundation Soil

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ABSTRACT

The topography between Zoar Valley Road and Cattaraugus Creek exhibits an irregular slope leading from the edge of pavement, down to Cattaraugus Creek to the southwest. The slope below the site includes slumps, flat benches, rills, and gullies. A terrace extends northeast from the site before the topography begins to rise again. These site conditions were consistent with previous geological mapping, suggesting that the Cattaraugus Creek Corridor is rimmed within a morphogenetic region referred to as landslides and slumps.

Design studies revealed that uncontrolled stormwater was the primary cause for the road failure; two culverts discharged onto steep and easily erodible soil adjacent to the road. Additionally, elevated pore-water-pressure in a silt layer and unsuitable organic fill further exacerbated the failure. The remedial design included a Geosynthetic Reinforced Earth Slope (GRE slope) to reestablish the road near original grade within the right-of-way, and subsurface drainage improvements to relieve the high pore-water-pressure in the GRE slope foundation soil. During construction, vibrating wire piezometers were used to monitor the pore-pressures in the foundation soil. Stormwater is conveyed down the slope in a pipe-slope-drain to a non-erosive outlet 100-feet below the road.

INTRODUCTION

Zoar Valley Road is aligned approximately parallel to Cattaraugus Creek, the latter forming the line between Erie County to the north, and Cattaraugus County to the south (Figure 1). The road traverses sections of floodplain, as well as upland areas where the river flows through narrow bedrock gorges. Much of the land in the area is comprised of forest and agriculture, with few small areas of concentrated residential development. The roadway and Cattaraugus Creek elevations are approximately 1240 and 990, respectively.

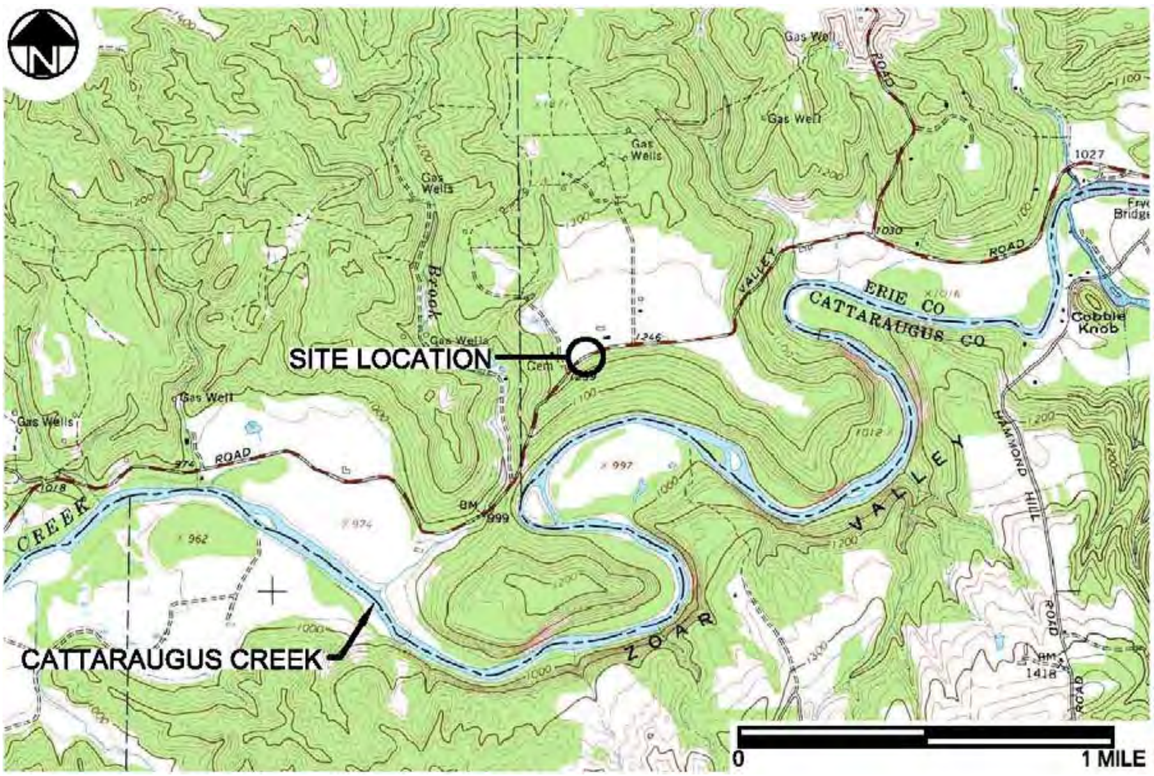

Figure 1 – Portion of the Collins Center and Ashford Hollow USGS Quadrangle Maps showing the site location.

A large portion of the Cattaraugus Creek Basin received a series of intense rain-fall events in August of 2009. Much of the soil in the Cattaraugus Creek Basin was saturated, causing rapid transport of precipitation as surface runoff. Szabo, Coon, and Niziol (2010) provide a narrative of the conditions leading up to flash floods within the basin. They reported as much as 6-inches of rain-fall in as little as 45 minutes, and as much as 7-inches in 24 hours spanning August 9th – 10th, 2009. They showed that the peak flows in a tributary 8-miles from the site had an exceedance probability of 0.2%, which is a recurrence interval of 500-years. These intense rain-fall events lead to the second-highest recorded discharge in the history of the gaging station at Gowanda, NY, which is only 13 river-miles downstream of the site (Figure 2).

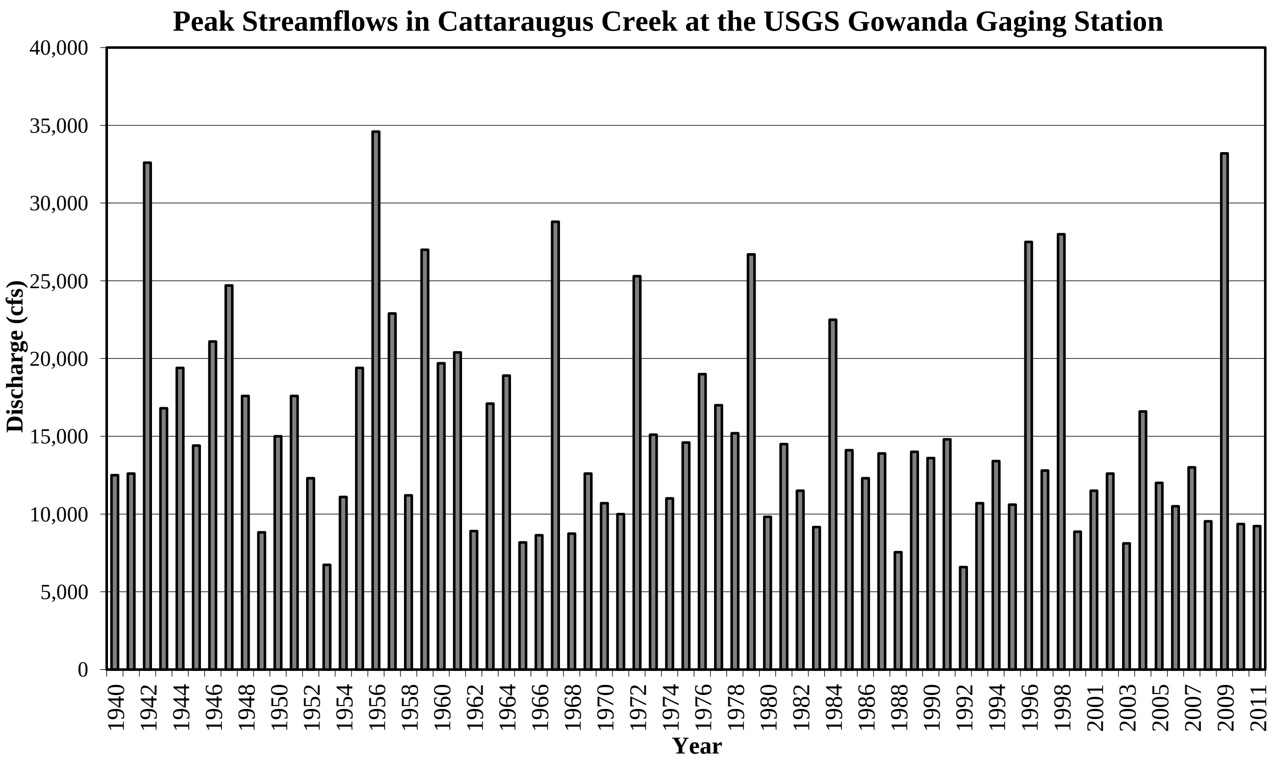

Figure 2 – Time plot of maximum annual discharge past the Gowanda, NY gaging station on Cattaraugus Creek (Data downloaded from USGS Streamflow Data).

REGIONAL GEOLOGY

The geology of southwestern New York dates back to the Devonian period of geologic time, when shale and siltstone sediments were deposited in a basin that resided in the area. The land was thrust upward during subsequent mountain building in the east, exposing the shale and siltstone to subaerial erosion along what is known as the Appalachian Plateau. As shown on Figure 3, semi-parallel, north-flowing drainage channels developed on the exposed shale and siltstone. Muller and Calkin (1993) described that the north-flowing drainage became deranged at the onset of glaciation.

Repeated glacial advance and retreat scoured the drainage channels into deep, north-trending valleys along the margin of the Appalachian Plateau. Muller (1997) provides a concise description of the geomorphic history of western New York, emphasizing the concept of numerous oscillatory retreats and advances of the ice front. The later oscillations filled the valleys with glacial till, lacustrine sediment, and outwash deposits. Figure 4 shows the drainage patterns as they are currently aligned following glacial derangement.

LaFleur (1979) mapped several USGS quadrangles in the upper Cattaraugus Creek Basin, showing that several reaches of the main branch of Cattaraugus Creek flow westward across the north-trending buried valleys. Extrapolation and extension of LaFleur's mapping onto the Collins

Center USGS Quadrangle suggests perhaps the largest valley fill within the upper Cattaraugus Creek Basin; north of Otto, NY. Cattaraugus Creek is incised into bedrock gorges where valley-plugging in the north, and higher till moraines in the south, forced flow over ridges. The channel meanders through broad terraces and floodplains in the valley fill areas, perpendicular to the main-axis of the north-trending valleys.

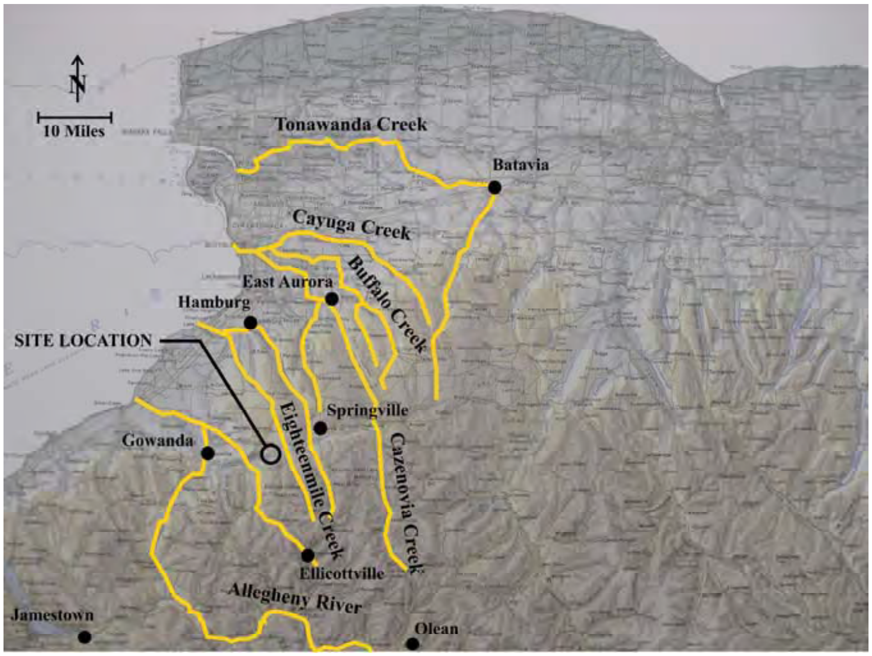

Figure 3 – Pre-glacial drainage patterns plotted on the modern topography of western New York. (Drainage patterns adapted after Muller, 1997; base map adapted from Allan Cartography, 1991).

LaFleur (1979) mapped the Quaternary geology of a corridor of the Cattaraugus Creek Basin, defining the locations of deposits resulting from the Olean, Kent, and Lavery oscillations. The Olean deposits cap the high hills south beyond Ellicottville, NY. Ice during the Kent oscillation did not reach as far and high as the Olean, however, Kent deposits were emplaced on the summits of lower-lying hills. Lavery ice, and therefore the coincident deposits, was constrained within slopes mantled with Kent deposits.

LaFleur (1979) defined the limit of ice-marginal kame, lacustrine, fluvial, and outwash deposits of the Defiance and Valley Heads glacial episodes. He showed that some of these deposits were emplaced on top of the Lavery till.

LaFleur (1979) defined shallow landslides and slumps that have developed on steep slopes within the clayey till of the Lavery. He showed that many of the terraces above the landslide and slump areas are covered by lacustrine and outwash deposits of the Defiance and Valley Heads oscillations.

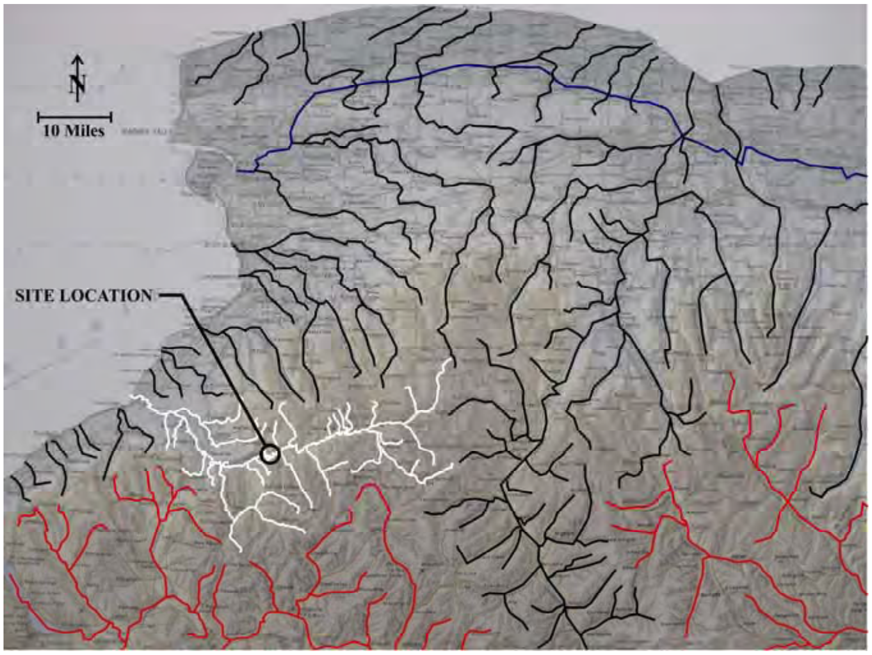

Figure 4 – Modern drainage plotted on the modern topography of western New York. (Base map adapted from Allan Cartography, 1991).

ENGINEERING GEOLOGY

Site investigations were performed to characterize the conditions that led to the Zoar Valley Road failure of August 2009. Historic imagery and mapping were used to understand the recent development of the land. Topographic ground survey was used to document the condition of the ground surface between Zoar Valley Road and Cattaraugus Creek. Exploratory soil borings were advanced to varying depths and a groundwater monitoring well was installed in one of the borings.

Surface Conditions

Zoar Valley Road was close in proximity and parallel to an abrupt slope break at the site of the 2009 failure. There was a broad terrace to the north of the site that was used as agricultural crop fields. To the south, however, an irregular slope extended 250 feet down to Cattaraugus Creek. The irregular slope between Zoar Valley Road and Cattaraugus Creek was forested with trees of wide-ranging maturity.

Slope Shapes

The irregular slope below Zoar Valley Road was comprised of variably steep concave and convex slopes separated by a series of noticeably flatter benches. Minor drainage rills developed on the concave slopes, while gullies with 5-foot vertical head-cuts were more characteristic of the convex slopes. The landowners graded trails and access roads into the slope at several locations, exacerbating locally steep slopes at several locations.

Outcrops

There were several outcrops along the slope between Zoar Valley Road and Cattaraugus Creek. There was a 10-foot exposed section of intact near-shore sand and silt showing cross-stratification within the upper 20-feet of the highway embankment (Figure 5). Two culverts discharged onto the sand and silt, resulting in severely eroded concavities.

Figure 5 – Photo showing one of the culvert outlets that eroded into thinly bedded sand of the sandy gravel unit (culvert is 18-inch diameter).

Below the exposed sand and silt, highly contorted and disturbed silt and clay formed a tongue resembling a mud-flow. There were several exposures of pebbly clayey till within cuts along midslopes, and shale and siltstone were exposed at the toe of the slope within 5-feet of the elevation of Cattaraugus Creek.

Seepage

Several groundwater seepage outbreaks were found along the slope between Zoar Valley Road and Cattaraugus Creek. One noteworthy outbreak was found approximately 20-feet below Zoar Valley Road. This groundwater seepage emanated from an iron-stained gravel layer that was underlain by a clayey silt layer. Seepage from this outbreak was apparently constant, as it was found in the exploratory phase, and it continued throughout construction.

Subsurface Conditions

Exploratory borings revealed three distinct soil units whose bounding surfaces dipped toward the west across the site; construction excavation exposed the units in detail. As shown on Figure 6, the three soils included a sandy gravel unit, overlying a wet sandy silt unit, overlying a clayey till unit.

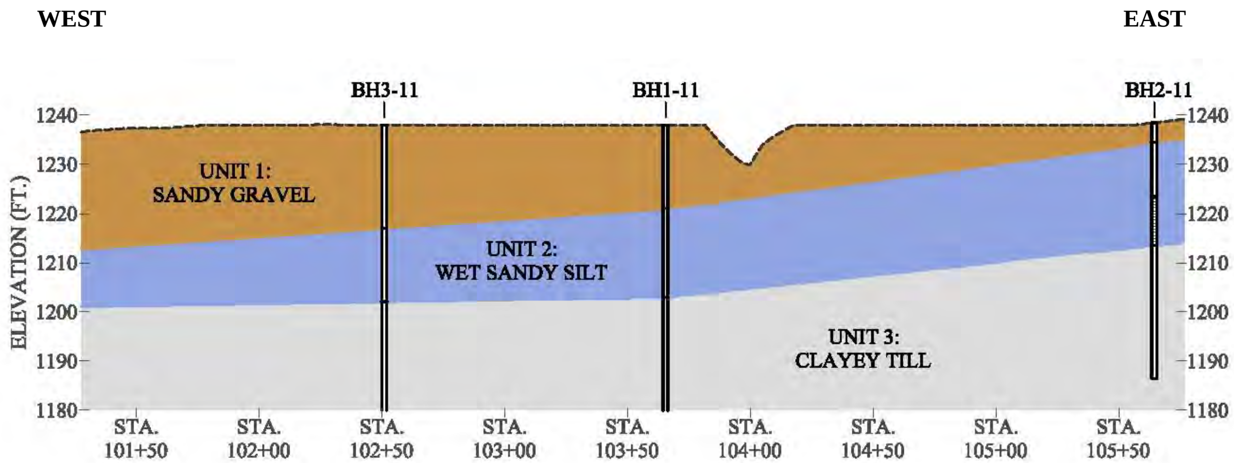

Figure 6 – Schematic profile of the three general soil units identified during the subsurface investigation.

Unit 1: Sandy gravel

Unit 1 ranged from 4-feet thick on the east end of the site to 21-feet thick on the west end. The fabric of the well-graded sandy gravel varied laterally and vertically at the site, ranging from well-graded sand to well-graded gravel, with all intermediary grades included. Some exposures showed strongly dipping, coarsening-upward sequences, while others revealed assemblages of poorly sorted sand and gravel juxtaposed beneath pure sand layers. The overall fabric of Unit 1 suggests an erratic, high-energy depositional environment, such as the outwash and other deposits of the Valley Heads and Defiance defined by LaFleur (1979).

Unit 2: Wet sandy silt

Unit 2 ranged from 15-feet to 20-feet thick across the site. The fabric of the sandy silt was generally homogenous; the only variation was mottling in the upper foot of the unit. The sandy silt was wet throughout the unit thickness, and it liquefied when agitated. The homogeneity of Unit 2 suggests a high fine sediment supply in a low-energy depositional environment. Unit 2 can only be weakly correlated to early Valley Heads lacustrine deposition as defined by LaFleur (1979).

A monitoring well screened in Unit 2 indicated a generally static water level within the upper few feet of the fine-sandy silt. Subsequent monitoring with vibrating wire piezometers revealed that the head pressure in the fine-sandy silt varied in correlation with precipitation events and loading/unloading cycles.

Unit 3: Clayey till

The thickness of Unit 3 was not determined because it extended below the level of all exploratory borings advanced at the site. Generally, Unit 3 was stiff silty clay with about 15% fine to medium gravel pebbles. There were two silt zones within the clayey till, both thicker than two feet, found at depths of 55-feet and 95-feet below the original ground surface. These silt zones were wet and the soil liquefied when agitated.

The fabric of the till matrix was unique in that it was comprised of laminations along silt partings; this suggested that the unit did not fulfill the definition of glacial till, namely that till is deposited in direct contact with an ice mass. Rather, the laminations and silt partings suggest deposition of a diamict in a low-energy basinal environment. The authors chose to consider Unit 3 as a till to be consistent with previous literature. Therefore, Unit 3 correlates best with the Lavery till defined by LaFleur (1979), but may also include several oscillations of the Kent complex.

ENGINEERING DESIGN

During wet times of the year, such as the severe storms of August 2009, the upper sand and gravel deposits became saturated, thereby exerting additional load to the underlying fine-sandy silt. Furthermore, concentrated discharge onto unlined culvert outlets caused erosion on the easily-erodible fine-sandy silt. The failure along Zoar Valley Road occurred where the fine-sandy silt was too weak to support the increased weight of the saturated soil, especially where there had been a loss of toe support.

Consideration of Remedial Options

Stabilizing the problematic conditions required creating a stable base upon which to reconstruct the road embankment. Traditionally, the stability of slopes is improved in four ways, including modifying the slope geometry, improving surface and subsurface drainage, internal slope strengthening, and constructing a retaining structure. Actual slope stabilization solutions may incorporate a combination of the various methods.

Slope Geometry

Modification of slope geometry to improve stability traditionally includes flattening the slope, adding weight to the toe of slope, or removal of weight from the top of the slope. Shifting the roadway away from Cattaraugus Creek to flatten the slope would have resulted in permanent property acquisition, which was not acceptable to the owner. Adding weight to the toe would have required constructing an embankment on the slope between Zoar Valley Road and Cattaraugus Creek. This option was eliminated due to concerns of placement of additional load atop steep and potentially unstable slopes. Removing weight from the top of the slope would require lowering the road, which was considered, but was eliminated due to vehicle line-of-sight and permanent property acquisition.

Surface and Subsurface Drainage

Improving drainage is an important factor in almost all slope stabilization problems. Seasonally and episodically high groundwater levels in the sandy gravel added weight to the soil, thereby increasing the stress within the soil mass. Minimizing the amount of infiltration into the sandy gravel was important to increasing the stability. Furthermore, it was important to reduce erosion of the sandy silt caused by concentrated surface flow.

Internal Slope Strengthening

Internal slope strengthening involves improving the shear strength and reducing the compressibility of the soil within the failure zone. Common examples of this include installing stone columns through the failure zone, deep mixing through the failure zone, or using reaction blocks and anchors to compress the soil and increase its strength and resistance.

Stone columns and deep soil mixing were evaluated but the cost proved prohibitive at this site. Reaction blocks and anchors were rejected because the soil was too weak to provide sufficient reaction and the potential for creep to occur along anchors installed in clayey soils.

Another mechanism considered for internal slope strengthening was excavating and replacing the wet fine-sandy silt with a stronger material. This would have entailed removing the overburden sandy gravel in addition to the 15 to 21 foot thick deposit of the weak wet sandy silt. This excavation would have been extensive and difficult. This concept was considered an acceptable alternative for further evaluation.

Retaining Structures

Retaining walls are often used to provide lateral resistance against slope movement. Driven sheet piles or drilled in soldier piles and lagging are the two types of walls that were considered feasible at this site, however any structure would have to be deep to provide the necessary resistance. This would involve driving sheet piles or drilling soldier piles through the wet sandy silt and into the denser glacial till. Due to the low strength of the sandy silt and the lack of toe support due to the steep slopes, the wall would have to be tied-back with anchors. Because of concerns about stability during construction and the length and expense associated with the anchors, these walls were not considered a viable alternative.

Another type of retaining structure considered was a geosynthetically reinforced earth slope (GRE slope). GRE slopes use reinforcement to stabilize slopes and retain the soil on steep slopes. The steep face allows removal of soil loads from the crest of the adjacent slope while allowing the alignment of the roadway to remain unchanged. In addition, GRE slopes are typically constructed using granular materials that allow free drainage. Based on these advantages, GRE slope was the selected alternative for further evaluation.

Slope Stabilization Approach

The original approach was to excavate and remove the sandy silt and reconstruct the roadway embankment with suitable material. Engineering estimates indicated that approximately 10,000 cubic yards of sandy gravel and 15,000 cubic yards of sandy silt would have to be removed. The owner requested limited truck traffic on Zoar Valley Road to minimize damage to the highway. As such, the approach was modified to accommodate the owner's request.

The modified approach included removing the sandy gravel, improving the strength of the upper portion of the sandy silt, and then constructing GRE slope to serve as the highway embankment. Stabilizing the upper surface of the sandy silt minimized the amount of material transported off the site and re-using the sandy gravel as embankment fill minimized the amount of material transported onto the site.

Design and Construction

The intentions of the design were to control surface water runoff, minimize erosion, reconstruct the road embankment without increasing the weight exerted on the underlying sandy silt, improve the underlying sandy silt, improve subsurface drainage, and reconstruct the highway within the right-of-way. Figure 7 shows a typical section through the reconstructed roadway embankment. Construction commenced in July and was completed in December 2011. The following describes the main points of the design and construction.

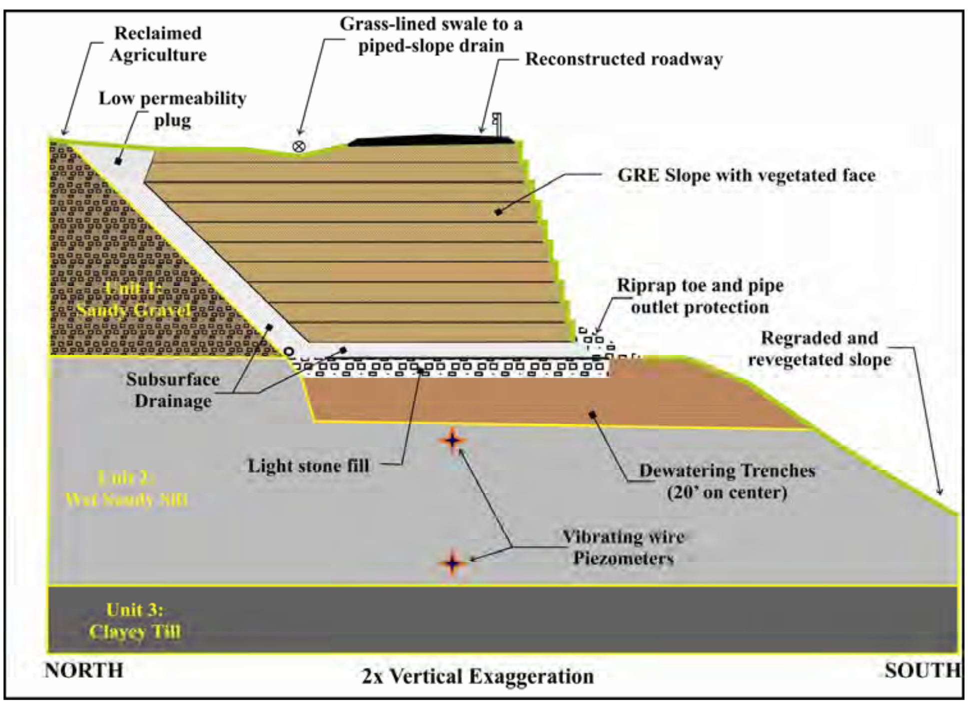

Figure 7 – Typical schematic section showing the surface and subsurface improvements made to reconstruct Zoar Valley Road.

Excavating the Sandy Gravel

The design included excavation of the sandy gravel layer down to the sandy silt layer. Soil with less than 5 to 10 percent fines would be considered suitable for reuse. Figures 8 and 9 show the typical excavated soil considered suitable for reuse. Unsuitable soil was transported to private property located approximately 0.5 miles from the site and was used as fill.

Figure 8 – Photo showing strongly dipping beds of rhythmically-bedded sandy gravel (field book is 7-inches tall).

Figure 9 – Photo showing rhythmic beds (shovel is 10-inches tall).

Slope Grading

Regrading the slope below the elevation of the GRE slope was designed to minimize the potential for concentrated flow. Regrading was limited to cuts and minor rework that resulted in less than 12 inches of fill in any area. All regraded areas below the GRE slope were seeded and covered with permanent erosion mat.

Improving the Sandy Silt

The design consisted of a network of dewatering trenches excavated into the upper portion of the wet sandy silt to drain the silt and improve its foundation characteristics. The design consisted of 3 foot wide by 5 foot deep trenches in the sandy silt (Figure 10). The excavated sandy gravel, generally meeting filter criteria requirements, was used to backfill the trenches to the proposed subgrade elevation. The base of the excavation was covered with a separation geotextile and a layer of crushed rock to provide a platform for construction of the GRE slope.

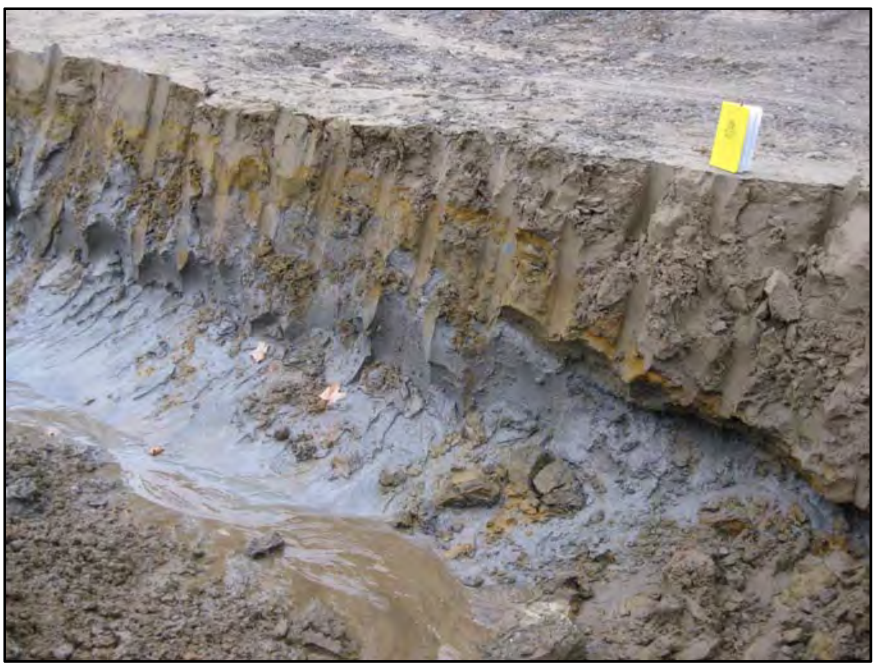

Figure 10 – Photo of a dewatering trench excavation into the sandy silt. The lower 10 – 15 feet of the sandy silt was gray, while the upper 1 – 2 feet was mottled reddish brown (field book is 7-inches tall).

Subsurface Drainage

Following excavation, a blanket of stone was installed on the base of the GRE slope and on the back-slope of the excavation to collect and transmit subsurface water to stabilized outlets.

The subsurface drainage blanket was terminated approximately 3 feet from final grade to prevent surface water from entering the subsurface drainage system.

Geosynthetically Reinforced Earth Slope

The design consisted of constructing a vegetated GRE slope to allow reconstruction of the road close to its original alignment. The GRE slope consisted of wire mesh form-work, primary geosynthetic reinforcement, and secondary geosynthetic reinforcement. The GRE slope was constructed using approved on-site soils supplemented with off-site structural fill. Approximately 70 percent of the GRE slope wall was constructed using on-site material. Design calculations indicated that the primary geosynthetic was required to be approximately 20-feet long. However, the primary geosynthetic was extended to 40 feet to provide coverage under the entire width of the roadway.

Surface Drainage

The design accounted for directing surface water into drop inlet structures, and conveying the runoff to a stable outlet structure at a bench location approximately 100 feet below the pavement elevation. An energy dissipation device at the outlet was used to disperse the flow. Other measures implemented included vegetated swales, riprap-lined culvert inlets and outlets, and permanent erosion mat.

Difficulties during Construction

Difficulties during construction included working on and in the wet sandy silt, working on the downgradient slopes, and encountering unsuitable material.

Pore-Water Pressure in the Wet Sandy Silt

Dealing with the sandy silt became difficult as the sandy gravel was removed. Removing the sandy gravel decreased the confining stress allowing pore-water pressure relief of the wet sandy silt, resulting in heaving and pumping.

A blanket of light stone fill was compacted into the surface of the wet sandy silt to allow access for excavation of the dewatering trenches. The dewatering trench locations were revised to target the wettest areas. The sides of the dewatering trenches slumped within several minutes of excavation, so the trenches were immediately backfilled with drainage stone.

Since unloading of the wet sandy silt resulted in shallow-surficial instability, the concern became reloading the entire mass of wet sandy silt too quickly, potentially resulting in slope instability. Vibrating wire piezometers were installed at various depths and locations to monitor pore-pressures during embankment reloading. Data from the four piezometers showed similar trends.

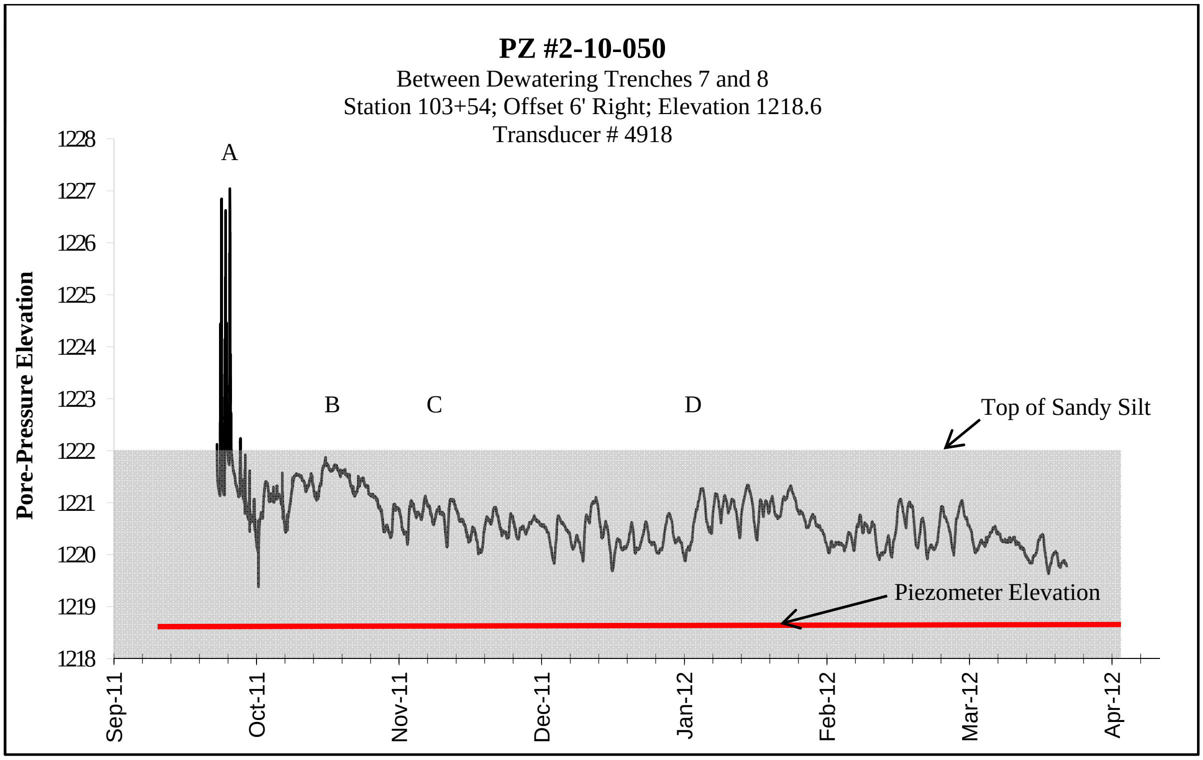

Figure 11 – Hourly time plot of pore-water pressure during and following construction. The three spikes at (A) were recorded during work in the sandy silt unit. Pressure elevations during embankment construction (B) were constrained below the top of sandy silt. After the completion of construction (C), and the five months following (D), the pressure elevations responded directly to precipitation events.

Data collected from one of the piezometers is shown in Figure 11. The data shows that the pore-water pressure spiked during installation of the dewatering trenches and placement of the light stone fill, but that it quickly recovered to pre-disturbance levels. Following installation of the separation geotextile and stone leveling pad, the pore water pressures responded primarily due to the weight of stormwater infiltrating into the backfill; therefore construction progress was not impeded by embankment reloading. After construction was completed, the pore-water pressures rose only due to precipitation events.

Downgradient Slope

Equipment on the downgradient slope was limited due to unstable soil. The weight of the construction equipment on the top of previously sloughed soils resulted in further sliding down the slope. The design was modified to allow for installation of a granular road to provide access for clearing and grubbing. During minor grading on the slope, excess groundwater seepage was encountered. Drainage blankets were excavated into the slope and filled with granular material to allow free drainage.

Unsuitable Organic Material

Woody debris was encountered at the proposed base and face of the GRE slope. Test pits indicated that the unsuitable organic material extended under the proposed GRE slope by approximately 7 feet. It was not feasible to excavate and remove the unsuitable organic material from the site. The design slope of the GRE slope was instead steepened to avoid the unsuitable organic material.

SUMMARY

Surface and subsurface observations of the site were used to extend previous geological mapping. This information, combined with data regarding an intense storm sequence, helped identify the causes of damage that Zoar Valley Road suffered in 2009. A comprehensive understanding of the causes of the slope failure provided the designers with the information necessary to tailor the repair to the site conditions. A combination of slope stabilization techniques were selected for the repair, while several other techniques were determined to be infeasible due to the site conditions. The conditions at the site were monitored during reconstruction to limit the potential for induced slope instability. Several of the selected slope stabilization techniques are shown in Figure 12.

Figure 12 – Photo of the completed slope stabilization.

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A BUMP IN THE ROAD – REMEDIATION OF THE SR 87 LANDSLIDE

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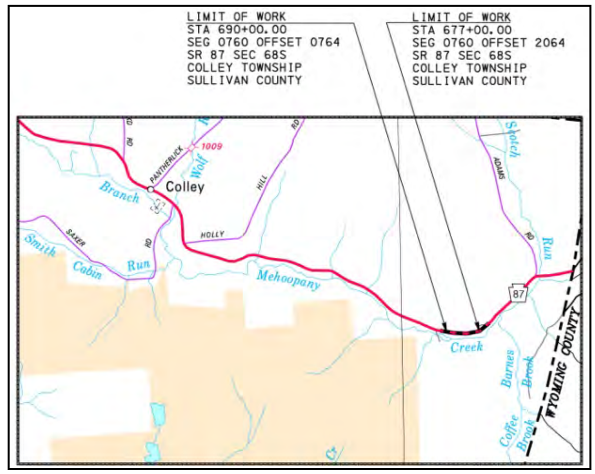
ABSTRACT

An ancient landslide along SR 87 in northern Pennsylvania re-activated in 2011 after the toe of the hillside was eroded due to flooding of the North Branch of the Mehoopany Creek during Hurricane Irene and Tropical Storm Lee. Slope movement damaged the SR 87 roadway creating a large bump, tension cracks, and rough roadway conditions throughout the landslide area. Published literature indicated that the project area was underlain by a glaciolacustrine deposit from the Pleistocene Age. An extensive subsurface exploration program, consisting of 33 borings and 6 test pits, was performed to determine subsurface conditions at the project site. Comprehensive laboratory testing was performed on soil samples collected during the subsurface exploration to estimate engineering properties of the glaciolacustrine material. Laboratory strength tests included direct shear with residual measurements and triaxial shear. Inclinometers were installed in eighteen borings and piezometers were constructed in nine borings. The Pennsylvania Department of Environmental Protection identified an exceptional value wetland within the upper portion of the landslide and preferred that the landslide remediation not affect the wetland. Of the multiple remediation alternatives considered, the selected alternative preserved the exceptional value wetland and included relocation of the creek, construction of a soil berm at the toe, and reconstruction of SR 87 along the roadway's existing alignment. Unique aspects of the project included varying varve orientations and thicknesses within the glaciolacustrine material. Inclinometer and piezometer readings continue to be obtained to monitor the area.

INTRODUCTION

An ancient landslide along SR 87 in Sullivan County, Pennsylvania, approximately one mile west of the Wyoming County border, re-activated in 2011 after the toe of the hillside was eroded due to flooding of the North Branch of the Mehoopany Creek during Hurricane Irene and Tropical Storm Lee. Maps showing the location of the project are provided in Figures 1 and 2.



Figures 1 and 2: Project Location Maps

SR 87 in this area traverses through a narrow valley, and is located along the North Branch of the Mehoopany Creek, which flows to the east. The landslide damaged the SR 87 roadway creating a large bump, tension cracks, and rough roadway conditions throughout the landslide area. Periodic roadway maintenance consisting of pavement patching and milling was required to restore rideability and warning signs were installed to caution oncoming motorists of the bump/rough roadway conditions at the project site (see Photo 1). Landslide remediation was completed in March 2013.

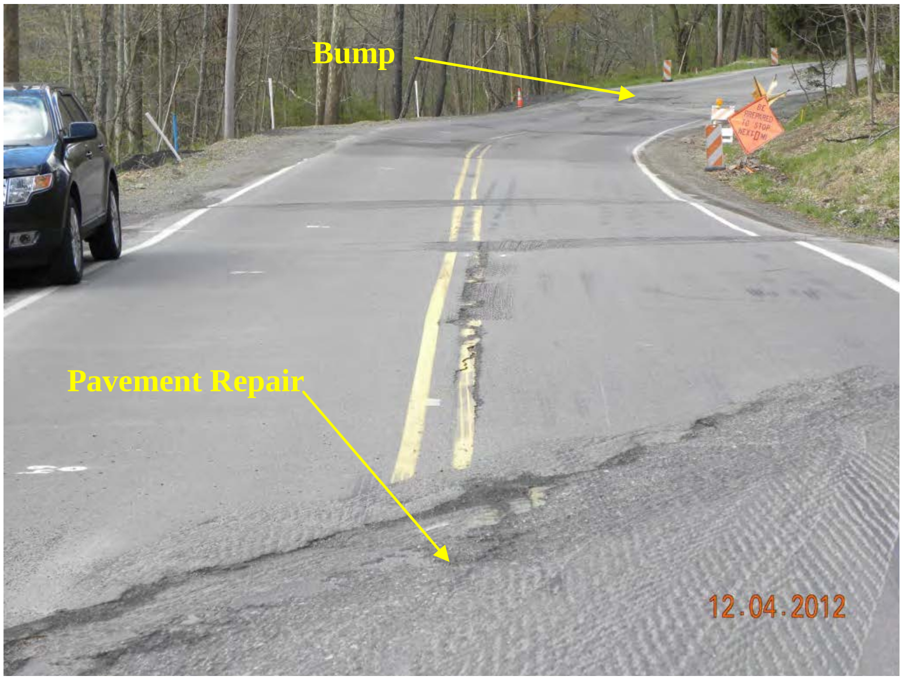


Photo 1: Bump/Rough Roadway Conditions Through Landslide Area

PROJECT HISTORY

PENNDOT performed a field view of the project area in 2011 based upon reports of a "bump" on the SR 87 roadway. Anecdotal information provided to PENNDOT personnel by local residents indicated that the bump on SR 87 had always been present at this location, but appeared to becoming larger. The field view revealed that the bump was likely associated with slope instability since tension cracks were observed within the bump area and approximately 200 feet to the north in the SR 87 pavement.

PENNDOT installed inclinometers in June and July of 2011 to monitor slope movement at the project site. The inclinometers immediately verified that a landslide was active in this area. PENNDOT indicated that the inclinometer readings showed accelerated slope movement after the North Branch of the Mehoopany Creek experienced severe flooding due to Hurricane Irene and Tropical Storm Lee in August and September of 2011. The accelerated slope movement was associated with the erosion of a significant amount of material at the toe of the landslide during the flooding. The alignment of the creek makes a hard bend into the hillside at the project area. Consequently, the bump on SR 87 became more pronounced and more/wider tension cracks were observed within the project area, including on the hillside upslope of SR 87, which resulted in PENNDOT providing warning signs for SR 87 motorists traveling through the project area.

FIELD RECONNAISSANCE

Field reconnaissance of the project area was performed in April 2012. Within the active landslide area, several tension cracks were observed upslope of SR 87 and within the SR 87 roadway, including large tension cracks at the bump in SR 87. Pavement markings had shifted downslope. Trees on the hillside were distorted due to slope movement and the ground surface was hummocky. Many seeps were present within the landslide area and a significant amount of water was observed flowing off of the hillside onto the shoulder of SR 87. The North Branch of the Mehoopany Creek is located at the toe of the landslide and it was evident that the creek had eroded the toe of the hillside during high flow events as shown in Photo 2.

Photo 2: Erosion at Landslide Toe

The topography observed beyond the active landslide area consisted of a series of low-height, relatively steep slopes and flat areas that were fairly consistent and wide spread throughout the area. The low-height, relatively steep slopes were oriented in an arc shape that resembled a head scarp of a landslide and the entire area was hummocky. These landforms were evidence that the area surrounding the current slide area had experienced slope stability issues in the past and the current landslide was a small portion of a much larger ancient landslide.

REVIEW OF PUBLISHED LITERATURE

Based on the Pennsylvania Bureau of Topographic and Geologic Survey the Catskill (Dck) Formation underlies the active landslide area, and the contact between the Catskill and Huntley Mountain (MDhm) Formations is located upslope of the active landslide area (*1*). The Catskill Formation is Devonian in age, while the Huntley Mountain Formation is late Devonian and early Mississippian in age. The Catskill Formation is a complex unit consisting of grayish-red sandstone, siltstone, and shale generally in a fining upward sequence. Gray sandstone and conglomerate are also present. The Huntley Mountain Formation is composed of two sandstone sequences. The upper unit consists of tan to olive quartzitic sandstone with some shale and mudstone interbeds, while the lower unit consists of gray to tan argillaceous sandstone containing some interbedded shale and mudstone (*2*). A Geology Map is shown in Figure 3.

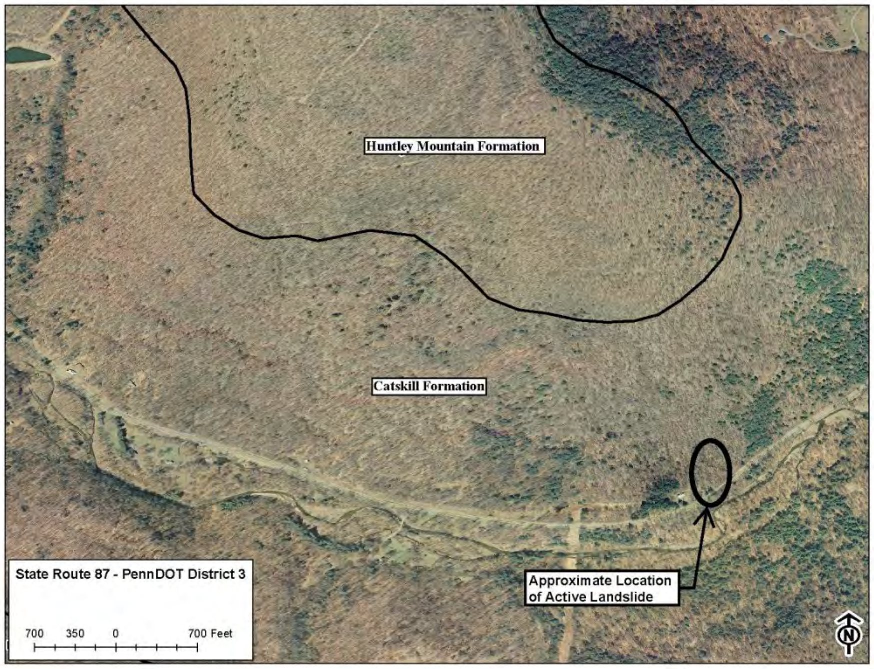


Figure 3 – Geology Map

The Pennsylvania Bureau of Topographic and Geologic Survey indicated that the majority of landslides in the study area occurred in the Huntley Mountain, Catskill and Lock Haven Formations and the transitions between them. The publication provided 13 inventoried landslides located within 2 miles of the active SR 87 slide, with 4 of the inventoried landslides being located in the immediate vicinity of the project site. Each of the inventoried landslides is located within the narrow valley that SR 87 traverses alongside the North Branch of the Mehoopany Creek. Furthermore, the publication classified the project location as a Moderate-Susceptibility Zone to landslides based on the past landslides identified in the area and the geologic and topographic conditions of the area may lead to future landslide activity (*3*).

Based on the Pennsylvania Bureau of Topographic and Geologic Survey, the surficial geology consists of Glacial Till and Lake Sediments (Varves), Undivided from the Pleistocene Age. The glacial till contains a poorly sorted mixture of clay, silt, sand, pebbles, cobbles and boulders and is either interbedded with or overlies the lake sediment varves. The lake sediment varves are composed primarily of silt and clay that settled out of the glacial lake waters. Each pair of silt and clay varves represents an annual deposition cycle as the silt-sized particles were deposited during the summer melt season and clay-sized particles were deposited during the winter freeze season (*4*).

The Pennsylvania Bureau of Topographic and Geologic Survey further indicated that the deposits of till overlying lake sediments or interbedded with lake sediments occur on the sides of the valleys that drain north or east, which the North Branch of the Mehoopany flows to the east. The topography of these deposits is described as having a steep bedrock slope at the top of the valley that becomes gentler where the hillside consists of till. The gentle slope continues to a point where the slope steepens and descends to the valley floor. The steep slope above the valley floor is generally a few tens of feet high and often has a stepped surface, which are scars of ancient slides. Some of the slumps become active where the slope is undercut by either a stream or human activity. The published description of the topography accurately describes the conditions observed at the project site during the field reconnaissance (*4*).

Aerial photographs and Light Detection and Ranging (LiDAR) data, were downloaded from Pennsylvania Department of Conservation and Natural Resources (DCNR) PAMAP system to identify features that illustrate signs of slope instability, or features that could cause instability of the hillside. The aerial photos reviewed were taken between 1939 and 2008 (*5*). The photographs do not show any evidence of slope instability. However, when the LiDAR hillshade is incorporated onto the photographs to visualize the terrain at the project site, slope instability features are evident. The hillshade plot indicated that the active landslide is a small slide within a much larger area of instability as evidenced by what appears to be a head scarp located approximately 1,000 to 1,500 feet upslope of SR 87 and extends approximately 1.2 miles to the west of the active landslide. Additionally, the ground within the area of instability appears hummocky, especially when compared to the ground surface beyond the area of instability. The hillshade plot is shown in Figure 4.

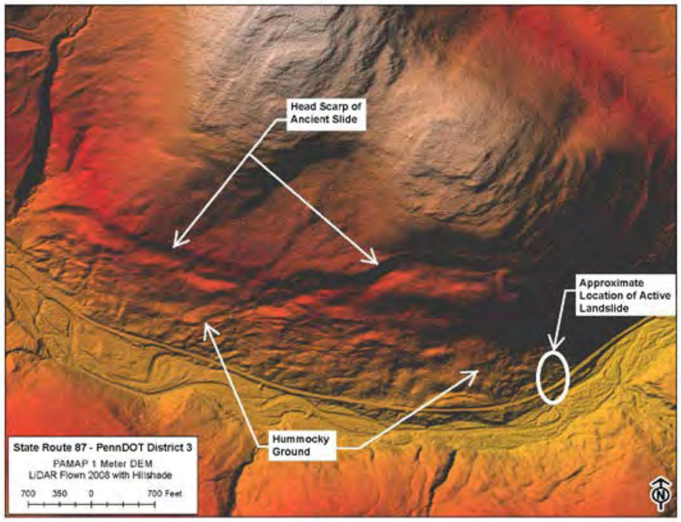


Figure 4: Hillshade Plot Showing Evidence of Slope Instability

EXISTING GEOTECHNICAL DATA

PENNDOT provided geotechnical data from a subsurface exploration program conducted in June and July of 2011. The subsurface exploration consisted of ten (10) borings, where inclinometers were installed in eight (8) borings to monitor the progression of the landslide and to determine the location of the failure plane. The inclinometers identified the location of the failure plane and generally indicated that the failure plane occurred within a glaciolacustrine deposit, near the interface with an underlying stratum of glacial till.

SUBSURFACE EXPLORATION

Four separate subsurface explorations were conducted at the project site and a total of thirty-three (33) borings and six (6) test pits were performed to determine the subsurface conditions, collect soil samples for laboratory testing and for installation of inclinometer casing and piezometers in the area of the landslide between June 2011 and November 2012. The borings were conducted on the hillside and the test pits were conducted in the valley floor. The boring and test pit locations are shown in Figure 5 and a typical subsurface cross section is included in Figure 6.

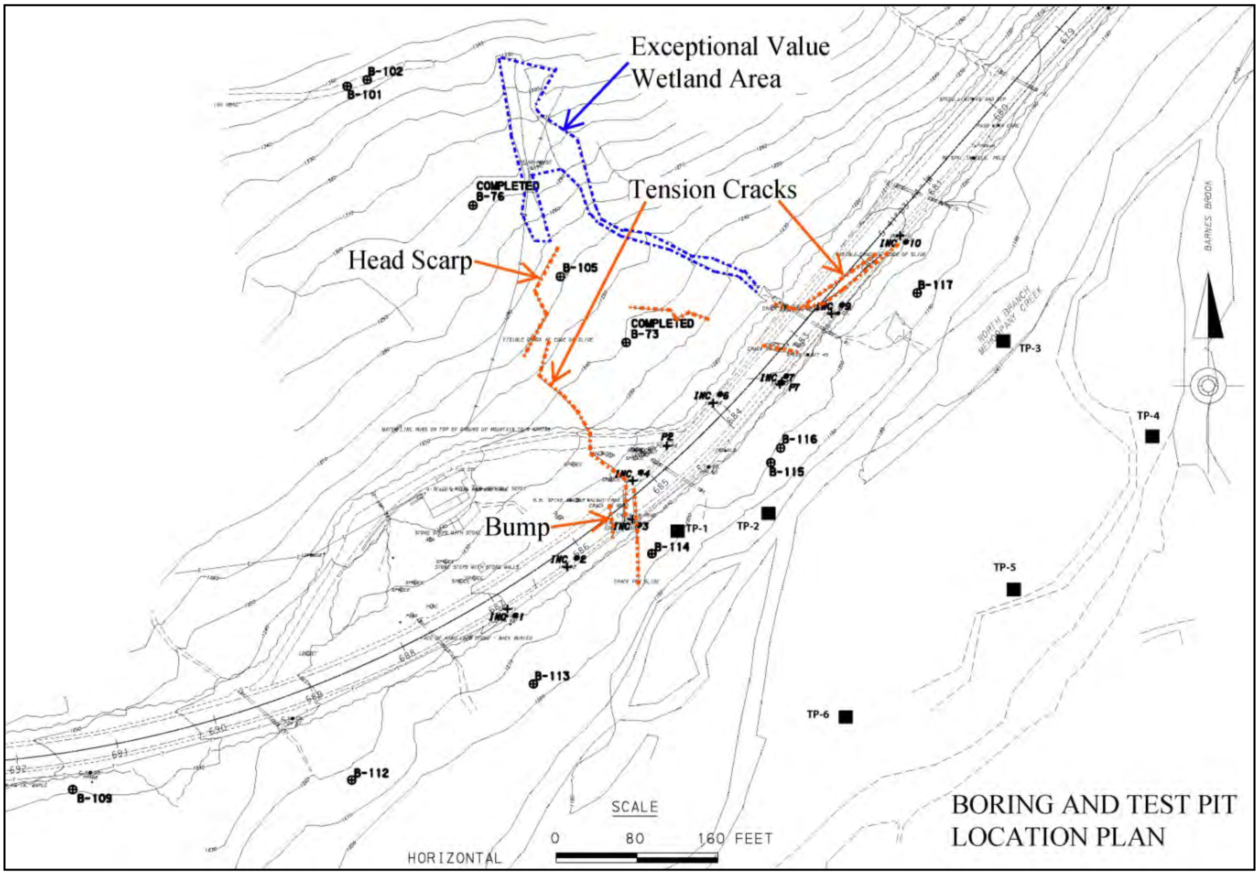


Figure 5: Boring and Test Pit Location Plan

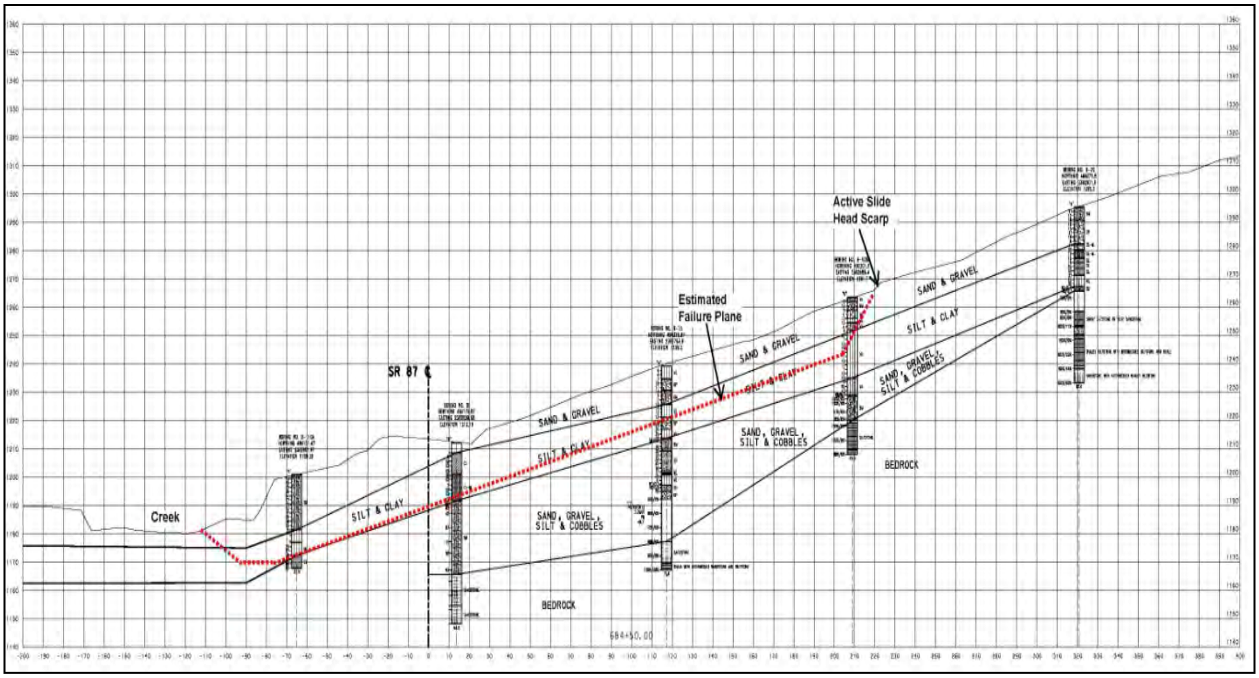


Figure 6: Typical Subsurface Cross Section

The boring logs indicate relatively consistent subsurface conditions throughout the project area that corresponded well with the published soil and geology literature. The overburden soils encountered in the borings consisted of ablation till overlying a glaciolacustrine deposit, glacial till and bedrock. The ablation till was typically described as sand and gravel with varying amount of silt and clay. The glaciolacustrine deposit was typically described as silt with some occurrences of clay and the underlying glacial till was typically described as sand and gravel with varying amounts of silt and clay. Bedrock was encountered beneath the glacial till and was primarily described as siltstone and sandstone which is typical of bedrock within the Catskill Formation (*2*).

Based on the published literature review, varves were expected to be encountered within the glaciolacustrine deposit during the subsurface investigation (*4*). However, varves were seldom identified within glaciolacustrine deposit in the borings performed on the hillside. When varves were identified in the borings conducted on the hillside, the varves were laminated as shown in Photo 3. No horizontal varves were encountered within the glaciolacustrine deposit on the hillside. The varves observed during test pit operations in the valley floor were oriented horizontal. The horizontal varves encountered in the valley floor are shown in Photo 4.

Photo 3: Laminated Varves Encountered on Hillside

Photo 4: Horizontal Varves Encountered in Valley Floor

LABORATORY TESTING

Soil samples collected during the subsurface exploration were tested in the laboratory. Tests performed included sieve and hydrometer analyses, Atterberg limits, natural moisture content, unit weight, direct shear with residual measurements, and triaxial shear. The existing geotechnical data provided by PENNDOT indicated that the failure plane was located within the glaciolacustrine deposit, therefore, the laboratory testing program concentrated on determining the engineering properties of the glaciolacustrine deposit. A summary of laboratory test results are shown in Table 1.

Boring	Sample	Depth (ft)	Material Description	USCS	% Gravel	% Sand	% Silt	% Clay	LL	PL	PI	% WC	Dry Unit Wt. (pcf)	Direct Shear		Triaxial	
														Peak Strength	Residual Strength	ϕ	c (psf)
B-73	S-10	13.5 - 15.0										34.5	-	-	-	-	-
B-73	S-12	16.5 - 18.0	SILT	ML	0.3	2.6	50.7	46.4	29	26	3	32.4	98.8	-	23.3°	-	-
B-73	S-16 & 17	22.5 - 25.5	SILT	ML	0.7	7.3	48.8	43.2	30	27	3	30.8	88.9	-	17.9°	-	-
B-76	S-17 & 18	24.0 - 27.0	SILT	ML	4.4	5.1	68.4	22.1	25	23	2	27.0	96.7	-	14.4°	-	-
B-105ST	ST-2	24.0 - 26.0	SILT	ML	0.5	4.3	43.9	51.3	33	25	8	31.4	-	26.6°	19.2°	-	-
B-105ST	ST-3	27.0 - 29.0	SILT	ML	0.4	0.2	45.6	53.8	34	29	5	34.1	-	23.6°	16.4°	-	-
B-105B	ST-2 & 3	24.0 - 28.0	SILT	ML	0.0	1.0	44.0	55.0	31	26	5	30.6	-	-	-	15°	800
B-113ST	ST-1	12.0 - 14.0	SILT	ML	0.0	3.0	40.6	56.4	31	24	7	30.8	96.6	14.8°	11.8°	-	-
B-113ST	ST-2	18.0 - 20.0	SILT	ML	0.0	0.0	33.6	66.4	35	25	10	32.8	95.4	22.9°	17.3°		
B-114ST	ST-1	12.0 - 14.0	SILT	ML	0.0	2.4	54.4	43.2	30	25	5	31.0	101.8	14.8°	13.2°	-	-
B-114B	ST-1	11.0 - 13.0	SILT	ML	0.0	0.1	59.7	40.2	30	26	4	30.6	-	-	-	15°	800
B-114B	ST-2	13.0 - 15.0	SILT	ML	0.0	0.5	76.2	23.3	27	26	1	33.5	-	-	-	15°	800
B-116A	ST-1	19.5 - 21.5	SILT	ML	1.5	2.0	43.7	52.8	31	23	8	30.5	83.6	22.1°	13.4°	-	-
B-116A	ST-1	19.5 - 21.5	SILT	ML	1.5	2.0	43.7	52.8	31	23	8	30.5	83.6	13.9°	10.7°	-	-
B-116A	ST-3	23.5 - 24.0	Lean CLAY	CL	2.1	8.7	42.6	46.6	32	23	9	27.1	-	25.3°	17.8°	-	-
Streambed	1		Lean CLAY	CL	0.0	0.1	30.1	69.8	36	23	13	38.0	-	-	-	-	-
TP-5	Bag 1		Lean CLAY	CL	0.0	0.1	24.2	75.7	35	23	13	33.7	-	16.0°	8.3°	-	-

Table 1: Summary of Laboratory Test Results

The glaciolacustrine material generally classified at silt (ML), but some of the samples classified as lean clay (CL). The natural moisture content of the glaciolacustrine material ranged from 27.0 to 38.0 percent, liquid limits ranged from 25 to 36 percent and dry unit weights ranged from 83.6 pcf to 101.8 pcf.

Twelve direct shear tests with residual shear measurements were performed to estimate the residual shear strength of the glaciolacustrine deposit. The peak shear strength of the glaciolacustrine material was measured in nine of the direct shear tests. Based on the classification results of the glaciolacustrine material, each of the samples tested were similar in composition. However, the results varied significantly as the peak shear strength of the material ranged from 13.9 degrees to 26.6 degrees and the residual shear strength of the material ranged from 8.3 degrees to 23.3 degrees. There was no distinct correlation between the depth of sample and residual shear strength of the material. Plots of residual shear strengths test results are shown in Figure 7.

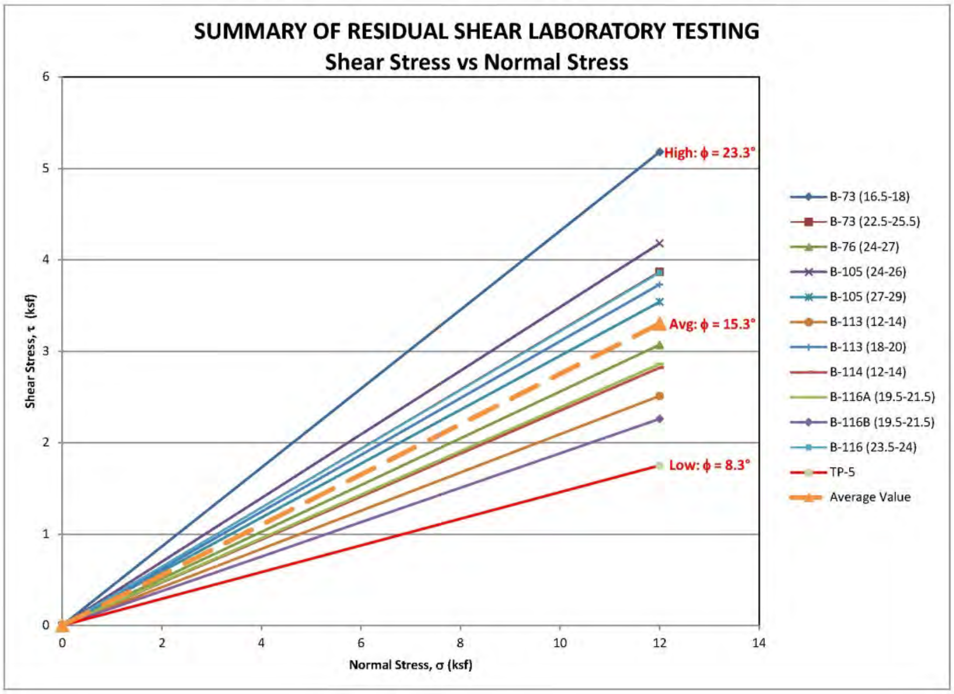


Figure 7: Summary of Residual Strength Laboratory Test Results

One consolidated-undrained triaxial shear test with pore pressure measurements was performed on the glaciolacustrine material. The test result indicated the effective shear strength of the glaciolacustrine material is 26 degrees (c = 0 ksf).

One consolidation test was performed on the glaciolacustrine material encountered in the valley floor. The test result indicated the pre-consolidation pressure of the material is 3.07 tons per square foot.

INSTRUMENTATION

Inclinometer casing was installed in 18 borings. Ten of the inclinometers are located within the active landslide and eight are located outside of the active landslide. Inclinometer readings were obtained until excessive movement of the casing prevented the probe from being lowered to the bottom of casing or the instrument was destroyed during construction. A typical inclinometer plot is shown in Figure 8.

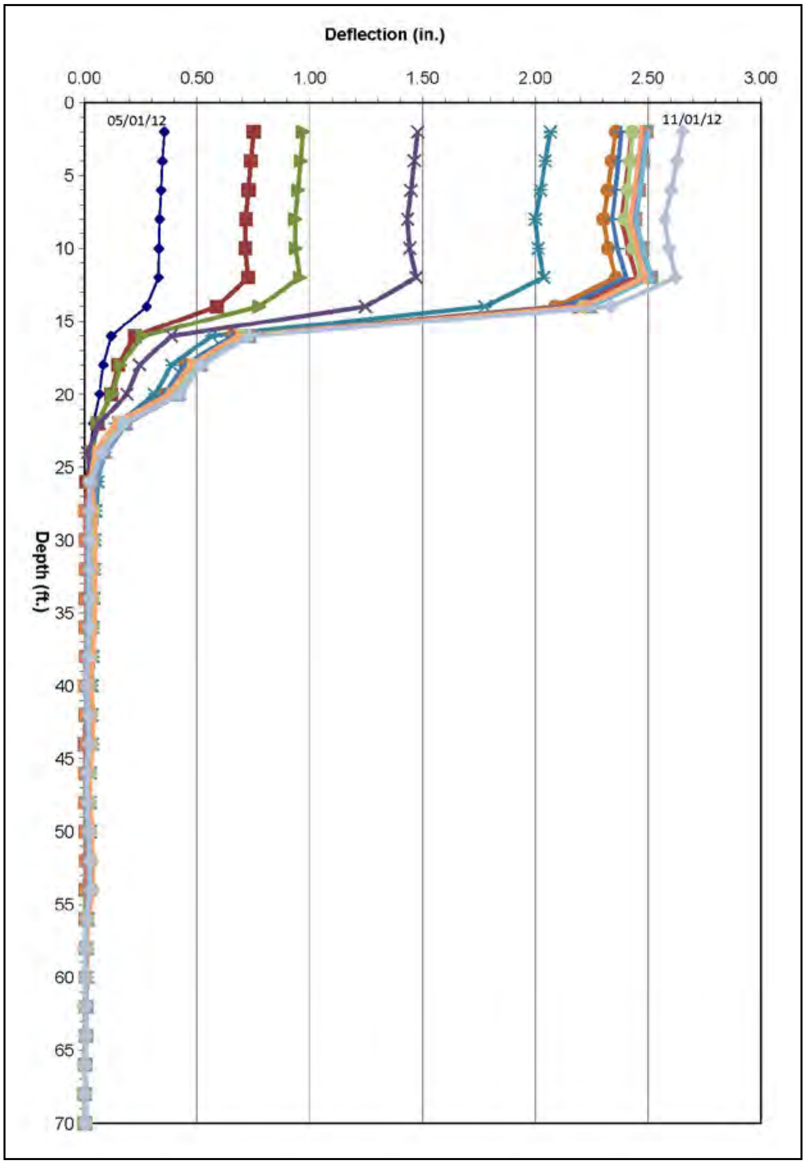


Figure 8: Inclinometer B-73 Located Within Active Landslide

Standpipe (Casagrande) piezometers were constructed in nine (9) borings located throughout the landslide area. Automated transducers were installed in some of the piezometers to monitor long term groundwater levels. Based on piezometer readings, the depth to groundwater varied from approximately 5 to 15 feet below ground surface at the toe of the active landslide and from approximately 15 to over 30 feet below ground surface at the upper portion of the active landslide.

LANDSLIDE TRIGGER MECHANISMS

Several factors are believed to have contributed to triggering the active landslide. Since the majority of the landslide failure plane is located within the glaciolacustrine deposit, the glaciolacustrine deposit is considered the main cause of the landslide. Based on laboratory test results, the glaciolacustrine material exhibits low shear strength values and the natural moisture content of the material is at or above the liquid limit of the material, indicating the material behaves more like a viscous fluid than soil.

Other factors that contributed to the landslide include the previous slope movement that occurred at the site as indicated by field observations and the LIDAR with hillshade data (*5*), and the published literature also indicated that the project area is susceptible to landslides based on the geologic and topographic conditions present at the site (*3*). Elevated water levels within the slope, as evidenced by seeps and wet areas at the ground surface within the active landslide area, were also likely contributors to slope instability. Finally, when a portion of the hillside toe was eroded during the flood events in 2011, and the likely inflated piezometric levels within the hillside due to the excessive rain leading to the flooding, conditions at the project site were ideal for the glaciolacustrine material to slide downslope.

SLOPE STABILITY ANALYSES

The computer program GSTABL7 (*6*) was used to perform the slope stability analyses of the slope before and after construction of the remediation. A back analysis was performed to assess the parameters developed for the stability analyses and to validate the stability model, which would eventually be utilized to design a remediation. A cross-section near the center of the active landslide (Sta. 684+50) was used for these analyses. Inclinometer data and visual landslide features (head scarp) were used to model the location of the landslide failure plane. The piezometer data was used to model the groundwater level in the stability model. Borings and laboratory test results were used to estimate soil strata properties.

The shear strength of the glaciolacustrine deposit was the least defined parameter in the stability model based on the variability of the laboratory shear strength test results and the variation of material type and varve orientations. The glaciolacustrine material encountered in the valley floor differs from the glaciolacustrine material encountered in the failure surface on the hillside. The glaciolacustrine material encountered in the valley floor laboratory classified as clay, while the material on the hillside classified as silt. Additionally, it was believed that based on the horizontal orientation of the varves observed in the valley floor, the material in the valley floor has not experienced past movement. Therefore, the peak shear strength of the material determined in the laboratory was assigned to the valley floor clay (Φ=16°). The residual shear strength of the glaciolacustrine material on the hillside was used in the model since the landslide was active and significant movement had occurred within the glaciolacustrine material located on the hillside.

The results of the back analyses indicated that the existing slope has a factor of safety of approximately 0.8 when the average residual shear strength of the glaciolacustrine material on the hillside was utilized in the analyses (Φ=15°). The result was believed to be unrealistic because the existing condition factor of safety was expected to be approximately 1.0 since the slope was at equilibrium prior to the slope movement identified in 2011. Since the majority of the failure plane occurs within the glaciolacustrine deposit, it was determined that the initial shear strength parameters of the glaciolacustrine deposit were underestimated in the stability analyses.

The CU triaxial shear laboratory test result conducted on the glaciolacustrine material encountered on the hillside and pocket penetrometer readings conducted on the glaciolacustrine material encountered in the valley floor both indicated that the glaciolacustrine deposit exhibited cohesion of approximately 1,000 pounds per square foot (psf). Therefore, a marginal amount of cohesion (100 psf) was included for the glaciolacustrine material in the stability model. This stability model resulted in a factor of safety of slightly less than 1.0, and had failure plane entry and exit points in the general area of the head scarp and toe observed in the field. Based on these results, the stability model appeared reasonable. A plot of this analysis from GSTABL7 (*6*) is shown in Figure 9.

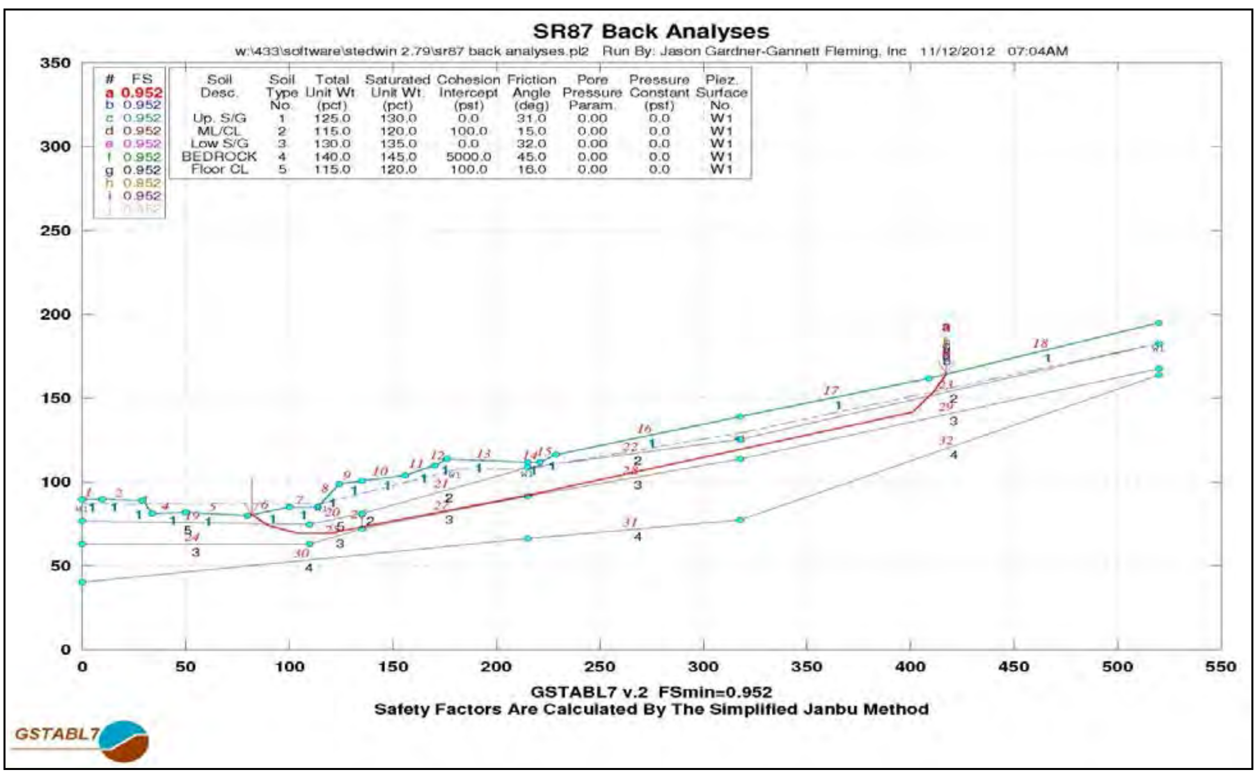


Figure 9: GSTABL7 Back Analyses Model

LANDSLIDE REMEDIATION ALTERNATIVES

Landslide remediation alternatives considered included: no-build, structural elements, and upslope drainage control with earth berm and stream relocation. The no-build option was considered but not recommended for this project because this alternative would not remediate the landslide and the landslide would continue to creep during periods of wet weather and as additional material was eroded at the toe of the hillside during flood events. Furthermore, periodic maintenance would be required to maintain traffic on SR 87. Structural elements, including driven and drilled-in piles were considered, but based on the size of the landslide and anticipated number of structural elements required, the alternative was cost prohibitive.

The upslope drainage control with earth berm and stream relocation alternative was considered the best remediation alternative for this project because the remediation required no future maintenance and does not have a design life. Relocating the stream is required with the use of an earth berm because the North Branch of the Mehoopany Creek is located at the toe of the hillside. Lowering the water table through the use of a series of trench drains upslope of SR 87 would assist in remediating the landslide. However, the wet area upslope of SR 87 that contributed to the slope instability was determined to be exceptional value wetlands per the Pennsylvania Department of Environmental Protection (PADEP) and PADEP directed that the solution to remediate the landslide must preserve the exceptional value wetlands. Therefore, the upslope drainage component of the remediation alternative was eliminated and the earth berm and stream relocation was the recommended remediation alternative for the project.

LANDSLIDE REMEDIATION DESIGN

The computer program GSTABL7 (*6*) was used to design the earth berm remediation. The subsurface model developed during the back analyses was used for the remediation design, including residual strengths of the glaciolacustrine material on the hillside. The soil parameters utilized in the landslide remediation design are provided in Table 2.

Soil Description	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion (psf)	Friction Angle
Upper Till	125	130	0	31°
Glaciolacustrine - Hillside	115	120	100	15°
Glaciolacustrine - Valley Floor	115	120	100	16°
Lower Till	130	135	0	32°
Bedrock	140	145	5000	45°
Earth Berm	125	130	0	35°

Table 2: Summary of Soil Parameters Used in Remediation Design

Earth berm configurations with varying berm widths and heights were analyzed in GSTABL7 (*6*) to determine the preferred berm configuration. The analyses indicated that a stepped berm configuration, starting at 170-feet left of the SR87 centerline and extending to Elevation 1200 and then at 120-feet left of the SR87 centerline the berm extends to Elevation 1210, provided a reasonable factor of safety of 1.3. In addition, the stepped berm provides flexibility to place additional material to stabilize the slope if future slope movement is observed at the site. A typical section of the earth berm configuration and relocated creek is shown in Figure 10.

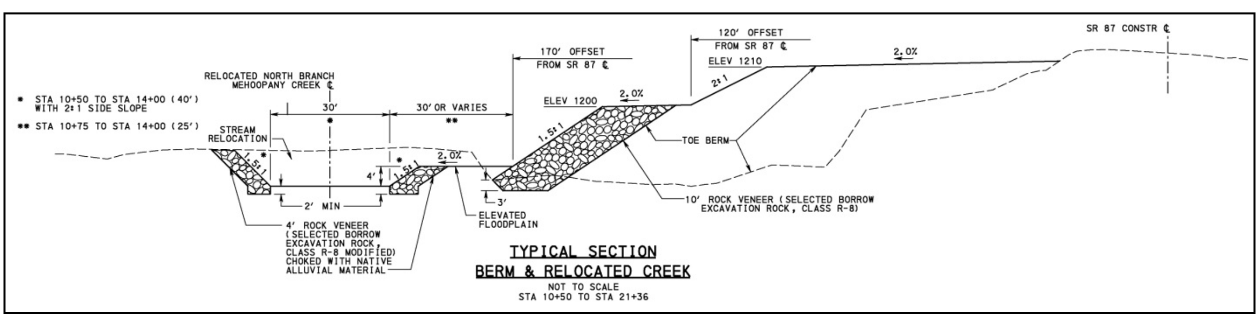


Figure 10: Typical Section of Remediation

To protect the earth berm from erosion during future flood events, a 10-foot thick rock veneer constructed of R-8 rip rap was recommended (*7, 8*). In addition, the relocated streambanks were protected with R-8 rock veneer. The remediation included reconstruction of the SR 87 roadway through the landslide area. To mitigate the potential for reflective cracking in the reconstructed roadway at the locations of tension cracks, the subgrade was reinforced by excavating 2 feet below the pavement subgrade and replacing with No. 2A coarse aggregate with four layers of biaxial geogrid spaced at eight inches.

LANDSLIDE REMEDIATION CONSTRUCTION

Glen O. Hawbaker submitted the low bid of approximately $1.8 million and was awarded the landslide remediation contract in fall 2012. Construction of the remediation was complete in March 2013. Photos 5, 6, 7 and 8 were taken during landslide remediation construction.

Photos 5 and 6: Earth Berm and Roadway Construction

Photos 7 and 8: Earth Berm and Stream Relocation Construction

Inclinometer Readings

Inclinometer readings since the completion of construction were not available from the majority of the in-service inclinometers at the project site at the time this paper was written. No additional inclinometers were installed during or after construction. The inclinometers will continue to be monitored throughout the summer of 2013 and the data will be provided at the 64th HGS in September 2013. Inclinometer B-113, located near the hillside toe, within the earth berm, was extended during construction and readings were available since the end of construction. The inclinometer shows minimal movement has occurred within the slide area since construction was completed. The inclinometer plot is shown in Figure 11.

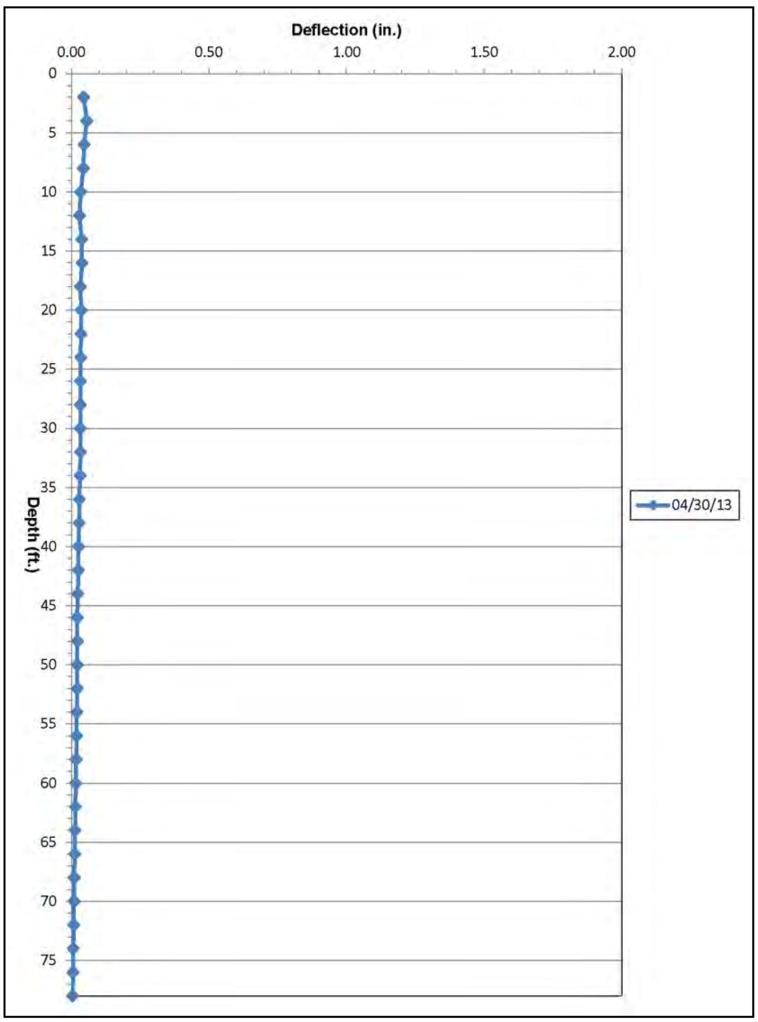


Figure 11: Plot of Inclinometer B-113 Post Construction (April 2013)

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Surface kinematics of the Ferguson rock slide revealed by repeat lidar and GPS measurements, Highway 140, California

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Disclaimer

ABSTRACT

High-resolution topographic data, such as that collected using lidar ("light detection and ranging"), allow examination of the complex morphology of landslide masses. When these data are collected repeatedly over temporally significant time intervals (i.e., days to years), the kinematics of slide motion can be extracted. This information can guide assessments of expected future deformation, and in turn assist hazard and risk assessments as well as steer the design of potential mitigation options. Here, we examine the motion of a large (approximately 800,000 m^3) rock block slide reactivation located in northern California. The Ferguson rock slide moved during the particularly wet spring of 2006 in an area of prehistoric instability as evidenced by multiple headscarps in the upper portion of the slope. The landslide is located on one side of the narrow Merced River canyon where both the river, nationally designated as Wild and Scenic, and California State Highway 140 share the canyon bottom. The 2006 reactivation caused a 3-month closure to this section of the highway, which receives about 875,000 vehicle trips per year and serves as the main all-weather entrance to the iconic and heavily visited Yosemite National Park. As of summer 2013, talus from the landslide still blocked the original roadway and traffic used a one-lane temporary road to detour around the closure.

We present surface and cross-section deformation analyses of the landslide surface using a total of four high-resolution terrestrial lidar data sets collected at approximately two-year intervals following the landslide reactivation. We couple these data sets with differential GPS data collected semi-continuously at three locations on the landslide surface during approximately this same time interval (late-2006 to late 2012) to examine patterns of motion within the slide. Our results provide a more complete understanding of the complex interactions between the upper, driving part of the landslide and the conveyor belt pathway that creates and deposits talus on the original roadway and into the river. Overall, we find that rock slide motion is mostly translational, and it moves at higher velocity in its middle and lower areas compared to the upper blocks. However, we also find that overall velocities have decreased over the 6-year period of investigation. This case study illustrates the use of repeat high-resolution topography for guiding hazard assessments related to ongoing motion of large landslides.

INTRODUCTION

The motion of large landslides is often complex and governed by a multitude of factors including topography, lithology, structural geology, and subsurface and surface hydrology. Geotechnical investigations aimed at discerning driving factors and expected future motion typically rely on a combination of surficial mapping and subsurface investigation, which may include the installation of borings and inclinometers to detect potential shear planes. On-site instrumentation can be challenging to install, maintain, and monitor if the landslide mass is steep and access is difficult. Recent advances in remote sensing, including lidar, allow the collection of high-resolution topographic data which can be used to examine the morphology of the landslide surface without needing to access the landslide surface itself. By examining multiple topographic data sets collected over temporally significant time intervals (i.e., days to years), landslide kinematics can be extracted using cross sections and tracking surface particle movement. Assessments of past motion can guide analyses of expected future deformation, assist with hazard and risk assessments, and aid in the selection of potential mitigation options.

During the spring of 2006, a large rock slide reactivated on the western slope of the Merced River canyon in Northern California, blocking approximately 200 m of State Highway 140 between the towns of Mariposa and El Portal (Figure 1). The highway is the main all-weather entrance road to heavily visited Yosemite National Park and typically conveys 875,000 vehicle trips per year (*1*). After three months of closure, this road into Yosemite was reopened using two, one-lane bridges constructed to route vehicle traffic to the opposite (east) side of the Merced River canyon. As of mid-2013 these bridges still routed traffic through the canyon using timed, alternating, one-way traffic lights. The landslide is approximately 800,000 m^3 in volume (*2*), extends about 380 m in length along its midsection from the upper scarp to the rock fall toe, and is approximately 180 m in width at its widest extent. It has been classified as a rock-block slide containing numerous internal slumps (and related scarps) with primarily translational movement (*3*). A steep talus slope is located immediately below the area of internal scarps and rock fall debris was being nearly constantly deposited onto this slope during the initial reactivation of the landslide.

Although various alternate routing strategies have been proposed for Highway 140 through this area (*e.g., 4, 5*) as a result of preliminary geotechnical analyses by both federal agencies and private consultants, to date no detailed subsurface investigation has been conducted of the landslide itself. In addition, with the exception of a report investigating the potential landslide runout should the failure mass rapidly mobilize (*6*), the overall kinematics of the landslide remain poorly constrained. Here we use four terrestrial lidar data sets of the entire landslide collected over a 6-year time period, coupled with nearly continuous GPS data at three locations in the upper half of the landslide during this same time frame, to identify the cumulative displacement of various segments of the landslide. We implement simple but robust particle tracking techniques across more than twenty points distributed throughout the landslide surface to determine the positional vectors of motion during three time intervals (2007-2008, 2008-2010, and 2010-2013). In addition to showing how the kinematics of a large active landslide can be determined using repeat terrestrial lidar data, the results provide a record of the landslide deceleration over time and illustrate the complexities of this landslide's motion.

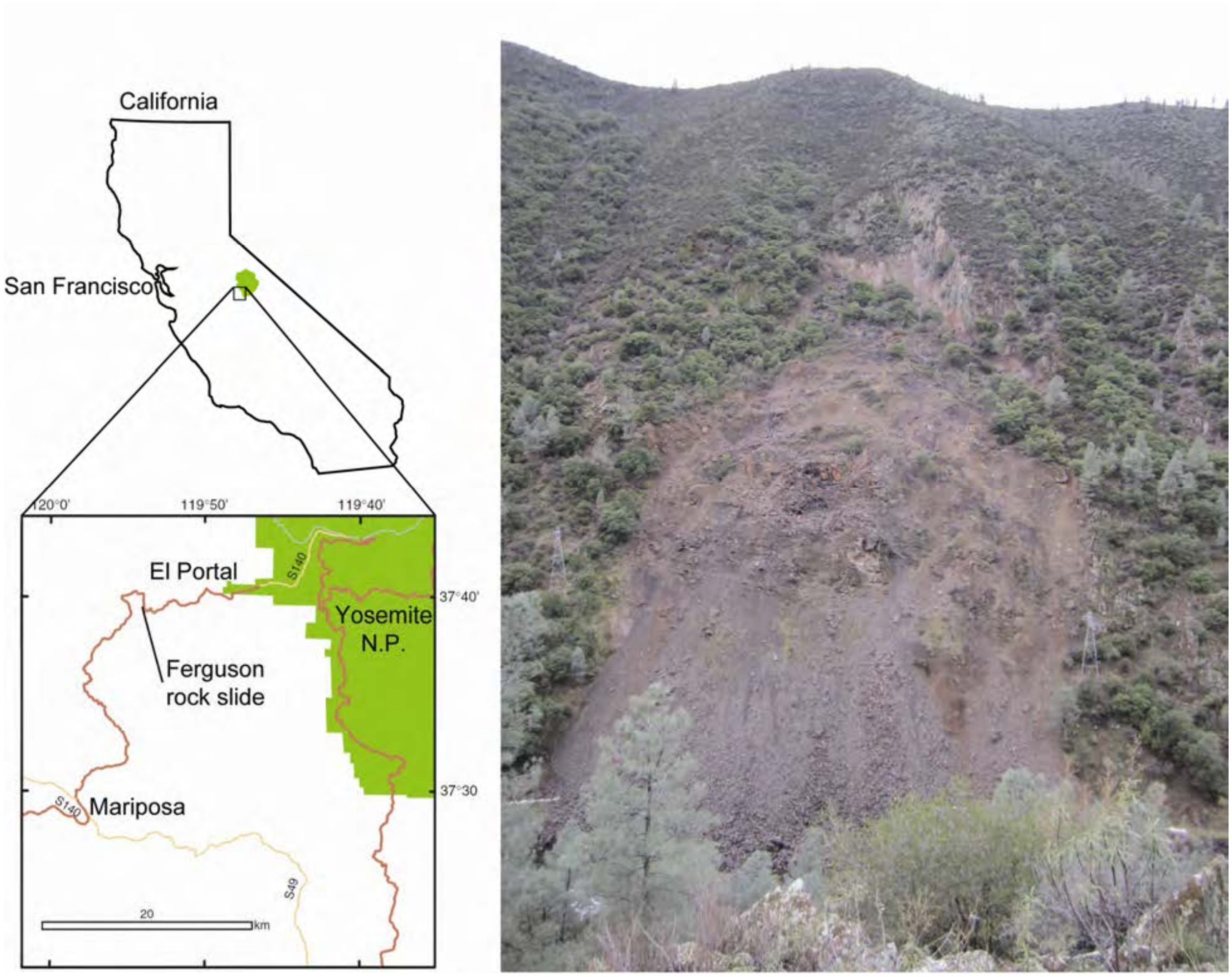

Figure 1 –Regional location map and oblique view of the Ferguson rock slide in northern California. Note transmission towers for scale. Base map modified from *Harp et al.* (*2*)

METHODS

Lidar Data Collection and Processing

We collected three terrestrial-lidar point cloud data sets of the entire landslide mass and talus slope, one each in December 2008, December 2010 and January 2013, using a Riegl Z420i laser scanner. Each data set was created using scans from four different laser positions located on the canyon floor and slope opposite the landslide (Figure 2). We obtained a fourth raw point cloud data set from Optech Inc., who had performed preliminary scanning of the site from a single scan position in February 2007. For each of the 2008, 2010, and 2013 data sets, we registered the point clouds of the four scan positions using a best fit of seven, 10 cm by 10 cm cylindrical reflective control point targets mounted on 2.5 m tall prism poles distributed throughout the more accessible lower third of the landslide (Figure 2 – inset). Typical three-dimensional root mean square (RMS) registration errors for each data set averaged 4 cm.

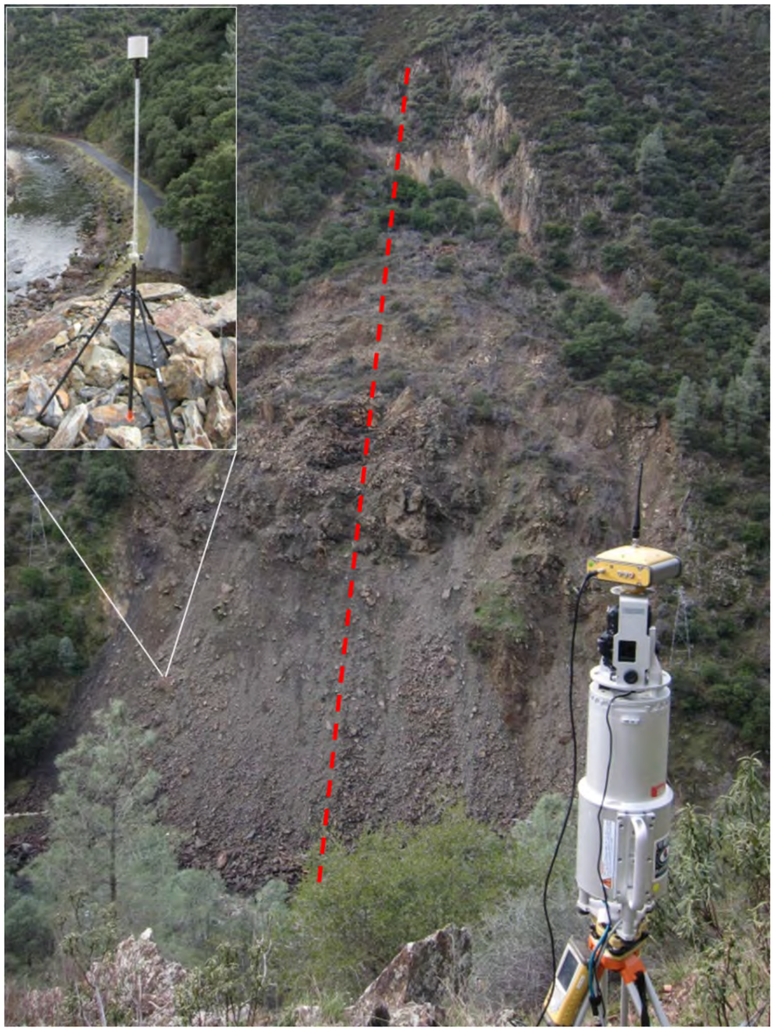

Figure 2 – Terrestrial lidar data collection from one of four scan positions opposite the landslide. Registration procedures used a network of seven reflective targets located on the landslide; one control point target is shown in the photo inset. Georeferencing was obtained via RTK-GPS coordinates of the targets and scanner positions. Dashed line indicates cross-section parallel to overall slope geometry through center of slide (see fig. 4).

Each data set was georeferenced to 1983 North American Datum (NAD83), Universal Transverse Mercator (UTM) Zone 11 coordinates relative to the North American Vertical Datum of 1988 (NAVD88) using GPS data collected at the time of scanning on each of seven control point reflective targets and the four lidar scanner locations. We conducted real-time kinematic (RTK) GPS surveys using a pair of Topcon Hyper+ GPS receivers and a base station located well off the landslide boundaries. Best-fit matching of control and laser scanner locations with GPS coordinates resulted in three-dimensional RMS errors averaging 3 cm. To optimize georeferencing for comparative purposes between data sets, we performed additional point to point registration on approximately 1600 common stationary points from electrical transmission towers located on either side of the landslide using automated methods available in our processing software (Maptek I-SiTE 3.5.1). We processed the Optech data from 2007 in a similar manner, but could only use point cloud registration procedures to integrate the various instrument orientations from this data. Resultant registration errors were 5 cm and georeferencing errors using the electrical towers were 6 cm. Combined with a mean laser scanner range and azimuth error of approximately 1 cm (*7, 8*) and treating each error term independently following methods from *Collins et al.* (*9*), we estimate the mean error for each point in the processed data for all time periods to be approximately 6 cm.

Three-dimensional Surface Construction

Three-dimensional (3D) surfaces are a typical product of terrestrial lidar point cloud data. For landslide analysis, they provide a means to visualize the shape and relative position of various components of the landslide ground surface with respect to such items as internal scarps, down-dropped blocks, and stable bedrock. They can also be used to identify and compute displaced volumes between date sets (*10, 11*). For the Ferguson data sets, we constructed 30 cm, bare earth, triangulated irregular network (TIN) surfaces (e.g., Figure 3) of each data set using a suite of lowest point and surface proximity filters available in our processing software. We then generated cross sections and performed color histogram matching of each surface to the preceding surface to identify active versus inactive parts of the landslide over time.

Particle Tracking Analysis

If the ground surface topography does not change shape significantly over time, and if the surface contains easily distinguishable elements such as rocks and other debris with sharp edges, repeat point cloud data can be used to track the trajectory and velocity of a landslide surface. Performed for a suite of points distributed throughout the surface, the data can act as a proxy to determine the shape and boundaries of the active parts of the landslide. We identified 22 points visible in each of the four temporally consecutive lidar data sets for particle tracking analysis at the Ferguson rock slide (Figure 3). We grouped the points into discrete sections (Old Scarp, Upper Block, Middle Block, Lower Block, Prow, and Talus – Upper, Middle, and Lower) of the landslide based on overall visual boundaries provided by scarps and block boundaries. The broken, angular phyllite, slate and chert rock debris (*12*) forming the majority of the landslide surface created points that were easily distinguishable in each data set. However, no points in the Old Scarp area and only one point in the Upper Block area with low point density could be identified due to dense vegetation. We also selected four points on the immobile, electrical transmission towers located outside the landslide boundaries in each data set to act as reference control for error analysis. The average 3D displacement for these four points for each time period is 16 cm. This amount represents an upper bound error estimate for our particle tracking analyses and is acceptable given the inherent difficulties in selecting a single representative point of an object with fairly coarse (6 cm) point spacing at more than 200 m range.

GPS Measurements

In the fall of 2006, we installed three USGS GPS spider units (stand-alone, low-cost, L1 single frequency GPS receivers and batteries mounted to a rigid steel tripod frame) on different blocks of the slide (*3*, *13*). These units were part of a system designed for automated data acquisition, rapid deployment, and prolonged operation in remote hazardous areas; the system has been used at the Mission Peak landslide (*14*) and Mount St. Helens (*15*). At the Ferguson

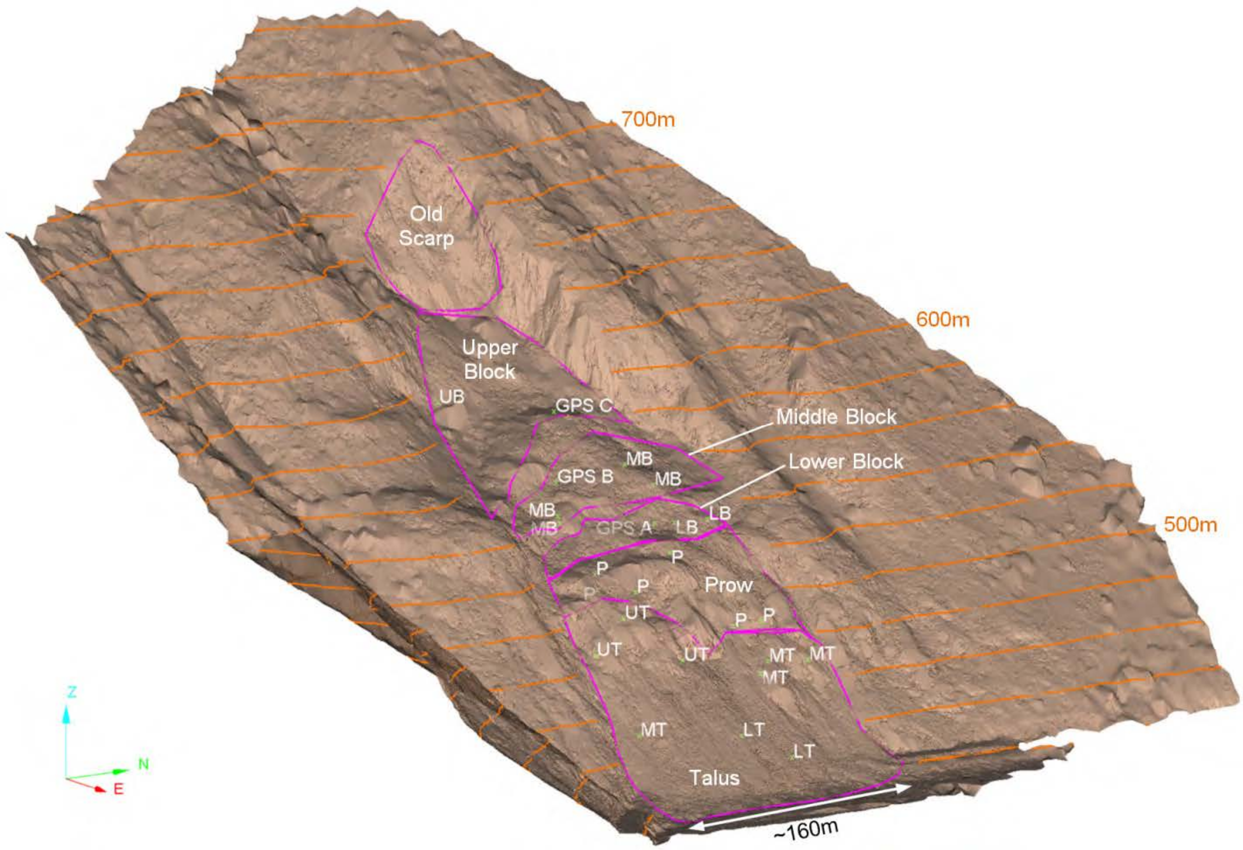

Figure 3 – Three-dimensional surface map from 2013 lidar scans showing generalized areas of the landslide and locations of particle tracking points (UB, MB, LB, P, UT, MT, LT) used to generate the results in Table 1. GPS spider locations (GPS A, GPS B, GPS C) are shown for reference.

rock slide, data from the spider units were transmitted hourly via radio telemetry, and processed in near-real time against GPS observations from a nearby USGS reference station, to obtain sub-centimeter displacements. These data provide a nearly continuous record of slide motion over the 6-year period of monitoring. The GPS spider units were dismantled in October 2012. Thus, the bracketing dates for the point velocities in Table 2 are slightly different than in Table 1.

RESULTS

Surface Deformation

Our lidar particle tracking and GPS results indicate a coherent, eastward (~84°) and downslope component of motion throughout the landslide surface during the 6-year monitoring period (Table 1 and Table 2). This supports previous interpretations (*2, 3*) that landslide motion is primarily translational, as rotational motion would typically be accompanied with at least some convergent motion near the lateral margins. Displacement magnitudes between adjacent sections (e.g., Lower Block, Prow, and Upper Talus; Middle Talus and Lower Talus) indicate that some parts of the landslide likely moved coincident with one another. For example, the Upper Talus could have moved with the Lower Block and Prow, indicating that the slip surface might toe out within the Upper Talus. This hypothesis was first suggested in 2006 by Ed Harp of the USGS (*2*). Both the lidar and GPS results indicate that distinct blocks in the upper reaches of the landslide moved at different rates from one another. In general, the Upper Block and Middle Block areas moved about two to five times more slowly than the Lower Block and Prow areas. The difference in displacement between blocks was likely accommodated by the opening of various tension cracks across the Middle Block and Lower Block areas.

Table 1 – Particle Tracking Results

Location	End of time period[1]	Avg. horiz. disp. (m)	Avg. horiz. azimuth (° from north)	Avg. vert. disp. (m)	Avg. horiz. velocity (m/yr)	Avg. vert. velocity (m/yr)	Avg. 3D disp. (m)	Avg. 3D velocity (m/yr)
Upper Block (UB)	2008	no data						
	2010	0.1	99	-0.1	0.1	0.0	0.2	0.1
	2013	0.3	100	-0.6	0.2	-0.3	0.7	0.3
Middle Block (MB)	2008	0.8	112	-0.5	0.4	-0.3	1.1	0.6
	2010	0.5	92	-0.2	0.3	-0.1	0.8	0.4
	2013	0.2	80	-0.1	0.1	-0.1	0.2	0.1
Lower Block (LB)	2008	2.9	85	-3.6	1.6	-1.9	4.7	2.3
	2010	1.2	92	-1.6	0.6	-0.8	2.0	1.0
	2013	0.7	80	-0.3	0.3	-0.2	0.8	0.4
Prow (P)	2008	2.8	80	-3.7	1.5	-2.0	4.7	2.5
	2010	1.8	75	-1.8	0.9	-0.9	2.6	1.3
	2013	0.8	79	-0.8	0.4	-0.4	1.1	0.5
Upper Talus (UT)	2008	4.1	91	-1.5	2.2	-0.8	4.9	2.6
	2010	1.7	84	-1.7	0.8	-0.9	2.5	1.2
	2013	1.0	80	-0.5	0.5	-0.2	1.1	0.5
Middle Talus (MT)	2008	2.0	87	-1.4	1.1	-0.8	2.4	1.3
	2010	1.2	89	-0.7	0.6	-0.4	1.4	0.7
	2013	0.4	76	-0.3	0.2	-0.1	0.5	0.2
Lower Talus (LT)	2008	1.9	77	-1.3	1.0	-0.7	2.3	1.2
	2010	1.0	81	-0.5	0.5	-0.3	1.2	0.6
	2013	0.4	90	-0.2	0.2	-0.1	0.4	0.2

[1]Time periods are denoted by their ending date: 2008 = 2/1/2007–12/12/2008; 2010 = 12/12/2008-12/13/2010; 2012 = 12/13/2010–1/24/2013.

The particle tracking analyses also indicate that very little new rock fall debris was deposited on the talus slope itself. Overall, we could easily identify similar pieces of talus in each of the lidar time series. If rock fall depositional rates had been high, pieces identifiable in one lidar image would be buried in the next image. This observation indicates that the talus slope is at angle of repose and that future rock fall debris will accrete at the toe of the talus slope, in or near the Merced River. Thus, the river rapids at this location will likely continue to change with future rock falls from the Prow area.

Particle tracking data indicate a clear deceleration pattern following the initial slope movements in 2006. In most parts of the slide, the 3D velocity decreased by a factor of 5 to 6 between 2007-2008 and 2010-2013 (Table 1). Our GPS measurements show similar decreases in velocities over nearly the same time periods (Table 2).

Volumetric Changes

Comparison of cross sections through the lidar-generated surfaces also highlight movement patterns within the landslide (Figure 4). During periods when known surface displacements occurred, only minor cross-section surface deformation can be detected in many areas of the slide; this supports the interpretation that the landslide is undergoing planar translation in one direction (eastward). In a plane translation slide, deformation is one-dimensional with respect to the slip surface and therefore cannot be easily detected by surface cross sections. However, the cross sections do indicate up to 6.3 m of down-drop throughout the Upper Block, Lower Block, and Prow areas (Figure 4). These measurements, along with vertical

Table 2 – GPS Spider Displacement Results

Location	End of time period	Avg. horiz. disp. (m)	Avg. horiz. azimuth (° from north)	Avg. vert. disp. (m)	Avg. horiz. velocity (m/yr)	Avg. vert. velocity (m/yr)	Avg. 3D disp. (m)	Avg. 3D velocity (m/yr)
Upper Block (GPS C)	2008	0.2	87	-0.1	0.1	-0.1	0.3	0.1
	2010	0.2	79	-0.1	0.1	0.0	0.2	0.1
	2012[2]	0.3	73	-0.2	0.2	-0.1	0.4	0.2
Middle Block (GPS B)	2008	0.8	71	-0.4	0.4	-0.2	0.9	0.5
	2010	0.2	70	-0.1	0.1	-0.1	0.3	0.1
	2012	0.1	80	0.0	0.1	0.0	0.1	0.1
Lower Block (GPS A)	2008	3.9	78	-4.6	2.1	-2.5	6.1	3.3
	2010	0.9	110	-1.7	0.5	-0.9	2.0	1.0
	2012	0.7	77	-0.6	0.4	-0.3	0.9	0.5

[1]Time periods are denoted by their ending date: 2008 = 2/1/2007–12/12/2008; 2010 = 12/12/2008-12/13/2010; 2012 = 12/13/2010–10/10/2012.

[2]Time period for GPS C for 2012 is 12/13/2010–9/29/2012.

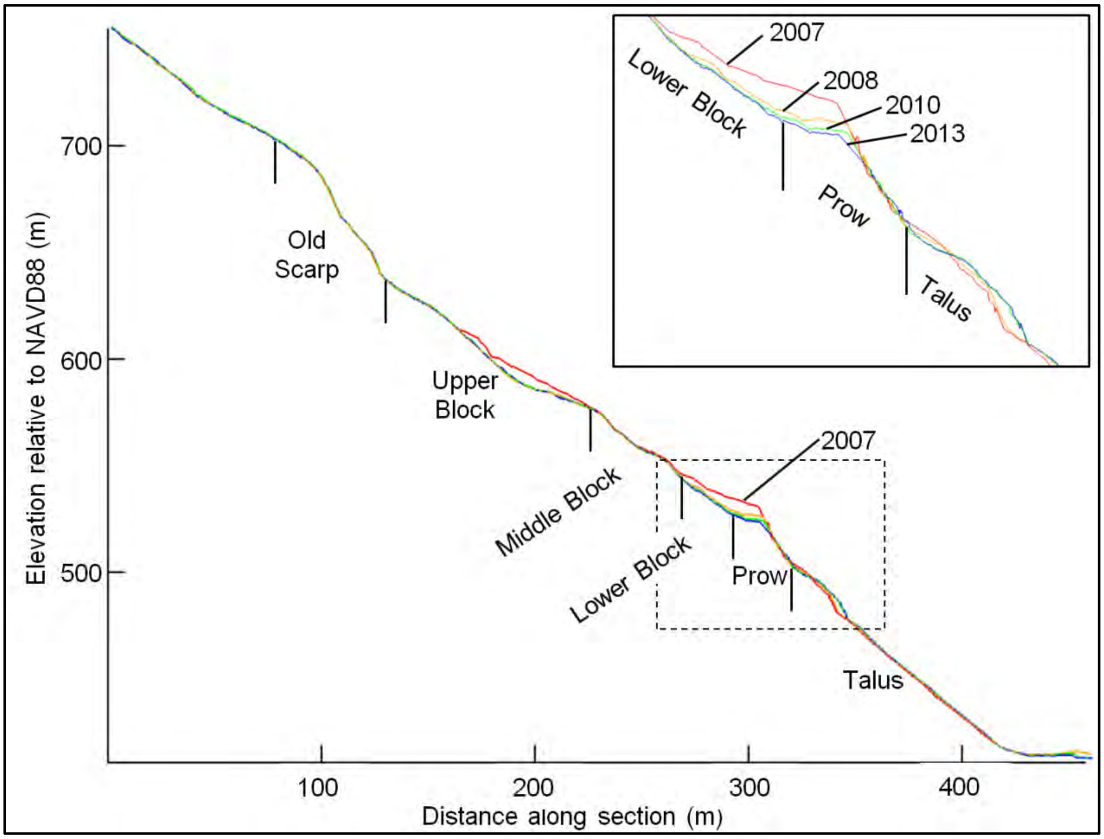

Figure 4 – Changes in cross section over time along the central long-axis (see fig. 2) of the Ferguson rock slide. Inset focuses on the Prow area and identifies each cross section by date. No vertical exaggeration.

displacement vectors calculated from the lidar and GPS observations (Tables 1 and 2), suggest that the central and lower portions of the landslide may be undergoing a somewhat more complex interaction compared with simple translational displacement. In some areas, the downward component of the displacement vectors points out of slope (i.e., with dip vector less than the topographic inclination, as measured from the horizontal), whereas in other areas, the downward component points into the slope; these differences indicate that some rotation may be occurring in the Middle Block and/or Lower Block-Prow areas. Additional analysis is needed to identify the precise motion for each block of the rock slide.

CONCLUSIONS

Through a combination of repeat lidar and nearly continuous GPS data collected over a 6-year period, we identified the overall deformation pattern governing the kinematics of the Ferguson rock slide. Although these results are preliminary in nature due to the limited number of point locations used in particle tracking analyses, they provide useful information with respect to the recent motion of the landslide. The overall deformation pattern is highly suggestive of planar translation to the east, with a potentially more complex interaction occurring immediately above the level of talus. Distinct blocks within the rock slide move at different velocities and overall surface velocities have decreased since initial reactivation in 2006. In this study, we made no attempt to relate movement to potential triggering factors; the movement history of the rock slide should be evaluated by linking deformation patterns to these factors.

In addition to providing site-specific information about this landslide, the methods used herein outline a complementary framework for performing deformation analyses on similar complex landslides. Whereas GPS data can provide a continuous temporal assessment of individual point locations, especially helpful for understanding potential triggering factors, the lidar particle tracking analyses provide a more detailed understanding of the spatial deformation regime of the landslide. Additional development of these and related methods will help with understanding the dynamics and morphology of this and other landslides.

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Remediation of an Active Landslide within a Prehistoric Landslide – SR 2065, Thompson Run Road, Monroeville, Pennsylvania

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ABSTRACT

The primary goal of the SR 2065 Thompson Run Road landslide remediation project was to stabilize the roadway without triggering movement along potentially healed prehistoric landslide failure surfaces. Groundwater trending through weak claystone and thick colluvial slopes above the active landslide presented formidable remediation design challenges. Treatment limitations extended beyond the site geology to include: slope geometry; existing and required right-of-way constraints; railroad right-of-way access restrictions; and the inability to encroach upon, or alter the course of, the Thompson Run stream. Gannett Fleming was tasked by the Pennsylvania Department of Transportation (PennDOT) with providing permissible treatment alternatives and a preferred conceptual remedial design for solicitation of bids from Design/Build contractors. The preferred conceptual design consisted of a caisson supported slope with reconstructed rip-rap embankment. Permissible treatment alternatives included roadway excavation and replacement with stabilized material or a soil nail slope treatment. Adequate roadway stabilization, site geology, right-of-way concerns, and stream encroachment/course alteration were all addressed by the preferred conceptual design. The caissons were designed to key into the Grafton Sandstone/Shale to provide stability for the active landslide while perforating potential prehistoric landslide failure surfaces. A steepened rip-rap embankment allowed for reconstruction of the roadway embankment slope without encroaching on the stream and provided a material that would resist erosion of the slope toe.

INTRODUCTION

The uniqueness of the SR 2065 Thompson Run Road landslide remediation project stems from the combination of natural and man-made design restrictions. Natural design limitations include the project site geology, prehistoric landslide limits, and the groundwater trend throughout the project site. The man-made design restrictions include the existing and required right-of-way, railroad right-of-way access restrictions, the embankment slope geometry, and the inability to encroach on, or alter the course of, the Thompson Run stream.

Project Location

The SR 2065 Thompson Run Road landslide remediation project is located in the Municipality of Monroeville, Allegheny County, Pennsylvania (see Figure 1). The Municipality of Monroeville is located approximately 12 miles east of the city of Pittsburgh and lies within one of three counties (Allegheny, Beaver, and Lawrence) that comprise PennDOT District 11-0. The Thompson Run stream parallels Thompson Run Road north to southwest through the project limits. The currently active landslide is impacting the left offset roadway embankment which lies on the western side of Thompson Run Road. Prehistoric landslide limits are mapped within the right offset slopes to the east of Thompson Run Road.

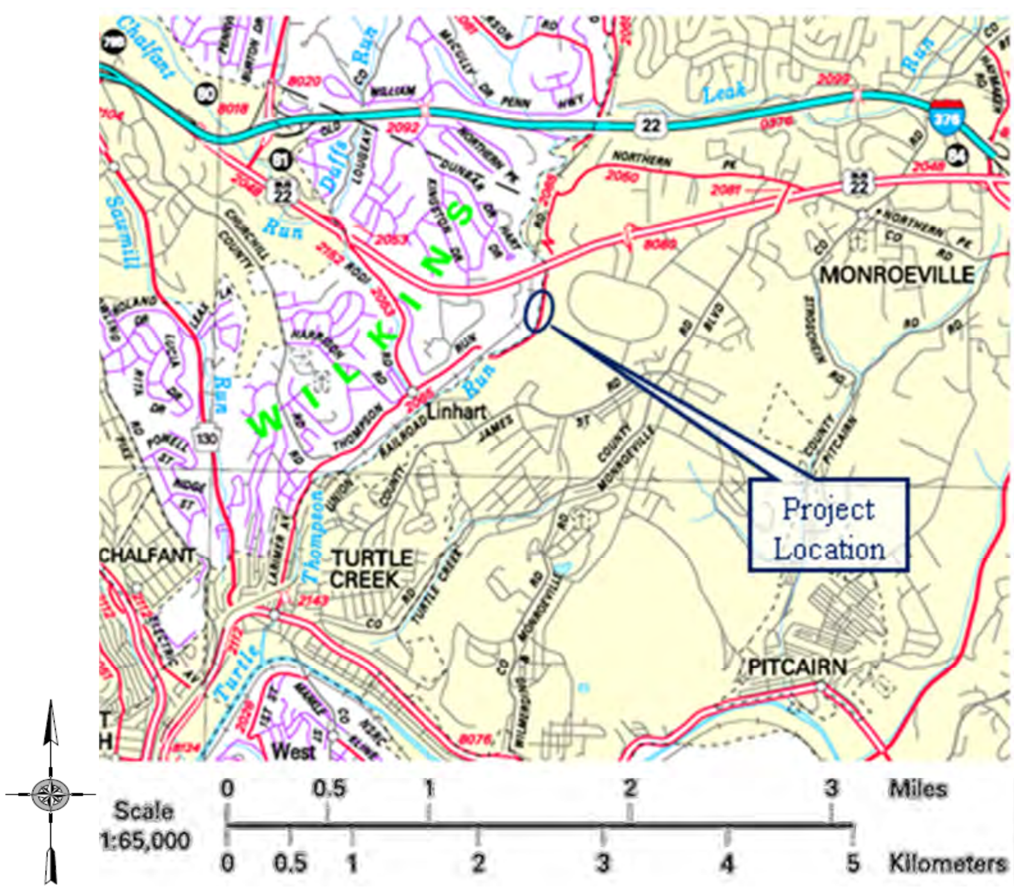

Figure 1 – Project Location Map (*1*)

Project Statement

Gannett Fleming was tasked by PennDOT with providing conceptual and preliminary geotechnical design services for the Thompson Run Road landslide remediation project. Gannett Fleming's scope of work included the determination of permissible treatment alternatives and selection of a preferred conceptual remedial design for solicitation of bids from Design/Build contractors. Conceptual design geotechnical engineering activities for the landslide remediation project began in July 2011. Completion of the Conceptual Design allowed for performance of Final Design geotechnical activities by the Design/Build team to begin in September 2012.

PROJECT SITE DATA COMPILATION

Office Research

Published resources consulted during office research of the project site included, but were not limited to: topographic mapping, soil survey mapping, outcrop/contact and structure contour mapping, Allegheny County stratigraphy, landslide susceptibility mapping, wetlands mapping, flood mapping, and historic aerial photographs.

Soil survey maps of the project area indicated two primary soil types present beneath the site (*2, 3*). The primary soil classifications include the Gilpin-Upshur (GQF) complex and Urban Land (UB). GQF soils are typically characterized to have slopes ranging from 25% to 70%. Gilpin-Upshur soils are characteristically well-drained soils with slow permeability and present a severe erosion hazard. Soils classified as UB typically have slopes ranging from 0% to 3% and are inherently variable in composition; therefore, the drainage characteristics, permeability, and erosion hazard associated with Urban Land are all unspecified.

Landslide susceptibility mapping of the project area indicates that the currently active landslide lies within the limits of a prehistoric landslide (*4*). The right offset slopes adjacent to SR 2065 outside the prehistoric landslide limits are classified as moderately to severely susceptible to landsliding (*4, 5*). The area classified as "landslide susceptible" encompasses the uphill slopes adjacent to, and south of, the project limits.

The preliminary site geology was determined based on structure contour mapping of the Pittsburgh Coal seam and contours of the Ames Limestone. The roadway elevation of Thompson Run Road throughout the project site is approximately EL 830. Structure contour mapping indicates that the crop line of the Pittsburgh Coal is at approximate EL 1070 within the project limits (*6*). The Pittsburgh Coal crop line elevation is 240 feet above the SR 2065 roadway elevation. Geologic mapping of the Greater Pittsburgh region indicates that the Ames Limestone crop line is at approximate EL 780 (*7*). Therefore, the Ames Limestone was anticipated to be present at 50 feet below the roadway elevation. The Generalized Geologic Section of Allegheny County (see Figure 2) shows that the project site bedrock was projected to lie within the Grafton Member of the Conemaugh Formation (*8*). The test borings drilled during the Conceptual Design subsurface investigation were used to verify the geologic findings of the office research.

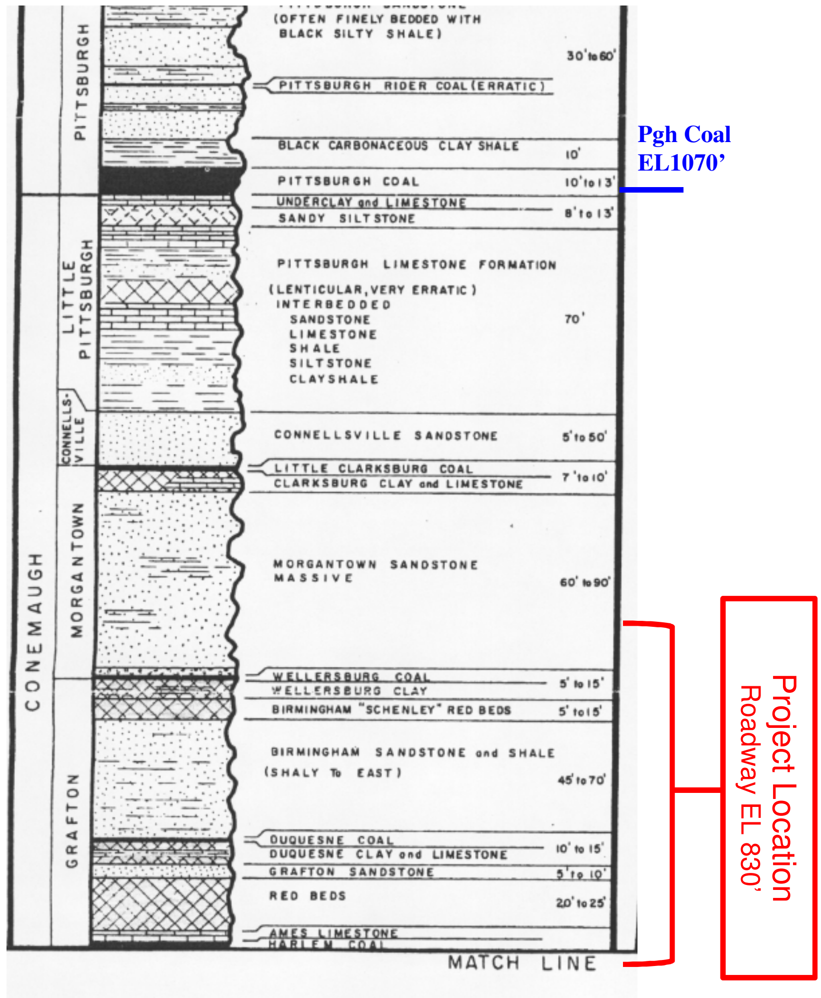

Figure 2 – Generalized Geologic Section of Allegheny County Showing the Project Site Bedrock (*8*)

Field Reconnaissance

Field reconnaissance visits were performed by Ackenheil Engineers and Gannett Fleming geotechnical personnel on November 2, 2011 and January 31, 2012, respectively. These site visits provided pre- and post-Conceptual Design subsurface investigation site observations. Data collected during the field visits was used to design the Conceptual Design subsurface investigation, to verify information gathered during drilling, and to provide additional insight for the Preliminary Design alternatives analysis. The following observations were collected during field reconnaissance.

The course of the Thompson Run stream has been altered due to movement of the active landslide mass. As shown in Figure 3, the left offset roadway embankment bulges into the stream. The stream flow continuously erodes the landslide toe in an effort to re-establish the normal course of the stream. Erosion of the landslide toe perpetuates sliding of the left offset embankment into the stream.

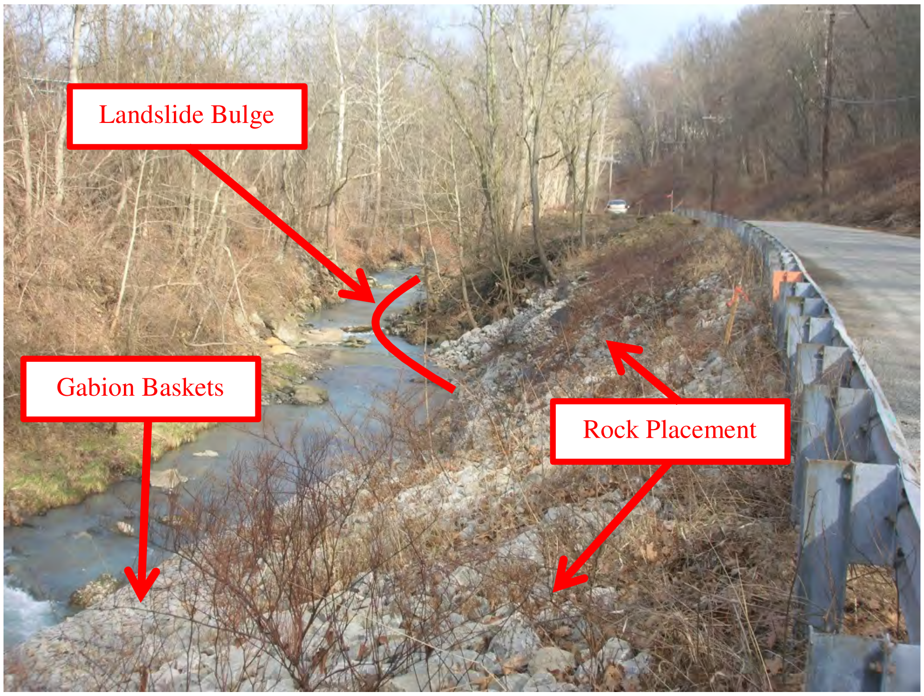

Figure 3 – Bulge of Landslide into Thompson Run Stream (January 2012)

Cracking within the SR 2065 roadway indicates creep of the active landslide. Figure 4 shows the longitudinal and transverse cracking present in the SR 2065 roadway adjacent to the landslide bulge depicted in Figure 3. The damage observed within the roadway during field reconnaissance consisted of the aforementioned cracking as well as slight horizontal displacement of areas of pavement. Damage to the roadway has not led to unsafe driving conditions; therefore, both lanes of the roadway remain open to traffic. No signs of active landslide movement were observed in the right offset slopes of SR 2065. Additionally, no indicators of recent movement of the previously discussed prehistoric landslide were noted during field reconnaissance. Shale outcrops were observed in the right offset slopes north and south of the project limits; however, the shale outcrops were noticeably absent within the areas of active landslide movement.

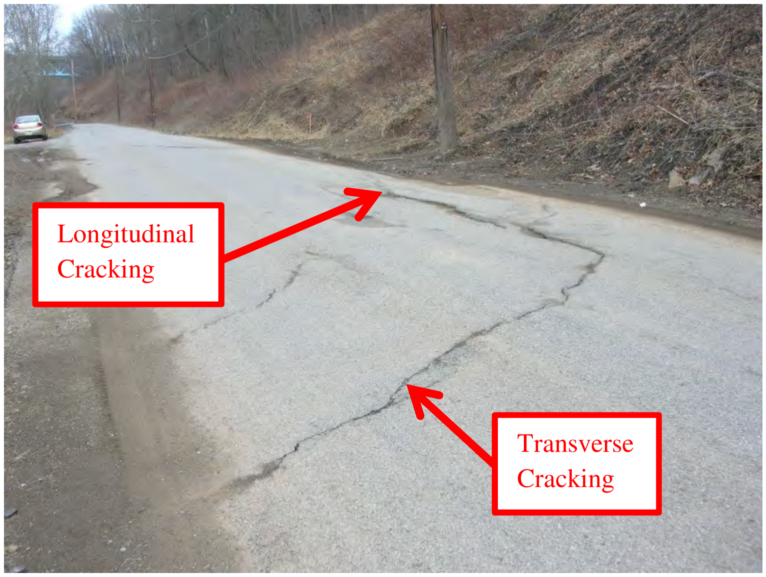

Figure 4 – Longitudinal and Transverse Cracking within the Roadway (January 2012)

Figure 3 shows several of the past remediation efforts within the active landslide area. Past treatments include rock placement on the left offset roadway embankment and gabion baskets supporting a portion of the left offset slope at the southern end of the project limits. Additionally, field conditions indicate several attempts to stop roadway cracking and movement through re-grading and re-paving efforts.

Conceptual Design Subsurface Investigation

Conceptual Design activities at the project site included advancement of two test borings to create a subsurface cross section through the project limits. The Conceptual Design test borings, identified as SR 2065-1 and SR 2065-2, were advanced by Pennsylvania Drilling Company in November 2011 with full-time drilling inspection provided by a PennDOT-certified drilling inspector employed by Ackenheil Engineers, Inc. Boring SR 2065-1 was advanced in the southbound lane of SR 2065 to investigate the subsurface conditions beneath the roadway and within the active landslide limits. Boring SR 2065-2 was advanced in the right offset slopes above SR 2065 to provide confirmation of the project site geology as well as to examine conditions present within the limits of the prehistoric landslide. Figure 5 includes the Conceptual Design boring locations as well as the limits of work and features considered in the landslide treatment design.

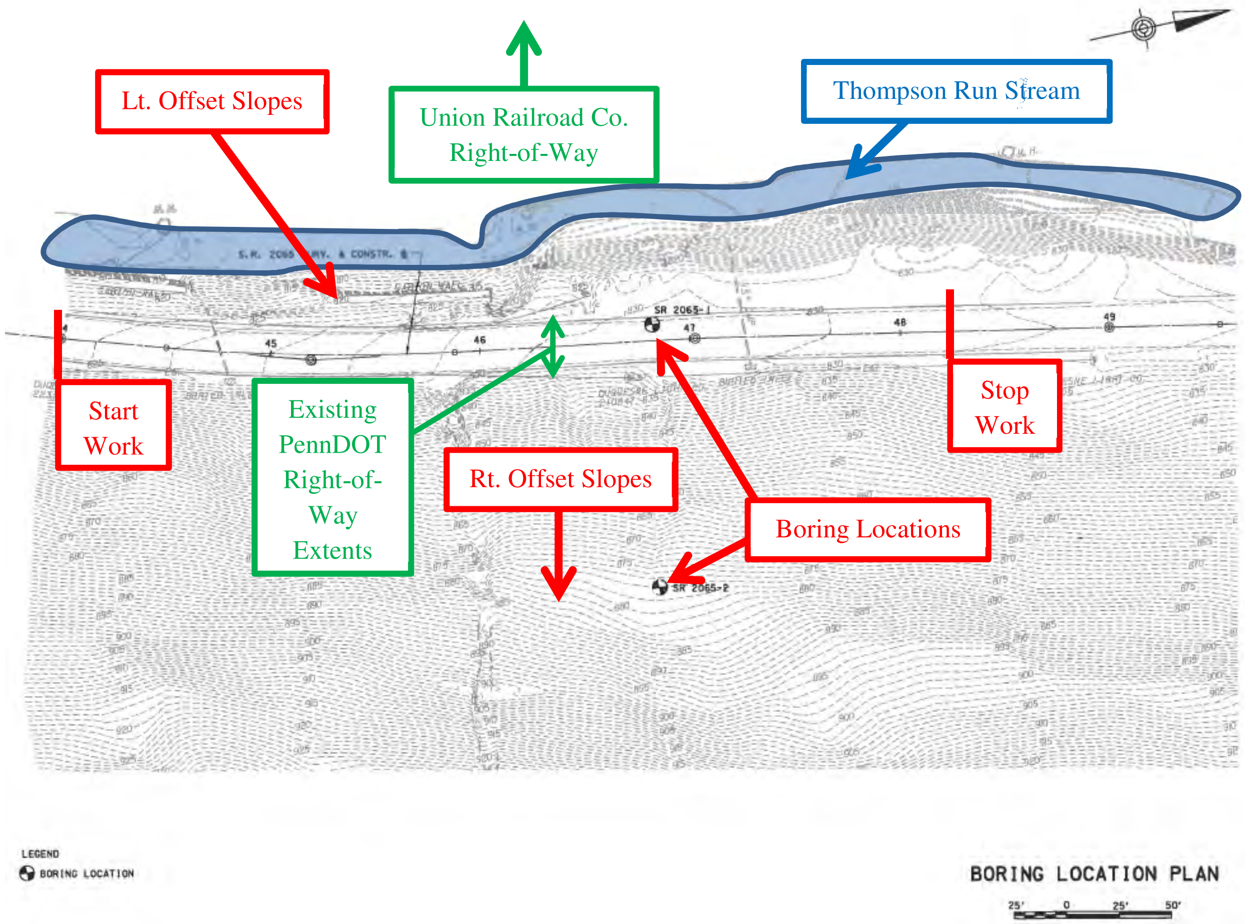

Figure 5 – Boring Location Plan and Project Site Features

Soil Conditions

The soil types encountered in roadway boring SR 2065-1 included fill, colluvium, alluvium, and residual soils. The soil in boring SR 2065-2 consisted solely of colluvium. Soil conditions considered for use in the site subsurface model were drawn primarily from boring SR 2065-1; therefore, Table 1 provides a summary of the soil conditions encountered in SR 2065-1.

Table 1 – SR 2065-1 Soil Conditions

Soil Type	Layer Thickness (ft.)	Composition	Density/Consistency	Average $N1_{60}$ (blows per foot)
Fill	5.0	Sand and Gravel	Medium Dense	29
Colluvium	20.5	Clayey Gravel with Sand	Loose to Medium Dense	20
Alluvium	1.5	Sandy, Silty Clay with Gravel	Stiff	11
Residual	5.7	Gravelly, Silty Clay	Hard	60

A total of 5.5 feet of asphalt was encountered overlying the fill in boring SR 2065-1. As previously discussed, SR 2065-1 was advanced in the southbound lane of SR 2065 within the active landslide area. The thick asphalt suggests that numerous repairs have been made to cracks in the pavement caused by creep. Sampling of the thick pavement section verifies the field observation of the cycle of cracking, re-grading, and re-paving as depicted in Figure 4.

Bedrock Conditions

Boring SR 2065-1 provided bedrock data within the active landslide area while SR 2065-2 allowed for verification of the project site geology. Table 2 summarizes the bedrock conditions encountered in borings SR 2065-1 and SR 2065-2.

Table 2 – SR 2065-1 and SR 2065-2 Bedrock Conditions

Bedrock Type	Unit Thickness (ft.)	Unit Rock Quality Designation (RQD) Range (%)
Schenley Redbeds	11.4	0
Birmingham Sandstone and Shale	54.0	0 – 100
Duquesne Coal, Clay, and Limestone	9.7	0 – 33
Grafton Sandstone	11.8 – 16.2	52 – 94
Unnamed Redbeds	8.7	0
Ames Limestone	2.9	97

The bedrock encountered in the Conceptual Design borings verified the project site geological information compiled during office research. As shown in Table 2, the site bedrock falls within the Grafton Member of the Conemaugh Formation and includes: the Schenley Redbeds; Birmingham Sandstone and Shale; Duquesne Coal, Clay, and Limestone; Grafton Sandstone; Unnamed Redbeds; and Ames Limestone. The presence of the Ames Limestone was significant to the verification of the project site geologic setting because the Ames Limestone is one of three primary marker beds in Western Pennsylvania geology.

SUBSURFACE DATA ANALYSIS

Site Subsurface Model

The project site subsurface model was designed based on a compilation of the information collected from published resources during office research, field reconnaissance observations, and the Conceptual Design subsurface exploration. Figure 6 shows the site subsurface model used to perform the remediation alternatives analysis.

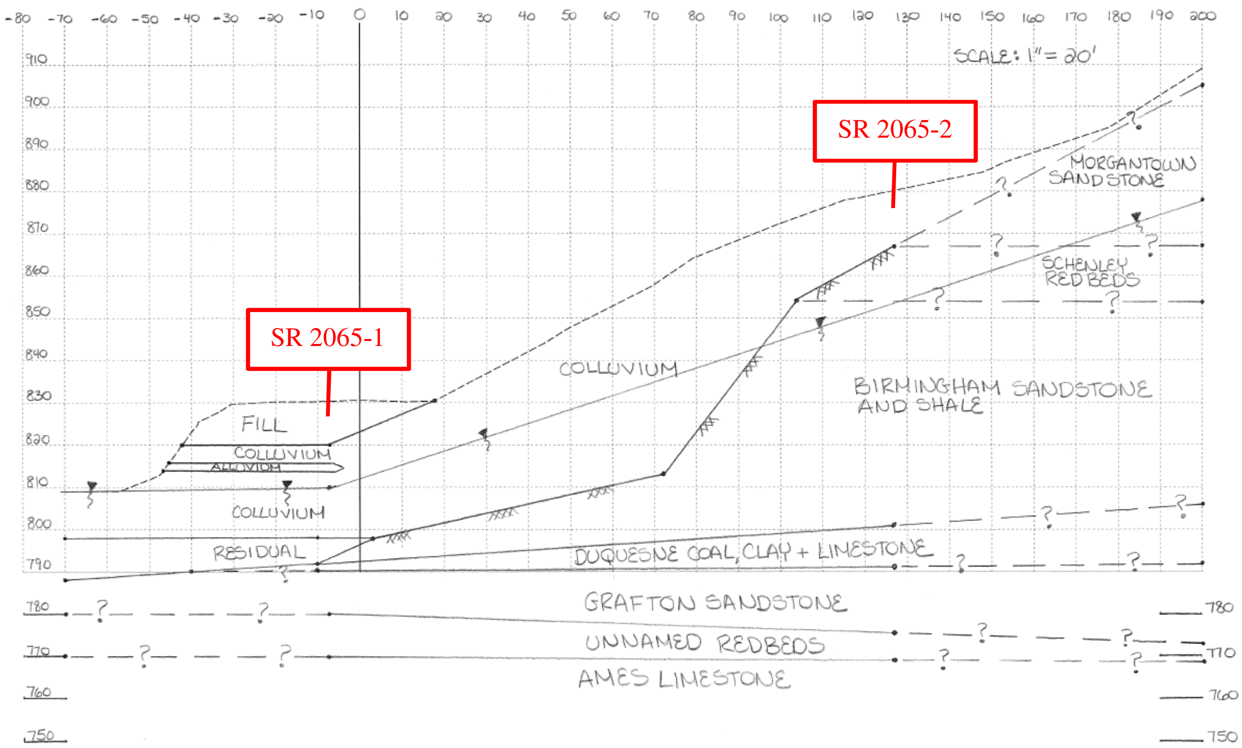

Figure 6 – Project Site Subsurface Model

The total soil stratum beneath Thompson Run Road is approximately 38 feet thick and consists of fill, colluvium, alluvium, and residual soils. The thicknesses of the model soil layers were based on the conditions encountered in boring SR 2065-1. Several pieces of information suggested that the top of bedrock dipped steeply within the right offset slope, creating a thick colluvial slope. The top of bedrock was interpolated based on the top of bedrock elevations in the Conceptual Design borings and the suspected right offset slope behavior derived from field observations. Landslide features, including leaning trees and float blocks, were noted throughout the right offset slopes during field reconnaissance. These observations, paired with the presence of a prehistoric landslide within the project limits, aided in the interpolation of the top of bedrock between SR 2065-1 and SR 2065-2.

The groundwater table included on the subsurface model was based on the groundwater level readings taken in the Conceptual Design borings as well as the Thompson Run stream elevation. The 24-hour groundwater reading in SR 2065-2 indicated the groundwater table was present at EL 853.7. As previously discussed, SR 2065-1 was drilled in the roadway; as a result, the boring was grouted upon completion. In lieu of a 24-hour water level reading, the 0-hour groundwater level reading in SR 2065-1 was considered for use in design. The 0-hour groundwater level in SR 2065-1 was at EL 809.9 which coincides closely with the Thompson Run streambed elevation. The resulting groundwater table, as shown in Figure 6, trends from the Schenley Redbeds in the uphill slopes through the thick colluvial soils to the Thompson Run stream elevation.

Representative soil properties were selected for each of the model soil types based on average $N1_{60}$ values in conjunction with published typical values. Representative bedrock parameters were established from the strata RQD and published typical strength values. The subsurface model geometry and material parameters were entered into GSTABL7 software to attempt to reproduce the existing landslide failure conditions observed in the field. The analysis of the active landslide area produced a Factor of Safety (FS) = 0.94, indicating marginal stability of the existing left offset slope. As a result, the subsurface material properties were considered to be representative of the project site and were utilized in the analysis of the landslide remediation alternatives.

Potential Basal Failure Surface

No signs of recent movement were observed within the right offset slopes during the late 2011/early 2012 field reconnaissance visits. As a result, the physical extents of the prehistoric landslide are unknown. The potential presence of a larger, prehistoric failure plane that extends into the right offset slopes was considered as the worst-case scenario for the project site.

Consequently, GSTABL7 software was used to analyze the stability of a potential basal failure plane through the thick colluvial slope. The GSTABL7 analysis of the potential basal failure surface resulted in a FS = 0.88 on the critical surface. A FS value less than 1.0 is an indication of instability; however, the absence of visible signs of recent instability in the right offset slopes signifies that a certain amount of healing would have taken place along the failure surface since the last movement. The project special provisions were written to require that the FS along the potential basal failure plane meets or exceed a FS = 0.88 for all proposed landslide treatments to avoid the potential for a large-scale failure during construction.

ALTERNATIVES EVALUATION

Project Site Limitations

The Thompson Run landslide remediation project site posed numerous challenges, both geologic and otherwise, that had to be considered in the selection of the treatment alternatives. Geologic limitations of the project site, as previously discussed, included the presence of the prehistoric landslide and the thick colluvial soil present beneath the roadway and in the right offset slopes.

The existing left offset slope geometry posed challenges due to the height and steepness of the slope. The left offset embankment has a height of 20 feet and a slope ratio ranging from 0.75H:1V to 1H:1V throughout the project site. Steepness and height of the existing slope provided a minimal amount of space with which to work to provide a stable finished slope.

The existing PennDOT right-of-way consisted solely of the left and right limits of Thompson Run Road and equaled a total width of 24 feet. All work performed outside of the roadway limits necessitates purchase of required right-of-way for any permanently changed property or a temporary construction easement for property utilized for construction access.

Union Railroad Company owns right-of-way on the stream bank opposite from the failing embankment. Due to the length of time required to obtain permits and property access from the railroad company, it was determined during Conceptual Design that all treatment alternatives would avoid requirements for access to the railroad right-of-way.

The Pennsylvania Department of Environmental Protection (PADEP) restricts alterations to the natural course of streams. In accordance with PADEP requirements, a lengthy permitting process is necessary to gain access to a stream in order to perform construction activities. Therefore, it was established in Conceptual Design that the project specifications would restrict the ability to encroach upon, or alter the course of, the Thompson Run stream.

Caisson Supported Slope with Reconstructed Rip-Rap Embankment

A caisson supported slope was selected for analysis as a potential remediation alternative based on the space restrictions posed by the existing left offset embankment. The use of vertical elements provides stabilization of the active landslide mass without requiring excessive room for construction. The reconstructed rip-rap embankment was included in the treatment alternative to prevent risking possible exposure of the caissons due to erosion of the existing soil slope.

As previously discussed, GSTABL7 software was utilized to perform a stability analysis of the existing slope as verification of the site subsurface model. The critical failure surface in the stability analysis of the active landslide area was at approximate EL 803. A caisson treatment must be long enough to cut off the active failure plane while keying into competent bedrock to effectively provide stability. Figure 6 indicates that the thickness of the soil over bedrock beneath Thompson Run Road is approximately 38 feet. The Grafton Sandstone and Shale bedrock beneath the road provided a minimum 12-foot thickness of competent bedrock with a Unit RQD ranging from 52% to 94%. Based on this data, the caisson design analyzed a 45-foot-long caisson that included a 5-foot key into the Grafton Sandstone and Shale bedrock.

The caisson size and spacing utilized in the alternatives analysis were selected based on the need to find an economical solution to meet the required loading capacity. A tangent caisson solution was initially considered; however, tangent caissons would not have provided an economical solution to remediate a proposed 370-foot treatment length. Upon considering several caisson sizes and spacings, 48-inch diameter caissons spaced at 12 feet center-to-center were selected for the caisson treatment analysis. The 12-foot center-to-center spacing was chosen for analysis purposes to take advantage of soil arching effects between caissons. A maximum center-to-center spacing of three times the caisson diameter (3 x 4 feet) was assumed to provide adequate soil arching effects without allowing for raveling of the soil between the caissons.

Reconstruction of portions of the left offset embankment was considered to be necessary as part of the proposed treatment due to the steepness of the existing slope. Removal and replacement of portions of the existing slope with a rip-rap embankment was analyzed for stability. The use of rip-rap as the embankment replacement material allows for the finished slope face to be placed as steep as 1.25H:1V without encroaching on the Thompson Run stream.

The caisson treatment analysis was performed by first resolving the passive and active forces applied to a single caisson into net horizontal forces. Only active and passive forces from the materials above the active failure plane at EL 803 were considered in the analysis. All subsurface materials below EL 803 were considered to be at-rest. The total passive resistance provided by the reconstructed embankment and soil in front of the caisson exceeded the active force of the landslide mass behind the caisson. Therefore, the caisson and rip-rap embankment reconstruction treatment was shown to provide sufficient resistance against movement of the landslide mass.

The maximum service shear and moment loadings for the proposed caisson configuration were determined through the use of LPILE software. Strength shear capacity of a single caisson was determined using GSTABL7 software. LPILE calculated the maximum service moment at 353 kip*ft. and the maximum service shear load at 76 kips. An iterative analysis was performed using GSTABL7 to determine the maximum shear capacity of a single caisson. The shear capacity of the 48-inch caisson was increased until the stability of the caisson treatment met a FS = 1.5. A single-caisson shear capacity of 500 kips achieved a FS = 1.5. Structural engineers confirmed that reinforcement in a 48-inch caisson can be reasonably designed to a maximum shear load of approximately 550 kips, therefore exceeding the required shear capacity of 500 kips per caisson.

GSTABL7 software was also used to evaluate the stability along the potential basal failure surface to ensure that the caisson slope treatment would not cause instability. As previously discussed, the stability along the potential basal failure surface has a FS = 0.88. The conceptual design was required to meet or exceed the stability of the existing conditions. The GSTABL7 analysis of the potential basal failure surface including the caisson slope treatment resulted in a FS = 1.13.

As a result, the caisson supported slope with reconstructed rip-rap embankment was determined to provide sufficient stability to prevent further movement of the active landslide. Figure 7 shows the caisson supported slope with reconstructed rip-rap embankment design detail.

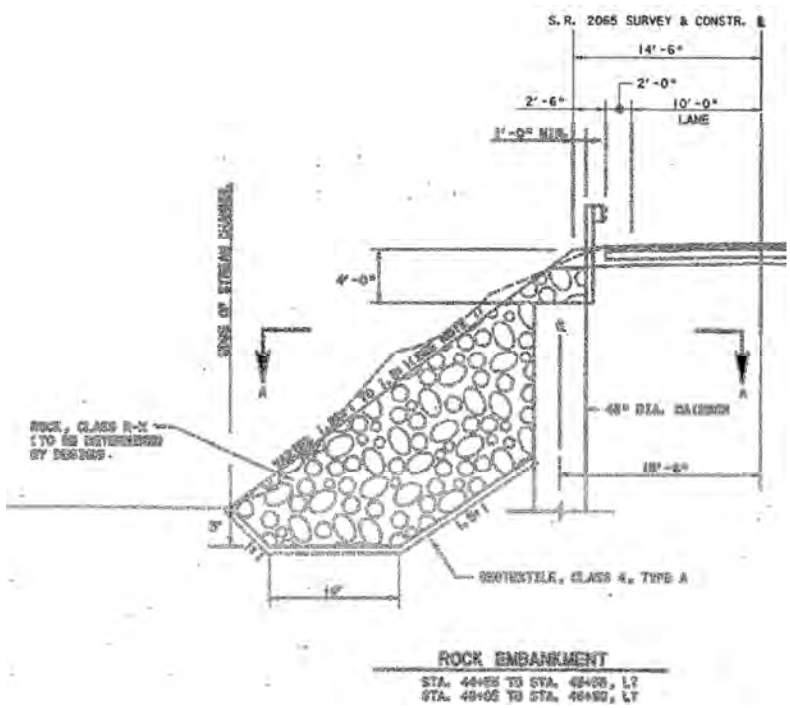

Figure 7 - Caisson Supported Slope with Reconstructed Rip-Rap Embankment Detail

Roadway Excavation and Replacement with Stabilized Material

A second alternative considered for the landslide treatment included excavation and replacement of the left offset embankment and subsurface beneath Thompson Run Road with stabilized material. This alternative was considered because it would eliminate the hazard of unstable soils within the active landslide area. Conceptual analysis of this alternative evaluated the potential excavation and replacement quantities as well as the stability of the remaining soils during construction. Analysis of this alternative was complicated by the unknown factors related to the temporary stability of the potential basal failure surface.

Soil Nail Slope Treatment

A third alternative considered for the landslide treatment was a soil nail slope treatment. Formal design calculations were not performed by Gannett Fleming for the soil nail treatment option. However, the alternative was considered based on the limited space available for the landslide treatment and the minimal space requirements for soil nail installation.

SELECTED PREFERRED CONCEPTUAL TREATMENT ALTERNATIVE

The caisson supported slope with reconstructed rip-rap embankment was chosen as the preferred alternative based on the ability of the design to satisfy the project site requirements. The remediation treatment fits within the physical confines of the site while addressing the natural and right-of-way challenges to provide adequate stability for the active landslide. The caisson supported slope with reconstructed rip-rap embankment alternative was presented as the preferred conceptual remedial design for solicitation of bids from Design/Build contractors. A selection of the design criteria for the caisson treatment as presented in the Geotechnical Engineering Report for Design/Build (GERDB) (*9*) is as follows:

- Caisson termination criteria: Advance the caisson a minimum of 5 feet into the Grafton Sandstone/Shale bedrock or to EL 785, whichever results in a longer caisson.
- Caisson diameter: 48 inches
- Caisson spacing: 12 feet center-to-center
- Maximum deflection = 0.15 inches
- Finished rip-rap slope face ranging between 1.25H:1V and 1.5H:1V.

The roadway excavation and replacement with stabilized material alternative and the soil nail slope treatment were presented as additional permissible treatment options in the GERDB. No additional treatment alternates are permitted for the project.

Bids were solicited for the Final Design and construction phases of the project with Raudenbush Engineering providing the geotechnical Final Design for the winning Design/Build team. Raudenbush chose to use the caisson supported slope with reconstructed rip-rap embankment as the selected treatment alternative. As of May 2013, the Final Design phase of the project is wrapping up and construction is anticipated to begin during Summer 2013.

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Rope Access for Rock Scaling Operations

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Disclaimer

Abstract

In 1989 the California Department of Transportation developed a rope access-training program that includes a code of safe operating practices and a corresponding training class for rock scalers, construction inspection, rockfall mitigation system maintenance, and geologic investigations. During the last 23 years over 1,600 students have successfully completed the training. The instructors have been trained by the Yosemite Mountaineering School, American Mountain Guide Association (AMGA), and most recently the Professional Climbing Instructors Association (PCIA). The techniques used utilize a combination of industrial and recreational climbing techniques. A manual and video are used during the class but the focus of the class is training on slopes in the field. There are two formal training sites and several back up sites with various slopes configurations ranging from 1 ¼: 1 to vertical. Two classes are available: an entry-level class and a refresher class. Each climber must attend the entry-level class then periodically attend the refresher class throughout their climbing careers. Emphasis is placed on basic skills and equipment for statewide uniformity in technique and equipment. Of the 12 Regional Transportation Districts, all have trained personnel. Nine districts have scaling crews. Three districts have an annual scaling program for slope maintenance.

Introduction

Scaling is the removal of loose rocks and material (that are marginally stable) from the face of the slope (Figure 1). One widely used method is to utilize specially trained crews suspended from ropes and using pry bars and other tools to dislodge marginally stable material (California Department of Transportation, 1985). Rock scaling is not “a random act of engineering” but is an organized, deliberate discipline founded on geologic and engineering principles and is a technique used throughout the world. As excavated slopes age, the aging process, often accelerated by winter storms and earthquakes, eventually weakens the surface of the slopes resulting in loose blocks of rock on the slope face. In time the slope surfaces need maintenance. While a variety of maintenance options and repair designs are available to mitigate rockfalls, rock patrols and rock scaling are typically the first line of defense. With over 3,000 miles of roadway in California having slopes with rockfall potential, maintaining them is a challenging endeavor (California Department of Transportation, 1985).

Figure 1. Scaling is the removal of loose rocks and material (that are marginally stable) from the face of the slope

Rock Scalers

In the 1860's during the great railroad project connecting Sacramento to Omaha, Chinese and Irish laborers moved loose rocks as the cuts were excavated (Ambrose, 2000). During the great dam projects of the early 1920's workers hung onto ropes with their bare hands or were slung in a crudely fashioned seat similar to a swing seat (Figure 2) (Redinger, 1949).

Figure 2. During the great dam projects of the early 1920's workers hung onto ropes with their bare hands

Following World War II with the introduction of mountaineering techniques from Europe, climbers started looking towards much safer techniques for accessing the slopes. At first these techniques, although sturdy, were heavy and limited movement on the slope. Today many improvements have been made for industrial climbing, recreational climbing and search and rescue climbing.

In the mid 1980's Caltrans engineering geologists began using these improved recreational climbing tools to access slopes for reconnaissance, mapping, and design purposes. While on the slopes loose rocks were removed as part of the investigation and quickly it was realized that these climbing techniques enhanced mobility and safety on the slope and suited more comprehensive scaling operations (Figure 3).

Figure 3. Caltrans engineering geologists began using improved recreational climbing tools to access slopes

Scalers work in small teams and have a need for many of the same tools and techniques used by climbers and mountain rescue teams including the need to use portable lightweight equipment, the need to easily position themselves in different locations, to build anchors, belay, rappel and ascend single ropes, and to solve common problems encountered in those activities. Scalers often have a need to use esoteric anchors utilizing small bushes, stakes, bolts, and pitons along with the desire to be able to approach and exit from the same direction (Tierney, 2013).

Caltrans engineering geologists realized this and were tasked to find training courses that would support the requirements needed for rock scaling operations. There were none to be found. There were search and rescue, industrial (structures), security and recreational training programs but nothing specifically for working on slopes for rock scaling.

Rope Access Training

Across the nation there are many programs teaching rope access for a variety of activities. These activities include:

- Recreational
- Search and Rescue
- Industrial (Structures)
- Military
- Security
- Avalanche Control
- Arborist

Each activity has its own unique gear requirements and techniques. While there is some overlap between each discipline many professionals and industry leaders clearly recognize these as separate and independent activities. In other words having been trained in search and rescue does not necessarily qualify a person for arborist's activities and visa versa. Industrial rope access is completely different in many ways from skills required on slopes from anchoring to mobility requirements. With that said, training in these activities does not necessarily transfer to the skills required to perform rock scaling on slopes. In scaling operations there is often the need for geotechnical and maintenance workers to be able to access slopes using 'light and fast' techniques (Vogel, 2013).

The California Department of Transportation recognizes the difference and has developed separate training programs for rope access. One is for working on structures, one for arborists, and another is for working on slopes for rock scaling. Training in one area does not qualify a person to work in one of the other activities. The Department clearly recognizes they are different and require different skills and techniques.

The State of California Department of Industrial Relations also recognizes this difference and categorizes a laborer who performs rock scaling and drilling (while protected from falling by rope and harness) as a High Scaler. There are no Structures or Bridge trades in this labor group, hence a clear distinction between working on slopes and on structures. In the Northern region of California, this classification falls under Construction Specialist, Group 1 (A), which is largely represented by Drillers and other Geotechnical trades. In the Southern region it is Group 3, which is similar in represented trades but includes underground workers such as pipe layers. There are no Structures or Bridge trades in these labor groups.

A search for Request for Proposals (RFP) with the description of High Scaling landed several advertisements for Rock Scaling and Log Scaling Contractors. Each of these two applications are very similar in description of work: a worker lowers himself down a very steep slope, working from a climbing specific rope, to assist in the removal of objects from that slope. Each RFP had varying levels of minimum experience requirements for the scalers but were very specific in that the work experience be on slopes.

Based on this information, it seems that the High Scaler is particularly unique, in skills/trade, and separate from structures work in the catagories of the California Department of Industrial Relations and the California Department of Transportation.

Caltrans Rock Scaling Training

Initially Caltrans engineering geologists and maintenance personnel went to Yosemite Mountaineering School where they learned basic skills for ascending, rappelling and anchor building. Then in the late 1980's Caltrans Engineering geologists were tasked with developing a class to teach the skills needed to access the slopes with ropes for rock scaling and blasting operations.

Engineering geologists, in the early 1990's, working closely with Caltrans rock and avalanche blasters began developing a class to train workers in scaling and teach the skills needed to access the slopes with ropes. First a maintenance code of safe operating practices was developed entitled "Bank Scaling and Rock Climbing." In conjunction with this an 8-hour class was developed and taught at the old Maintenance Equipment Training Academy (META) facility at Camp San Luis Obispo, California.

Since those early classes many changes have occurred. Through ongoing training with the American Mountain Guide Association (AMGA) and the Professional Climbers Instructors Association (PCIA) the training program has evolved into a 16-hour program and taught at the

Kingvale Maintenance Academy located in the Sierra Nevada Mountains in Kingvale, California (Figure 4). There are two formal training sites and several back up sites with various slope configurations. Two classes are available: an entry-level class and a refresher class. Each climber must attend the entry-level class then periodically attend the refresher class throughout their climbing careers. A manual and supporting video have been developed and there are over 20 volunteer trained instructors. During the last 23 years over 1,600 students have successfully completed the training.

Figure 4. Kingvale Maintenance Academy located in the Sierra Nevada Mountains in Kingvale, California

Caltrans Rock Scaling Program

The rock scaling program has been divided into three categories:

1. Rope skills and techniques
2. Slope stability
3. Scaling operations

The "Caltrans Bank Scaling and Climbing" training program as discussed above provides rope skills and techniques training internally.

Slope stability assessment is two tiered. First, the slope is assessed by a competent person or persons who evaluate the slope to identify:

1. Slope stability

2. Scaling suitability
3. Scaling degree of difficulty

The preliminary form used in this assessment is shown in Figure 5. Depending on the assessment results a second level of review is performed by geotechnical personnel. However a second level review is not always required.

Slope Assessment Form
Date, _______, Reviewer. *_________, Dist. __, CO.___, Rte. ___, PM_____
Previously Classified yes no if yes when__________

Slope
Cut Slope Natural Slope Old Slide (requires 2nd level Geotech review)
Slope Length (sketch in cross section)
<100', <200', <400, >400' (significant exposure time)
Slope Angle (sketch in cross section)
35 to 45, >45 to 70, >70
Slope Materials (sketch in cross section)
Soil Soil and Rock Rock
Rockfall Size___________
Slope Width (sketch in front view) ________________
Slope Above (anchor location) (sketch in cross section)
Access ______ Angle _____ Vegetation ________ Rock Outcrops ______
Presence of Water (sketch in cross section and/or front view) ________________
Chutes (sketch in front view)
How Many__________ Where ___________
Overhangs (sketch in cross section and/or front view)
How Many __________ Where ___________
Previously Scaled yes no
If yes when_________ and how many yards scaled_______
Slope Activity
Seasonal ___________ Active _________ Extremely Active ____________

Cross Section Sketch (use range finder)	Front View Sketch

File copy with Supervisor, in Personal Climbers Log, and Send copy to qualified person * with digital photos.

Name of Submitter______________________ Date_______________

Note: A qualified person is a Caltrans Engineering Geologist or Transportation Civil Engineer and is a certified Caltrans climber and has participated in scaling operations.

Figure 5. Caltrans Slope Stability Assessment Form

Every slope is different and is distinguished by its size, character, and properties. Assessing these characteristics falls into the responsibility of engineering geologists and maintenance personnel. Maintenance forces know their slopes and understand each slopes behavior for rockfall. Engineering geologists study the rock properties, structure, and slope geometry as they relate to rockfall behavior. Together, the slope can be properly evaluated and the decision when to scale or not to scale is determined.

Scaling operations can begin once the slope is assessed. Maintenance personnel are directed to the Maintenance Manual chapter on Rock Scaling. The manual provides guidelines for scaling operations and relies on the supporting “Caltrans Bank Scaling and Climbing” manual and Caltrans “Rockfall Mitigation” manual. In addition each climber maintains a climbing log, which includes climbing training and climbing projects. Included in this log is a copy of the slope assessment form (Figure 5). This enables the scaling supervisor to place each climbing team member in in the appropriate position (scaler, spotter, ground control, etc.) based on experience.

Summary

Proper slope assessment and appropriate climbing training are essential for a successful and safe rock scaling operation. Historically, there was no available training for rock scaling for Caltrans employees. Standard training for recreational climbing, search and rescue, security, and structures are very task specific. That said it should be acknowledged that rock scaling on slopes is also task specific. Today it is common for some practitioners to train for industrial/structures climbing. But the difference between working on slopes and working on structures is significant and each requires different skills, techniques and equipment. The Caltrans program has strived to fill this gap. Over 1600 students have successfully completed the training. In all twelve transportation districts in California all have trained scalers. Nine have scaling teams and three have regular scaling programs. Caltrans scaling training program is the only one of its kind and Caltrans regularly receives inquires about the program from across the United States and abroad. Caltrans scaling teams have been featured in the National Geographic special “Landslides” and the Learning Channel Special “Disaster Detectives.” It is truly a unique program enabling Caltrans to employ best management practices in managing and maintaining its highway slopes.

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Rock Slope Protection and Rock Mechanics

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ABSTRACT

Rock slope stabilization requires (1) engineering geology/geotechnical engineering input, (2) selection of suitable protection means and (3) a means for evaluating the nature and level of protection appropriate for the particular project. The interrelationship between these requirements is presented using the example of an existing, typical hard rock slope along the entrance road to a large development. Solutions using conventional analyses and the Geobrugg SPIDER® and RUVOLUM® Online Tools were used to develop a design for rock mechanics problems on a slope.

The programs are online tools available to engineers and designers and where the user will input site conditions based upon field evaluation, be able to select anchor spacing and size and see results that are in an optimized arrangement. The programs are based on Mohr-Coulomb Equilibrium theory and it establishes the relationship between driving and stabilizing forces. The programs use a trial and error method and it is quite easy to change the input parameters. Unfortunately, the programs cannot currently analyze wedge failures. However, as an example of the procedures we will use, a wedge failure analysis that is performed in a conventional manner to provide rock discontinuity strength properties for use in the programs for a suspect, wedge-shaped body within the rock type.

Mapping, analytical and evaluation procedures are straight forward and can be used by any competent geotechnical organization charged with developing appropriate rock slope stabilization. The information collected is critical for the program. The last step in the process is installation and using a qualified and experienced rock slope remediation contractor is the best approach. The contractor should be also able to provide assistance during the project development stage.

INTRODUCTION

Rock slope protection and the need for rock slope evaluation in design are well recognized. The symbiotic relationship between rock mechanics and rock slope protection is relatively obvious. There are only two ways to evaluate the stability of a rock slope; let it fail or analyze and evaluate with the available data. It is generally preferable to provide some means of slope protection before a failure occurs so as to avoid the possible loss of life, property or significant economic penalties. However, it was a large rockfall onto the main entrance road to a residential development that finally prompted remediation efforts on, at least, the most critical areas of the site.

In this paper, the authors examine this symbiotic relationship by using a typical hard rock slope, the Geobrugg Ruvolum Rock System and the Geobrugg Spider® Rock Protection System. The approach, described herein, essentially describes the process that one would follow after an engineering geologic evaluation of the slope has identified potential hazards, or after a rockfall (small one we hope) has occurred. Rockfall Hazard Rating Systems are currently in use by a number of highway departments and/or their consultants (e.g., (*1*) and (*2*)) and hence, are an excellent precursor for the evaluation of an actual failure, represented herein.

SITE GEOLOGY

The example site is in northern New Jersey just to the west of the Ramapo Fault. The rocks underlying the site are shown on Figure 1, Site Geology Map, and outlined below. The slopes were mapped conventionally as subsequently discussed, but unfortunately, remediation was not undertaken until the rockfall shown on Figure 2 occurred early one February morning.

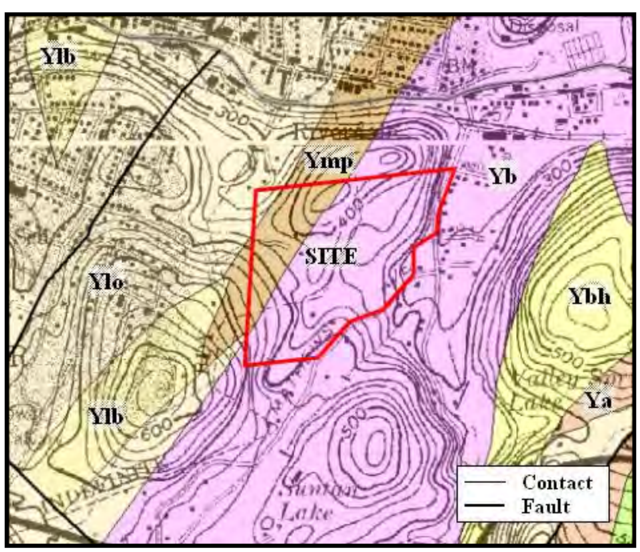

Figure 1 – Site Geology Map

The slope behind the Figure 2 rockfall lies along Timber Ridge Road, an entrance road to the site of a townhouse and condominium complex, and is only one of the slopes of concern at the site. The slope where this rockfall occurred will serve as the example used in this paper.

Figure 2 –Rockfall

The slope along Timber Ridge Road is some 1,300 feet in length and reaches a maximum height of more than 70 feet. The slopes at the site were evaluated using conventional means as subsequently discussed

The preliminary report (April 2004) identified the local geology using New Jersey Geological Survey Data (see Figure 1). The various rock types shown on Figure 1 are:

Yb: Biotite-quartz-feldspar gneiss - Moderately layered and foliated gneiss.
Ymp: Clinopyroxene-quartz-feldspar gneiss - Commonly interlayered with amphibolite or pyroxene amphibolite.
Ylo: Quartz-oligoclase gneiss –Contains thin amphibolite layers.

Ybh: Hornblende granite –Some phases are quartz syenite or quartz monzonite.
Ya: Amphibolite –Some amphibolite is clearly metavolcanic in origin, some metasedimentary, and some appears to be metagabbro.
Ylb: Biotite-quartz-oligoclase gneiss – Com-monly interlayered with amphibolite.

SLOPE MAPPING

Geologic mapping and analyses followed a preliminary (diagnostic) evaluation that indicated a large number of suspect locations along roadways and behind housing units. A survey line was set-up along the base of the various slopes and the geologic mapping progressed from the ground upward, including the use of a man-lift for the highest and steepest slopes. Slopes up to about 70 feet in height were mapped by these methods.

Essentially, conventional mapping techniques were used to gather a broad distribution of data regarding the attitude, orientation and condition of discontinuities present in the rock masses comprising the various slopes present at the site. Mapping was accomplished at a scale of 1-inch = 10-feet or 1-inch = 5 feet, vertical and horizontal, depending on the detail required to represent the geologic conditions observed. An articulated boom was used to reach the higher portions of slopes. Other slopes, where the boom truck could not be physically positioned, were climbed and mapped where it was safe to do so. Specific information collected in preparing the geologic section maps included: 1) the type of rock present at that location; 2) the strike and dip of discontinuities mapped; 3) the rock mass rating, where enough diagnostic characteristics were present to make an interpretation; 4) the character and nature of any observed joint-filling material; and 5) the presence of water (and sometimes ice flow) emanating from the mapped fractures that could affect the stability of the slopes.

ANALYSES

The resultant sections of geologic data were evaluated utilizing conventional rock mechanics analytical techniques (e.g., (*3*)). The techniques included:

1. *Stereographic projection of data* on an equal area stereonet, where mapped planar discontinuities are shown as traces of planes on a reference sphere in two dimensions. These traces of planes define the dips and dip directions of the mapped discontinuities as taken from the slopes in the field. The analysis defines the structural fabric present in the various rock slopes at the site and affords the opportunity to evaluate whether a kinematically-possible failure mode is present in the rock mass being evaluated. There were several types of rock mass failure modes potentially occurring, including planar, wedge, toppling, and raveling failures. Once it had been identified that a particular failure mode was kinematically

possible for a set series of mapped discontinuities for a slope, analyses were performed to evaluate the stability of the rock slope.

2. *Stability Analyses* of the various slopes present at the site employed a limit-equilibrium approach, wherein the shear strength along potential failure surfaces, the effects of pore-water pressure, and the influence of external forces were considered. The geologic data gathered and analyzed under the previous item were used as input to a spreadsheet program where the basic stability equations were resolved for each case considered.

3. Assumptions as to strength properties along rock discontinuities were made for the analyses as is conventional for extensive slopes in metamorphic or intrusive rocks. The assumptions made were consistent with analyses for similar rocks in the literature (e.g., (*3*)). Typically, a rock density of 160 pounds per cubic foot (pcf), an internal friction angle (Φ) of 30 degrees and cohesion of 70 pounds per square foot along the failure surfaces were used.

GEOLOGY OF THE SLOPE AT TIMBER RIDGE RD

The rocks along Timber Ridge Rd. change in orientation along the descent from the development to the main thoroughfare, hence the stability conditions of the rock slope face change in relation to critical joint orientation and foliation direction.

The general condition of the rock in this area of the site varies from extremely weathered at the northwesterly end of the road to weathered along the rest of the exposure. The rock is biotite-quartz-feldspar gneiss with varying amounts of garnet and sillimanite as accessory minerals. In appearance, it is gray-weathering, locally rusty from the iron-bearing minerals present, gray to tan or greenish-gray, fine- to medium-grained, moderately layered and foliated gneiss.

The geotechnical report of the analyses using the previously noted strength parameters described the portion of the slope that eventually failed as: “Between Stations 67 and 69 the slope face is at a modest angle with the foliation strike and dip, which makes wedge-type failure the favored kinematic model for this area. Critical joint and foliation surface orientations were modeled for this portion of the slope and it was found that factors of safety between 1.1 and 1.4 were calculated, suggesting a marginal factor of safety against failure at this location. At Station 68+20 feet, a highly altered and deformed zone occurs in the rock with gouge present on all joint surfaces. This deformed zone is extremely weathered and appears to be a flexural-induced fault zone that occurred during folding of the rock. Foliation surfaces within exposed overhangs in the rock slope face were open up to a ¼-inch. The wedge fell during winter weather (freeze/thaw, rain/snow cycles).

Figure 3 shows the stereographic projection (lower hemisphere) of the failure surfaces involved for the suspected failure location. As can be seen, the intersection of the release joint, which strikes North 25 degrees west and is near vertical, with the gouge-filled foliation plane falling within the instability region of the diagram. Because one could measure the size of the block that fell, it was possible to back-calculate what the equilibrium conditions were just prior to failure. Using a density of 160 pounds per cubic foot for the biotite-quartz-feldspar gneiss, and a measured failure surface of 140 square feet, it was estimated that the cohesion along the failure surface at a safety factor of 1 would be about 330 pounds per square foot (if it were dry).

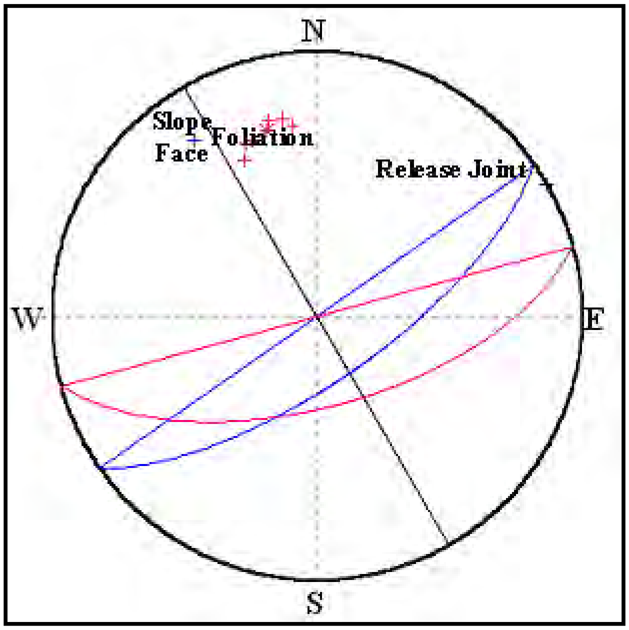

Figure 3 – Failure Surface at Timber Ridge Road, Sta 68+20

However, there was more than normal precipitation during the time of the failure, making it likely that water in the slope, coupled with the freeze/thaw cycles, contributed significantly to the failure. With water present in the slope along the foliation failure surface, the cohesion would drop off significantly to about 70 pounds per square foot just prior to failure. Figure 4 shows the location of the failure as it was originally mapped in July 2004 and Figure 2 shows the actual failed surface and block.

Figure 4 – Failure Location Sta 68+20

Thus, the slope mapping and the geotechnical analysis of the failure provided input for the Geobrugg® Ruvolum® program and SPIDER® program to design a protection system that would have prevented the observed rock fall.

DESCRIPTION OF THE ROCKFALL PROTECTION SYSTEM

The TECCO® Slope Stabilization System was developed for soil and highly weathered rock slopes and the Ruvolum® program is used to design the anchor spacing based on anchor type. The site conditions are entered into the program along with an anchor size and spacing and the program will provide an "okay or not okay" for the anchors and for the mesh. The program also does a number of proofs for bearing safety which can be reviewed.

The anchors go across and down the slope using the spacing determined by the program; plus, the spacing is staggered down the slope. If the shape of the slope and material changes it is possible to change the anchor spacing. The mesh is TECCO® Mesh G65/3 which is single twist construction and the mesh has an elongated diamond shape. The mesh is made with 0.118-inch (3mm) diameter galvanized high tensile strength (246 ksi) alloy steel wire. Spike plates, which have an elongated diamond shape, go on the anchors and they are used to pin and tension the mesh against the slope. The key to the system performance is surface contact.

The SPIDER® Slope Stabilization System was developed for non-weathering rock slopes and the design presented herein was done using the SPIDER® program. The site conditions are entered into the program along with an anchor size and locations and the program does a number of proofs bearing safety which must be fulfilled for the design to work.

The anchor or anchors are placed above and below plus along both sides of the rock block or formation using the arrangement determined by the program. Depending on the size of the rock formation or block, the combination of anchors above, below and along the sides may not be adequate to stabilize the rock. Anchors can be installed in the rock and the program used to redesign a new arrangement. The first net checked was SPIDER® Net S4-230 which is single twist construction and the mesh has an elongated diamond shape. The net is made with 1x3 strand and the strands are made with (3) 0.157-inch (4mm) diameter galvanized high strength alloy steel wires. Spike plates go on the anchors and they are used to pin and tension the net against the slope.

PROCEDURES FOR PROTECTION REQUIREMENTS

Currently, the Ruvolum® analyses cannot handle a wedge failure in the conventional manner. However, we believe that a conservative procedure would be to use a block of the same weight and lateral dimensions.

The Ruvolum® program requires information on slope angle, the thickness of the overburden layer, the friction angle and cohesion of the overburden materials, the "volume weight of the ground" (unit weight) for the overburden, the "slope-parallel force" and the safety factors desired for various portions of the design. It also has provisions for earthquake forces and water pressures. Aside from the slope characteristics, the program requests the desired anchors to be used (with options to add the specification for anchors not immediately available from the provided list) and how much load is applied when they are to be tensioned.

Once these parameters have been established, the program will provide a "pass or fail" response as to whether the mesh and nails can handle the strain. The spacing of the anchors is easily adjusted so the most appropriate arrangement, using the least amount of anchors, can be determined.

The results of our analyses using the Ruvolum® Online Tool established that the TECCO® System did not have adequate strength to hold back the mass. The program is available on-line from the Geobrugg website (http://applications.geobrugg.com/). You will have to request a password prior to having access to the program through the website. However, the system would have proved quite useful for the majority of the subject site to reduce the need for rock scaling, overburden soils failures and failures of smaller blocks than the case analyzed herein.

Therefore, we undertook the same analysis using the SPIDER® Online Tool. Similarly to the Ruvolum® program, the SPIDER® (sometimes referred to as Ruvolum Rock) program is available on-line from the Geobrugg website (http://applications.geobrugg.com/). You will have to request a password prior to having access to the program through the website.

The program input starts simply by requesting the same slope angles, material characteristics (less the overburden) and anchor type. However, the section of rock is stabilized by surrounding and tensioning the potential failure rather than using the grid pattern throughout the slope as TECCO® System does. It is used to calculate the number of anchors above, below and to the sides of the potential failure with the lower anchors taking the majority of the stress.

The most difficult portion of the SPIDER® program is determining the geometry of the anchored mesh to the slope and the forces related to the area being remediated (see Figure 5). Force P is the stabilizing force required to hold the block in place. Zu and Zo are the applied direction of the stabilizing force in the line of the net and are based on the angle the net makes with respect to the slope. This geometry is determined by the shape of the area and anchor location above, below and along the sides. For instance, if the block extends out from the face, the angle that the net makes once anchored below the block (Z_u) would be different than the mesh angle to the top anchors (Z_o). The area to be remediated needs to be mapped in order to determine these angles.

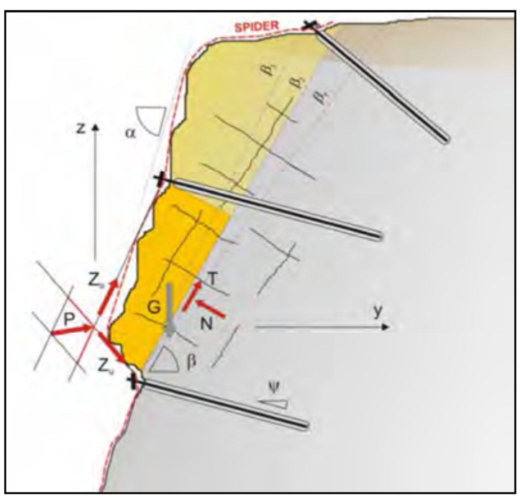

Figure 5 – Forces Acting on the Formation

If lateral anchors are to be used, they have a lateral stabilizing influence and need to be taken into consideration (see Figure 6A & 6B). Force S is the lateral stabilizing force and it is transferred laterally to anchors, nails and boundary ropes on both sides of the block. The field mapping will determine the angles the mesh makes across the slope.

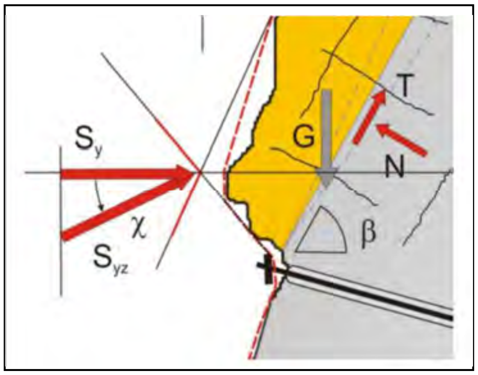

Figure 6A - Resultant Lateral Force in the Line of the Slope

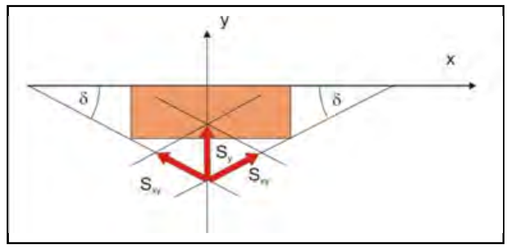

Figure 6B – Resultant Lateral Force across the Slope

CONCLUSIONS

The rockfall experienced at the site was a small and relatively simple exercise for the SPIDER® program and the weights and pressures were easily handled by the SPIDER® Rock Protection System using almost any anchor. In our experience, we did not expect the system to handle such a large rockfall with just seven anchors. In the end we would likely chose the configuration of the three anchors on top, one on either side and two at the bottom so as to limit the exposure of any personnel placing anchors below the potential failure area. The analyzed configuration is shown on Figure 7.

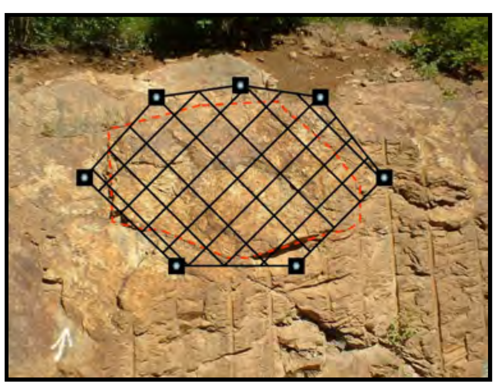

Figure 7 – Configuration of anchor pattern chosen from the analyses performed in this paper.

The Ruvolum® Online Tool was the easier program to use but it also showed that the system was not adequate for the block in question. The SPIDER® Online Tool was not difficult to use but it does require more information about the site in order to accurately determine a solution. The information is obtained through mapping the location.

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Testing of Rockfall Post Foundations in Colorado

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ABSTRACT

The Colorado Department of Transportation (CDOT) recently conducted testing of different types of post foundations used to support rockfall barriers and attenuator systems in Colorado. Current testing of rockfall barrier systems typically does not involve impact testing of the posts but rather impact testing to the center of a net or panel system that transfers a portion of the loading to the post foundation. These transferred loads are a fraction of the load that would be generated from a direct impact to a post.

Based on full scale rock rolling tests in Colorado, in which posts were knocked down during a rock rolling event, it was evident that if the post and foundation system could resist at least one or two direct impacts during a multiple rockfall event, the performance of the rockfall barrier or attenuator system could be greatly increased. Additionally, understanding the failure characteristics of the post foundation system could provide insight into reducing maintenance costs and improving management practices of these systems. To determine the loading conditions and evaluate the effectiveness of various foundation designs under direct post impacts, a pendulum test site was constructed in Colorado to generate at least 220 kJ of impact energy.

The post foundation testing to be discussed in this presentation consisted of 29 direct post impacts. The testing conditions ranged from a rockfall post connected to only a base plate in contact with the ground, to a post that was attached to a 6 foot deep (1.8 m), 36 inch (0.9 m) diameter foundation. The testing also consisted of various combinations of uphill retaining anchors that were instrumented with load cells on the foundations and load cells on the retaining anchors.

INTRODUCTION

Rockfall hazards are common along highways in Colorado. Rockfall incidents create safety risks to the traveling public and cause economic hardship by disrupting mobility and the performance of transportation corridors. Mitigation of these rockfall hazards has become an important concern of transportation authorities, especially those located along heavily travelled major highways. Mitigating hazards along corridor systems so that rock does not reach the roadway is a paramount goal for the Colorado Department of Transportation (CDOT).

Over the past decade, full scale testing of attenuators and rockfall barrier fences in Colorado has yielded valuable insights into failure and damage of these systems. During full scale rock rolling it was evident that if a post could stand and support an attenuator or fence barrier for an additional few seconds during a rockfall event or for multiple rockfalls over a longer time period, the likelihood of rockfall reaching the roadway could be greatly diminished. Many current practitioners believe that a rockfall post system is considered a temporary or replaceable system that only functions to hold up the rockfall panel system. This is correct; however, if a post system can be designed to function for multiple rockfall impacts, the increased performance of the system is of great benefit. Additionally, many practitioners do not design for a direct hit to a post. Typical rockfall barrier designs only account for an impact to a panel system in which the loading is transferred to the post and anchor systems. Manufacturers generally have instrumentation data obtained from secondary loadings on the post systems resulting from direct panel impacts, but do not necessarily have data for direct post impacts. The intent of this paper is to evaluate and gain insight into the design of post foundations for further use in:

- Failure mechanisms from direct impacts to a post and post foundation system
- Maintaining a functioning post system during multiple impacts
- Asset management

CDOT and Yeh and Associates, Inc., specifically focused on testing the interaction of posts and post foundation systems at a constructed test facility near Empire, Colorado. A multitude of systems were tested at this facility. This paper describes 29 direct post impacts to 4 different post foundation systems with various combinations of uphill retaining anchor systems. The testing instrumentation consisted of load cells on the foundations and load monitoring cells on the retaining anchors.

TEST SITE FACILITY

The test site facility consisted of two sets of braced W10x74 posts that were approximately 35 feet (10.6 m) to 30 feet (9.1 m) in height, and spaced approximately 20 feet (6 m) apart (Figure 1). The 30 foot (9.1 m) high frame functioned as the main support for swinging rocks into the test post foundations. The 35 foot frame (10.6 m) was used for leverage to pull the rock into place in order to release the rock to impact the post system. A forklift was used to pull the rock into place. Once set, the test rock was released with a pneumatic device that was developed by Protec Engineering of Japan.

Figures 1 and 2 depict the overall test setup showing the location of the two frame systems with a rock in position for impact to a post. Figure 3 depicts the pneumatic release system components developed by Protec Engineering. The frame posts were held in place by concrete foundations and support tie-wires similar to telephone pole supports. Rather than grouting anchors in-place which likely would have been pulled out over time during testing, uphill retaining anchors were simulated by using stacked Type 7 concrete barriers to approximate a 15 to 50 kip (67 to 222 kN) pullout capacity depicted in Figure 4.

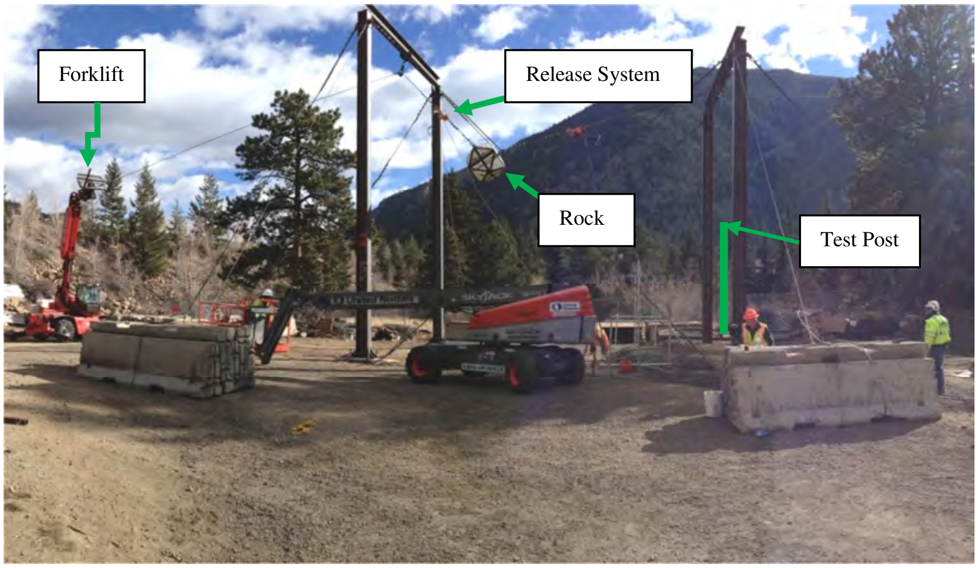


Figure 1. Overview of test site. Frame on left is approximately 35 feet in height. Frame on right is approximately 30 feet in height. Rock is hoisted in center of photo. Release system and test post location depicted in green.

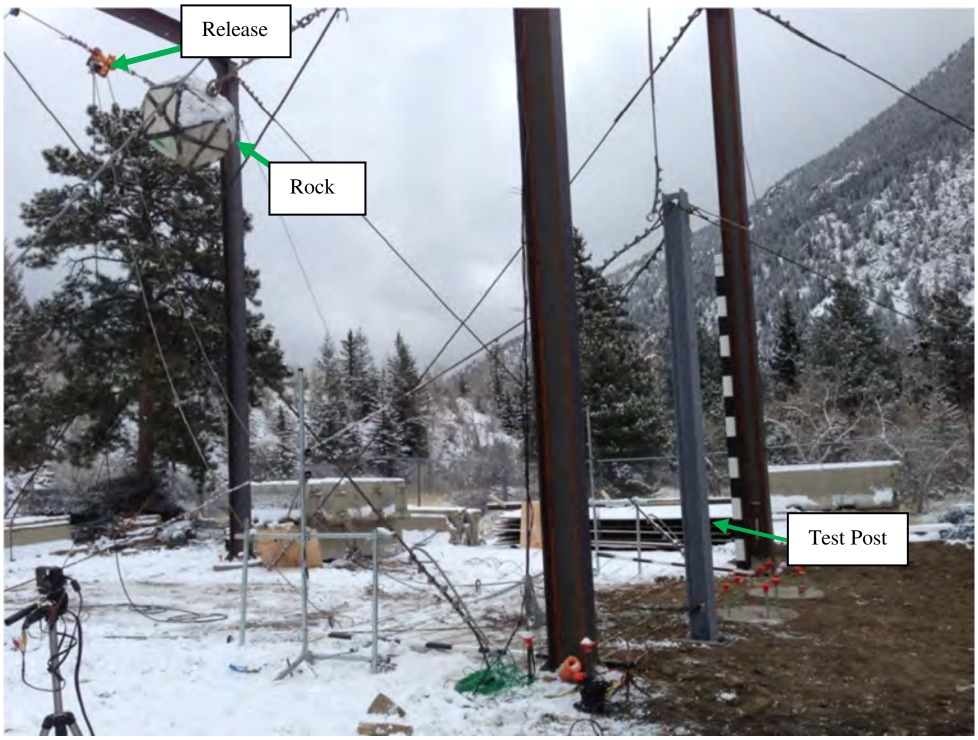


Figure 2. View of rock prior to impact with post. Orange pneumatic release device is located in upper left. Gray test post is depicted with top and bottom retaining wire ropes.

Figure 3. Pneumatic release systems used to release the rock.

Figure 4. Stacked Barrier Anchoring System. Green arrow depicts load (link) instrumentation to record loads in retaining wire ropes.

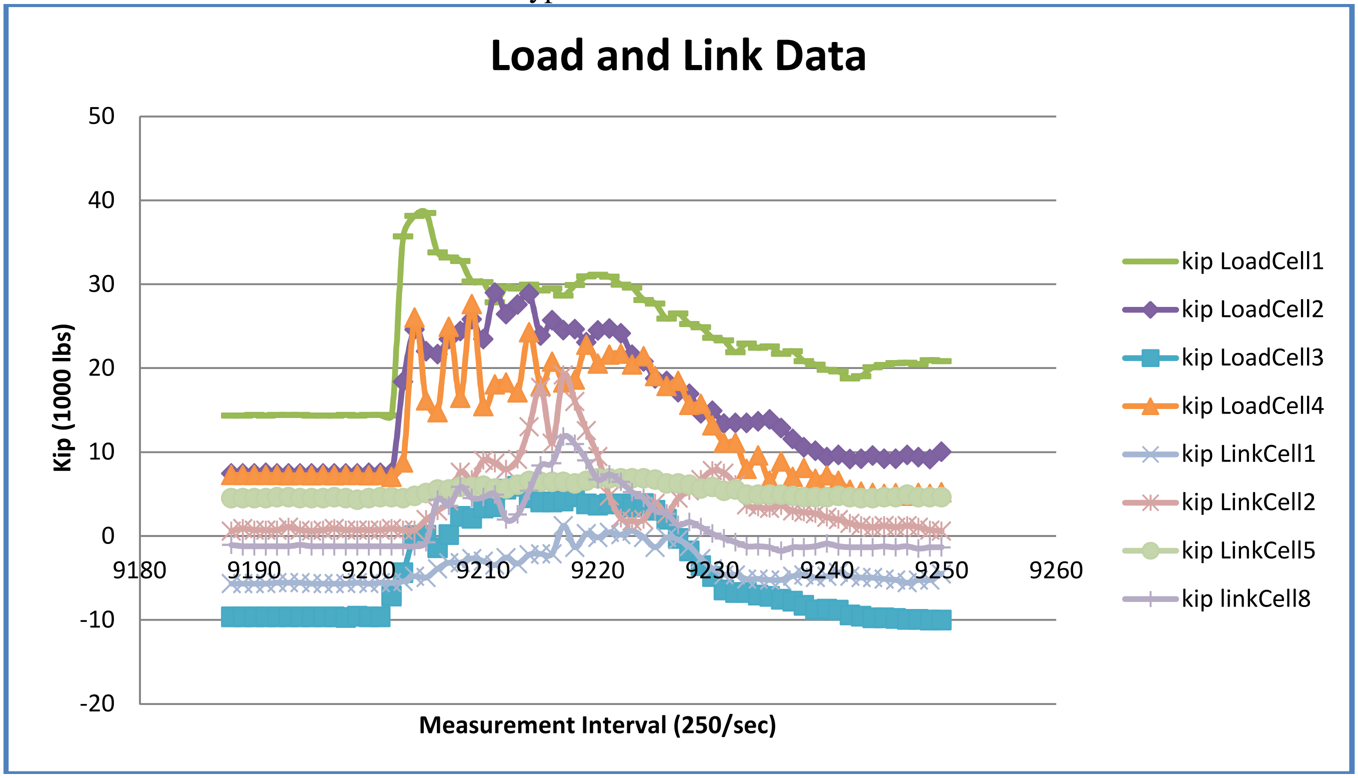
INSTRUMENTATION

Load cell instrumentation was attached both to the retaining wire ropes (Figure 5) and to the base plate of the post systems (Figure 6). The load cells, otherwise known as link cells (Figure 5), were custom manufactured by Geokon for measuring the loads in wire rope up to 100 kips (445 kN). The post base load cells (Figure 6) were also manufactured by Geokon to measure the compressive loads transferred in the base plates to the load cells. To measure the effects of tensile loading, the all-thread nuts were tightened and the loads recorded prior to the testing to determine the difference in compressive loading (i.e. tension) in the load cells. Table 1 depicts a typical graph of the four load (link) cells on the retaining wire ropes and load cells on the base plate. The loads measured were changes in loading rather than absolute values.

Figure 5. Close up of load (link) instrumentation to record loads in restraining wire ropes.

Figure 6. Load cells in compression on post system.

Table 1. Typical Instrumentation Load Chart.



TESTED POST FOUNDATION SYSTEMS

A total of 29 post foundation tests are described in this paper. A total of 4 separate foundation types were tested. The foundations consisted of:

1. Shallow foundations with various combinations of top and bottom support ropes.
2. Grouted bar foundations with various combinations of top and bottom support ropes.
3. Shaft foundation systems consisting of 3 feet (0.9 m) diameter shafts with 3 feet (0.9 m) and 6 feet (1.8 m) of embedment.
4. Base plate only with top and bottom retaining support ropes.

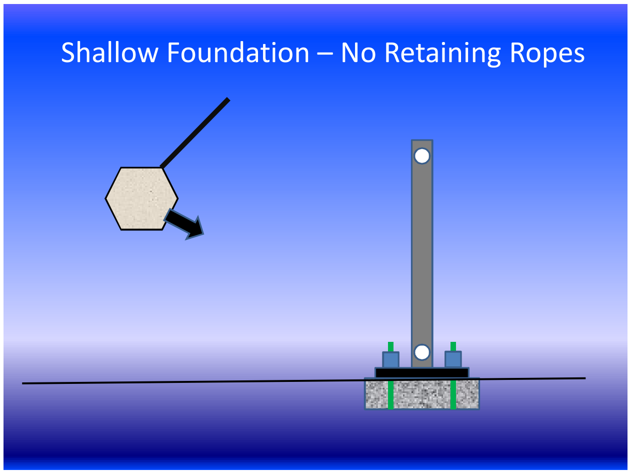
For the purposes of this paper we have depicted the test setup with the actual photographs that correspond to the test.

Shallow Foundation System

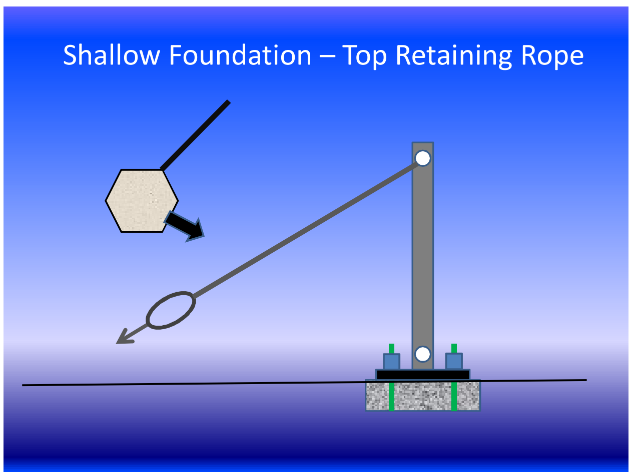
The shallow foundation system consisted of a W8x48 post welded to a 20 inch x 20 inch x 1 inch (51 cm x 51 cm x 2.5 cm) base plate that was attached to a 36 inch diameter (0.9 m) concrete pad cast 6 inches (15.2 cm) into the subsurface. Four, all-thread number 8 bars were cast in the concrete to provide a connection between the post and foundation. Various combinations of retaining anchors were tested as depicted in the following figures. Load cells were placed on top of the base plate and tightened. Figures 7a, 7b, 8a, 8b, and 9a and 9b depict the system.

Observed Results

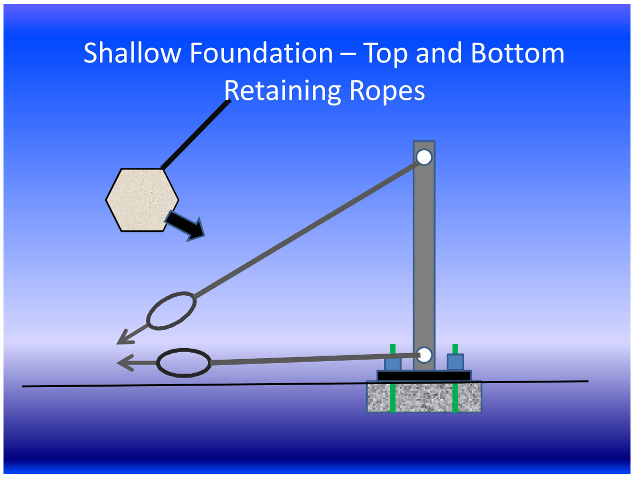
Overall, the top and bottom retaining ropes were necessary to maintain an effective system. At the time, the shallow foundation system with top and bottom retaining support was only tested to 67 kJ. The results of this test led to other tests in which only the steel base plate was placed on the ground with both top and bottom retaining ropes. These later configurations were tested to 218 kJ, indicating it is likely that the shallow foundation system with top and bottom support ropes could have been tested up to 200 kJ and still been effective.



Figures 7a and 7b. Shallow Foundation - No retaining ropes. System subjected to a 9 kJ impact. (Note - Post simply fell over)



Figures 8a and 8b. Shallow Foundation – Top retaining ropes. System subjected to a 20 kJ impact. (Note – bottom of foundation began to kick out)



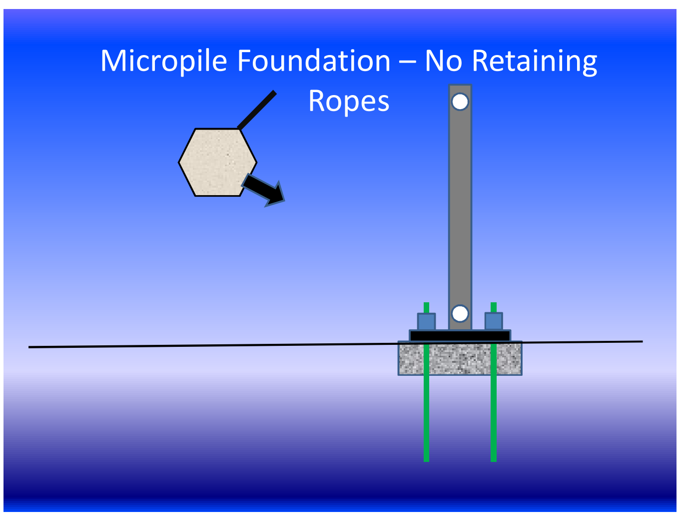
Figures 9a and 9b. Shallow Foundation - Top and bottom retaining ropes. System subjected to 67 kJ. (Note - concrete base broken but system still functional).

Grouted Bar Foundation System

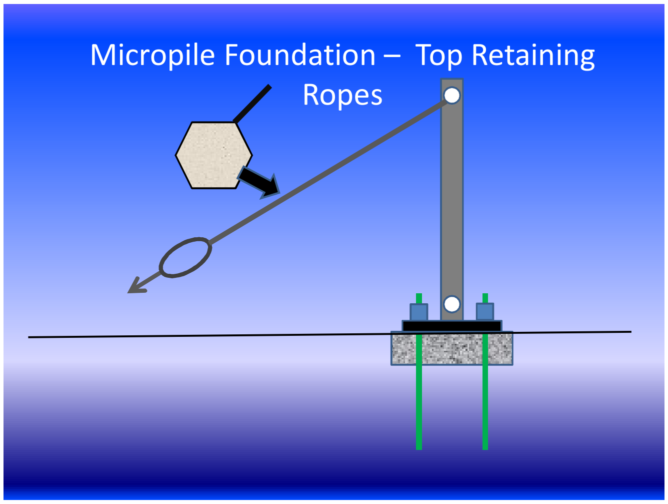
The grouted bar foundation system consisted of a W8x48 post welded to a 20 inch x 20 inch x 1 inch (51 cm x 51 cm x 2.5 cm) base plate that was attached to a 36 inch (0.9 m) diameter concrete pad that was cast 6 inches (15.2 cm) into the subsurface. Four all-thread number 8 bars were drilled and grouted a minimum of 5 feet (1.5 m) into the ground to provide a connection to the post. Load cells were placed on top of the base plate and tightened. Figures 10a, 10b, 11a, 11b, 12a and 12b depict the system.

Observed Results

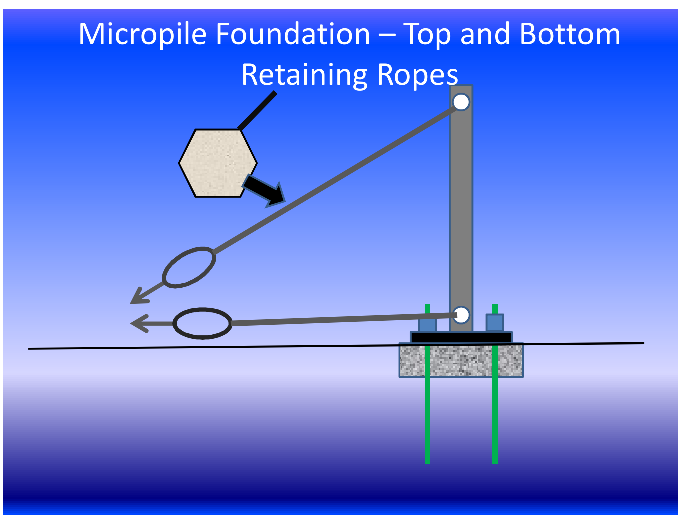
Overall, the top retaining rope was necessary to maintain an effective system. It appeared that the lower retaining rope was effective up to 116 kJ, but the post began to bend with or without the lower retaining rope indicating the weakest part of the system was the post and not the grouted bar foundation. Testing was stopped at 116 kJ since the 8x48 post had been compromised in this foundation scenario. The bottom retaining anchor was less effective with the deeper grouted bar system.



Figures 10a and 10b. Grout Bar Foundation – No retaining ropes. System subjected to a 20 kJ impact. (Note – post deflected without retaining ropes).



Figures 11a and 11b. Grouted Bar Foundation – Top retaining ropes only. System subjected to a 116 kJ impact. (Note – post began to deflect with only top retaining rope).



Figures 12a and 12b. Grouted Bar Foundation – Top and bottom retaining ropes. System subjected to repeated 112 kJ impacts. (Note – lower retaining ropes less engaged as load went directly to foundation).

Shaft Foundation System

The shaft foundation system consisted of a W10x60 post welded to a 24 inch x 24 inch x 1 inch (61 cm x 61 cm x 2.5 cm) base plate that was attached to a 36 inch (0.9 m) diameter concrete shaft that was cast 6 feet (1.8 m) and 3 feet (0.9 m) into the subsurface. Four all-thread number 8 bars were cast in the concrete to provide a connection to the post. Load cells were placed on top of the base plate and tightened. Figures 13a, 13b, 14a, and 14b depict the system.

Observed Results

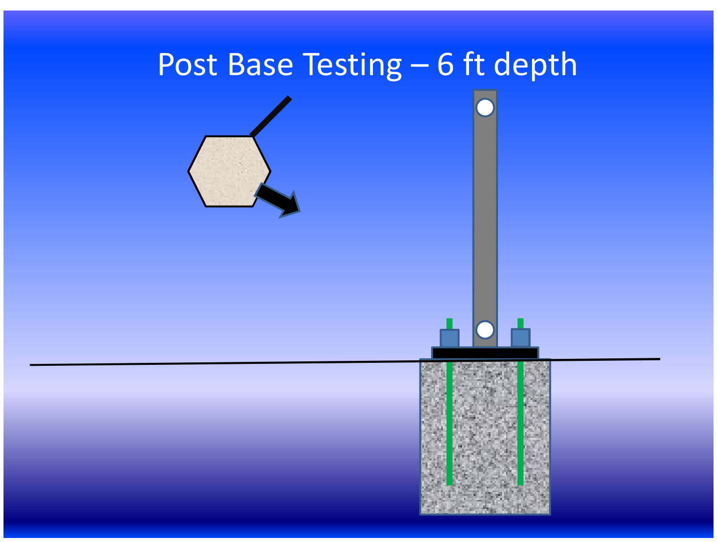
No retaining ropes were used in the sequence of testing of the two foundation depths to fully evaluate the foundation types.

The notable results from the testing of these foundation systems were:

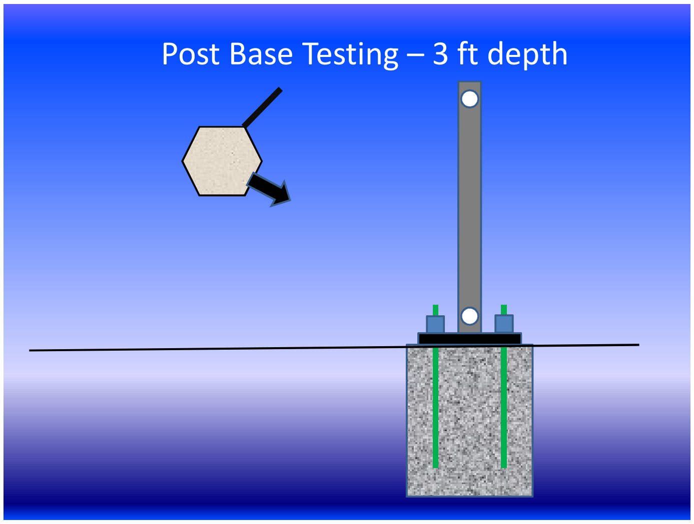
1. Review of the 6 foot (1.8 m) embedment system indicated that the 10x60 post and plate system sheared prior to significant movement of the concrete foundation at a relatively low 68 kJ impact.

2. Review of the 3 foot (0.9 m) embedment system indicated that the entire base rotated out of the subsurface without retaining ropes at an impact of 20 kJ.

Overall, the testing indicated that a 6 foot (1.8 m) embedment depth without retaining ropes would far surpass the moment capacity of most rockfall type post systems. Additionally, the results from this testing and previous testing indicated that the 3 foot (0.9 m) embedment depth would provide satisfactory results with the use of top only retaining ropes.



Figures 13a and 13b. Shaft Foundation, 6 foot depth. No retaining ropes. System subjected to 68 kJ impact. (Note - the 10x60 post and plate sheared prior to significant movement of the concrete).



Figures 14a and 14b. Shaft Foundation 3 foot depth. No retaining ropes. System subjected to 20 kJ impact. (Note - the 3 foot deep embedment rotated out of the ground without the aid of a top retaining rope to restrain it).

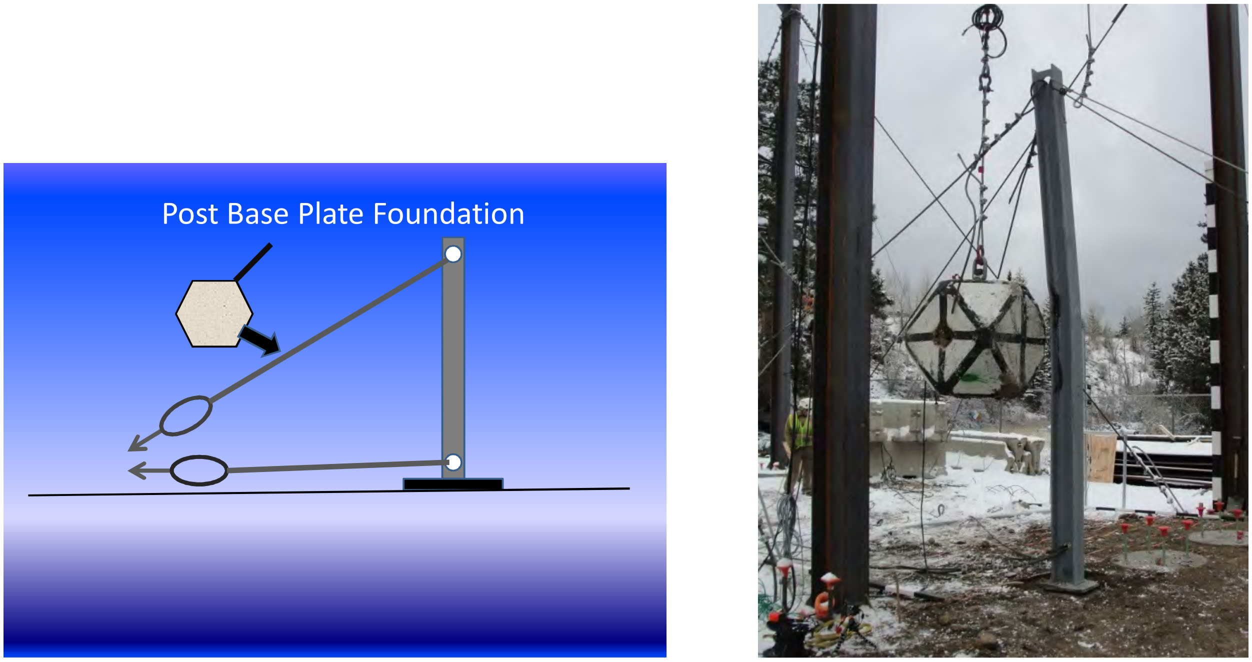
Plate Foundation System

Based on the previous test results, the following two post testing scenarios were developed and performed using a simple plate foundation placed on the ground surface. The first test system consisted of a W8x48 post welded to a 20 inch x 20 inch x 1 inch (51 cm x 51 cm x 2.5 cm) base plate that was placed on the ground surface (Figure 15a). No other foundation element was present; however both top and bottom retaining ropes were employed.

The other test consisted of a W8x48 post welded to a circular 18 inch diameter by 1 inch (46 cm x 2.5 cm) thick plate that was placed on an approximate 1H:1V slope to approximate field conditions on a slope (Figure 15b). This test also had top and bottom retaining ropes.

Observed Results

The notable results from the testing of these foundations is that the first scenario was able to withstand repeated 96 kJ impacts with minor damage to the post system, and the second scenario was able to withstand multiple 218 kJ impacts with minor bending to the post. Both systems were still functional at keeping the rockfall system functioning after repeated impacts at much higher energies than anticipated.



Figures 15a and 15b. Base Plate Foundation – Top and bottom retaining ropes. System subjected to 96 kJ impact. (Note – minor post bending after repeated impacts)

Figures 16a, 16b, and 16c. Base Plate Foundation with top and bottom retaining ropes. Figure 16b shows the side of the test just prior to the impact. Figure 16c shows the back of the post after a 218 kJ impact.

RESULTS AND CONCLUSIONS

Overall the testing of the post foundation systems provided useful results to CDOT in the design of the rockfall post systems for direct impacts, multiple impacts, and management of the systems. The following conclusions were drawn from the post testing conducted and are divided into rigid and flexible foundation systems:

Rigid Foundation Systems (Grouted Bar and Shaft)

- The testing indicated that a 6 foot deep (1.8 m), 36 inch diameter (0.9 m) foundation system without top or bottom retaining ropes greatly exceeded the moment capacities of the connection system of the W10x60 post to plate system. Stiffening the connection system would provide an incremental increase the capacity, but would produce an even stiffer system.
- Recorded loads on the bars and base plate for the 6 foot deep (1.8 m) shaft system exceeded 60 kips (267 kN) at relatively low 20 kJ impacts.
- The overall stiffness of the shaft systems reduced the effectiveness of the post foundation at absorbing rockfall impacts.

Flexible Foundation Systems (Shallow and Base Plate Only)

- The use of top and bottom retaining ropes had significant contributions to the impact absorbing performance of the post systems. If both top and bottom retaining wire ropes were employed then the requirement for a deep foundation system was effectively reduced or eliminated.
- By eliminating the foundation and using top and bottom retaining anchors only, it was possible to double the impact absorbing capacity of the post system up to 217 kJ.
- Based on the results of the instrumentation and performance of the wire retaining ropes, the load on the retaining ropes recorded for the plate foundation system did not exceed 50 kips (222 kN) during maximum energies of 217 kJ.
- Flexibility of the shallow and base plate foundation system was the key to increasing the impact performance of the system.

Combination Foundation Systems

- A grouted bar or shaft foundation only required top retaining anchors since the foundation did not deflect laterally enough to engage the bottom retaining anchors. Bottom retaining anchors could be used for additional redundancy, but would not engage until the foundation deflected significantly at a point where the post would probably be compromised.
- A grouted bar or 3 foot (0.9 m) deep shaft foundation would perform adequately for the test loads up to 116 kJ, however, since the post foundation was restrained the post began to bend after repeated impacts.

Overall, based on the results of the testing, the post systems should be designed as a balanced system meaning that the post system is only as strong as the weakest link. Foundations greater than six feet in depth would exceed the requirements of most post base systems since the stiffness generates much higher loading conditions. Alternatively, the flexible systems such as the base plate with top and bottom retaining ropes provided the greatest impact capacity for a given post. It should be noted that the base plate system may present unique construction challenges on a remote slope since the post is not set on a foundation and would need to be braced until the retaining anchors are attached on all sides. Overall knowledge of the performance characteristics of the various foundations provides insight into managing the components of the system by knowing what to expect from the performance during an impact.

Figure 17 reflects a generalized concept for designing post foundations based on the results of the testing in Colorado. Darker shading within the triangle depicts more impact capacity of the post system relative to the other systems. It should be noted that constructability issues and site constraints will also need to be evaluated in the design phase.

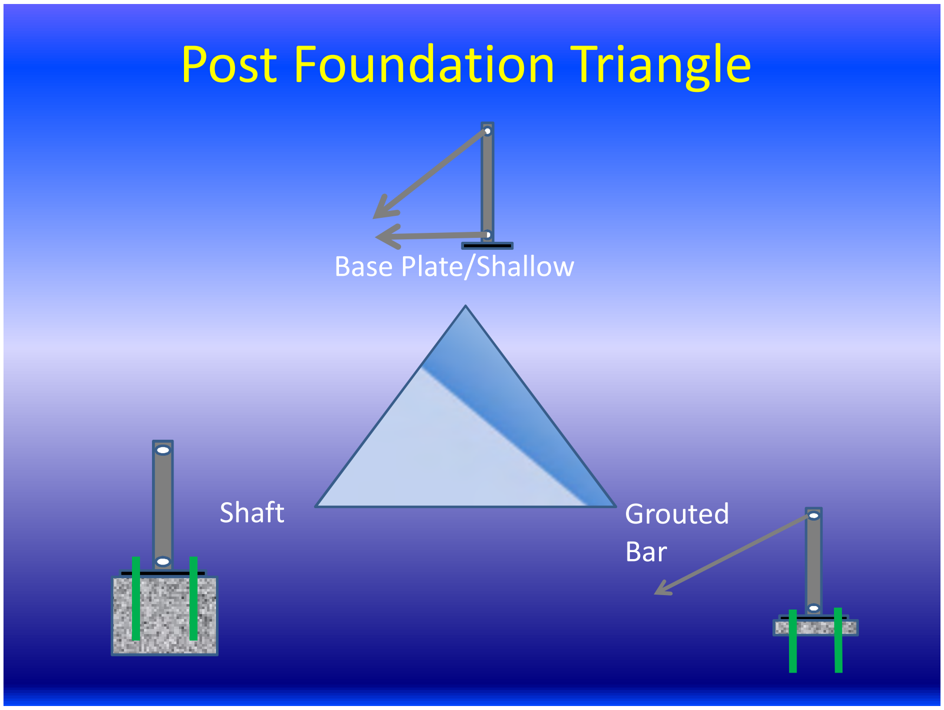


Figure 17. Design triangle for post foundation design. Darker shading within the triangle depicts more impact capacity of the post system.

64th Highway Geology Symposium

Development of a Modular Brake Element for the use in Modern Rockfall Catchment Fences

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Sebastian Mientki – former Pfeifer Isofer

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ABSTRACT

Rockfall catchment fences have a long history, with their beginnings being rooted in rigid structures. Building upon experience, mitigation structures became increasingly more flexible. Eventually the modern rockfall catchment fence was born, consisting of steel posts, continuous bearing ropes that support a flexible net structure and brake elements.

Early brake elements primarily functioned by absorbing energy during an impact through friction. Support cables were lead through steel plates with several holes drilled in them. Another example of early brake elements utilized the deformation of steel to absorb energy by leading support cables through steel tubes in the shape of a ring. As a force was exerted on the cable, it tightened the ring, ultimately pulling it into a knot. A further advancement removed the support ropes from the brake element entirely and relied on the deformation of steel, for example a coil of steel that uncoils as one end is held in position and the opposite is pulled or a strip of steel that is forced through a roller to make a bend at a defined angle (e.g. 180 degrees).

The authors will present a new type of brake element that further relies on the properties of steel to absorb energy, but instead of focusing on friction or the deformation of a profile, it harnesses the controlled failure of steel.

INTRODUCTION

With the increasing vulnerability of modern infrastructure, traffic ways and rural areas, dynamic catchment fences progressively developed to become the most economical mitigation measures for many cases of rockfall. The evolution started by improving rigid line-type wooden or steel structures with a mesh or wire rope net overlay. Shortly thereafter, the pioneers of these constructions, such as engineers, forestry and highway maintenance as well as component suppliers, realized that fence structures could be improved by adding active energy dissipation devices to the bearing and retaining ropes.

Early energy dissipation devices, so called brake elements, used in rockfall protection kits consisted of wire rope loops assembled by rope clips. The impact energy of a rockfall was dissipated by the friction between moving ropes. These friction based brake elements were enhanced for instance by using perforation plates to become technical devices with repeatable force progressions. Friction based and friction-deformation combined systems could be adjusted in both, brake distance and trigger force. They are established in various barrier systems and cover a wide range of energy classes.

More recently, system manufacturers focused on deformation type brake elements. Commonly the deformation of steel springs, rods or profiles is used to dissipate energy. This type of brake element shows better performance and efficiency than the earlier friction based brake elements. The brake length and the activation force can be adjusted by the selection of the material used. Most of these systems are large in size and some may be vulnerable to blockage from external debris.

The R&D team of Pfeifer Isofer have invented a product solution that combines deformation and brittle braking behavior of steel to an ultra compact modular brake element. The characteristic response curves are adjusted to optimize the interaction of the system components within in a modern rockfall catchment fence.

The emphasis of this paper is on the development and optimization process of the Isofer modular brake element. The path chosen to optimize the initial concept to a fully developed serial product is shown. Finally the test series that led to the ETAG 027 certification for the 250 kJ barriers is described.

RESEARCH AND DEVELOPMENT

Idea and Principle

The development of a new brake element is a process driven by a combination of a need for the improvement of existing products and the limitations set by existing patents. Therefore a geo-bionical approach was chosen to invent a product relying on natural structures and the properties of goods known from daily use. The desired characteristics for a new brake element were that it should be compact, easy to install, efficient to produce, have the ability to be placed in series and parallel within the system, be relatively unaffected by external conditions (e.g. weather, debris, vegetation, etc.) and above all dissipate energy efficiently. The end product is the Isodisk.

The primary component of the new brake is in allusion to the left-handed spiral of a meteorological storm system. It consists of a flat steel disc with a spiral perforation path (akin to that of a tear-up edge of a writing pad) that spirals from the center of the disc to the outer edge (Figure 1). The path consists of a line of drilled holes in the disc (Figure 8). The center, or core, is shaped like a Chinese yin-yang sign, which turned out to be the most efficient configuration. When a force is applied to the disc, it tears along the perforated path thus causing two arms of the disc to form, extending in opposite directions. In doing so, energy is dissipated. Once fully activated, the disc becomes a single strip of metal.

With this basic disc, modular units can be assembled depending on the energy dissipation requirements of the system. Individual discs can be stacked to achieve higher activation energies while their connection in series allows for more dissipation at a set level.

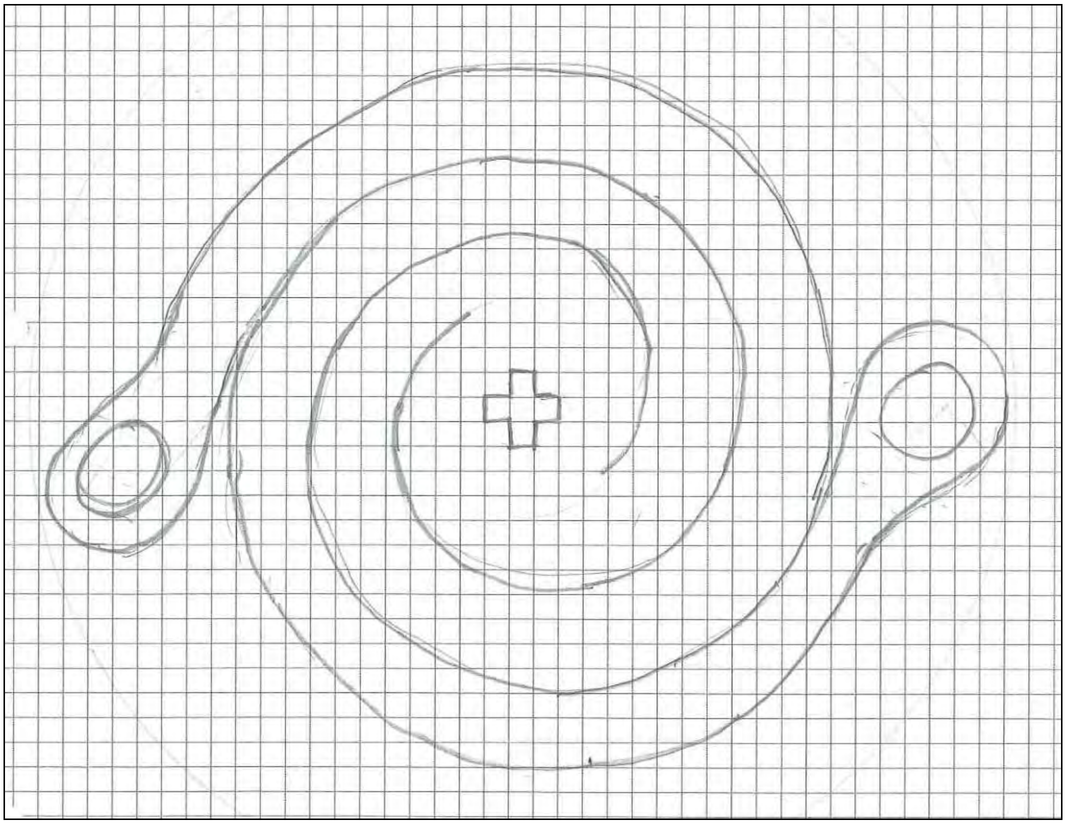

Figure 1 – First sketch of the principal Isodisk idea

Steps of the Developing Process

For the technical realization of a serial production brake element, the development process was split into five defined steps. A timetable was established that had distinct milestones and checks for optimization of the different aspects of the brake. Most of these processes took place simultaneously and were interactive with other processes. With this clearly structured progression, the main development process was realized within five months.

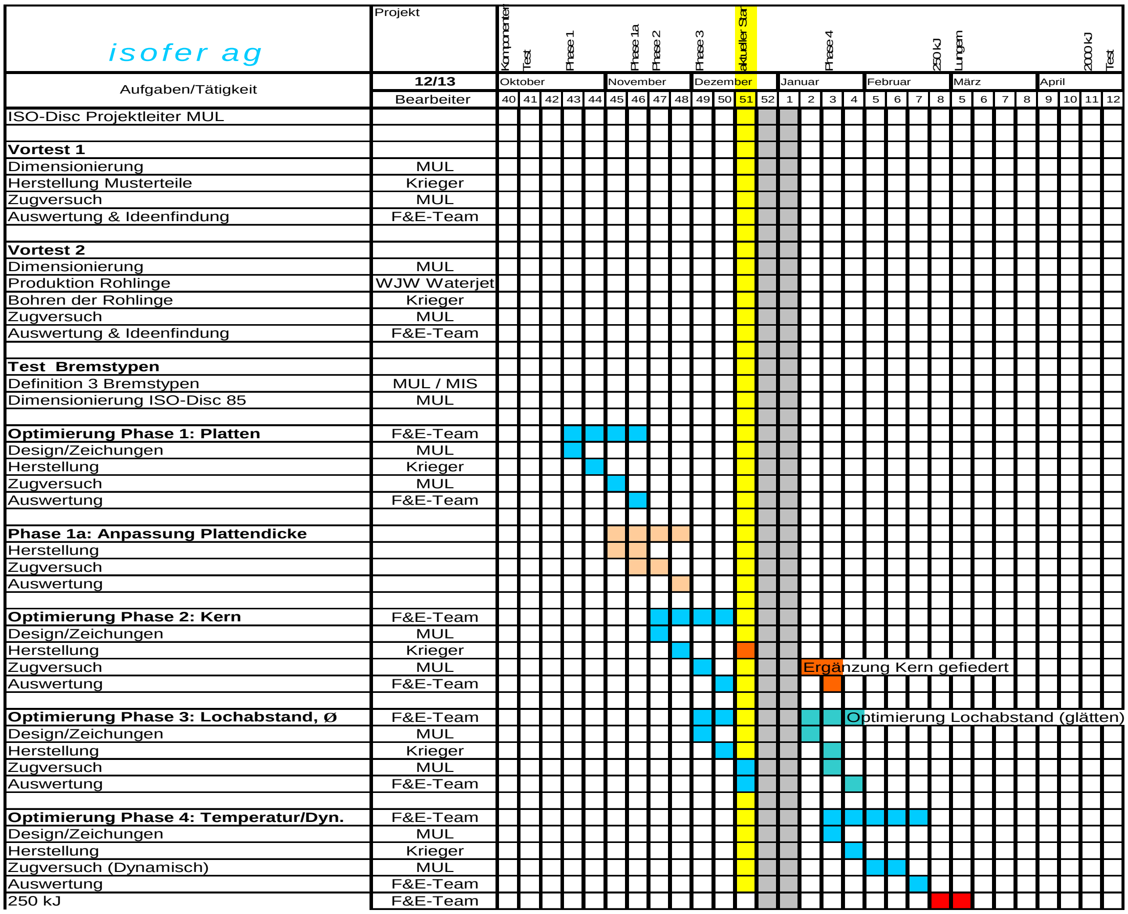

Table 1 - Project plan for the optimization process of the disc

Feasibility and Patent Check

The first stage of the development process concerned the feasibility of production and the behavior of the disc in a tension test. Focus was given to two different types of discs. One showed two cuts, continuing from the levers to the disc core (Figure 2a). The second showed two pre-determined tearing paths, with a sequence of material bridges (Figure 2b).

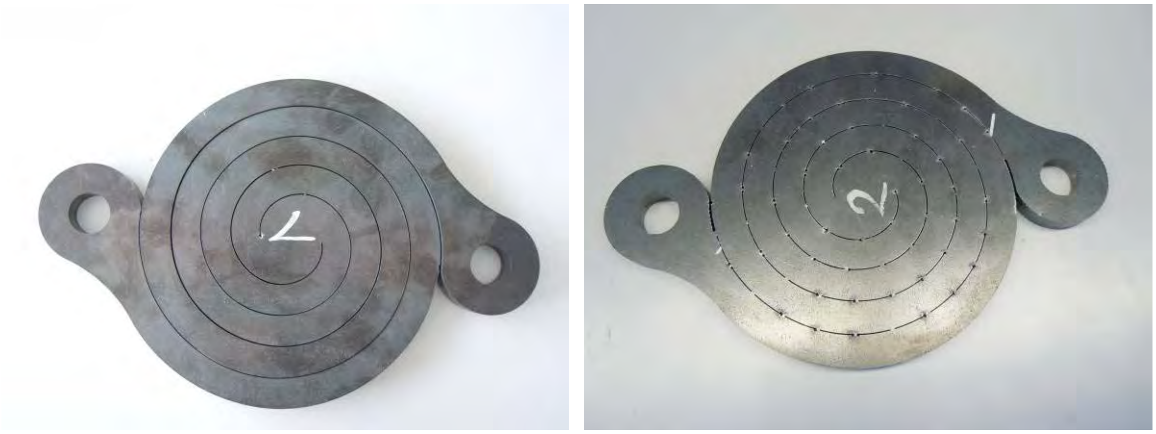

Figure 2 - Prototypes with: a) continuous cut and b) a perforated path

Using CAD generated STEP-data, the production of the prototypes with a plasma cutter was a standard procedure that implicated no limitations to the developing process or a future serial production.

The tension tests showed that the perforation, which was meant to increase the tensile strength of the disc, was necessary to guarantee its performance. The disc with continuous cuts started winding and failed after a limited extension (Figure 3a), while the perforated disc was ripped open the entire way to the core (Figure 3b).

Figure 3 – a) Tension test of disc with continuous cut and b) tension test of disc with perforated path

After the technical feasibility of the brake element was deemed possible, legal clarifications were initiated to search for possible patent conflicts. The disc turned out to be a completely new invention and a German utility-patent was formulated by a lawyer and submitted to the national patent office.

Numerical Modeling

To keep the development cost for optimal dimensioning of all influential parameters as low as possible, a partial feasibility study was carried out in cooperation with a specialist in dynamic FEM-simulation. For validation, it was necessary to compare the simulation results with the output of the first dynamic field-test. In this regard, certain material parameters and fundamental values were required that were not known at this stage.

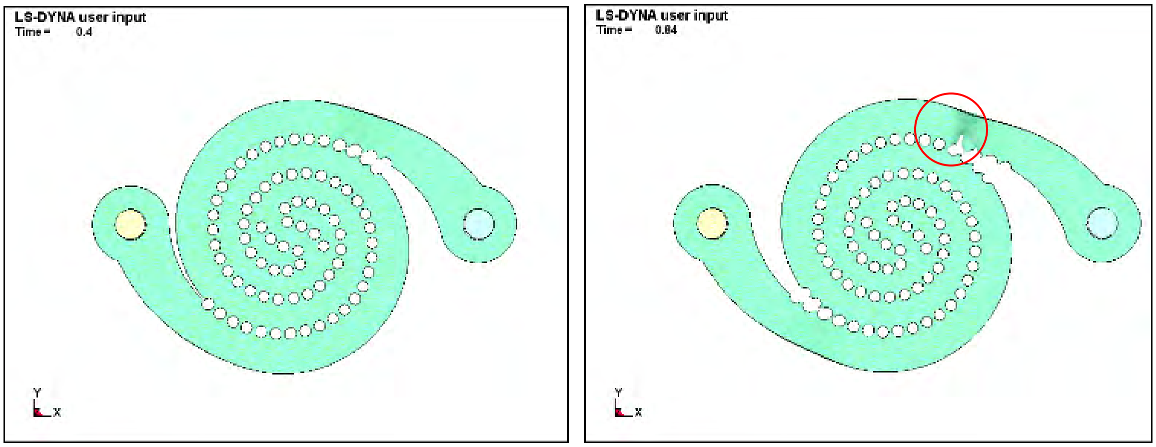

Figure 4 – FEM simulation of disc with force applied at: a) t = 0.4s and b) t=0.84s.

The result of the study was an incorrect calculation of the failure criteria of the FEM-modeling. Figure 5 shows a resulting strong saw-tooth shape of the tensile behavior of the disc while Figure 4 shows the FEM model results. After a relatively small energy absorption the notch effect was attained (Figure 4b), which resulted in failure of the brake. More reliable numerical modeling would have been possible after an extensive material study that was deemed not possible both from a cost and time perspective. As such, optimization of the disc was decided to be undertaken manually.

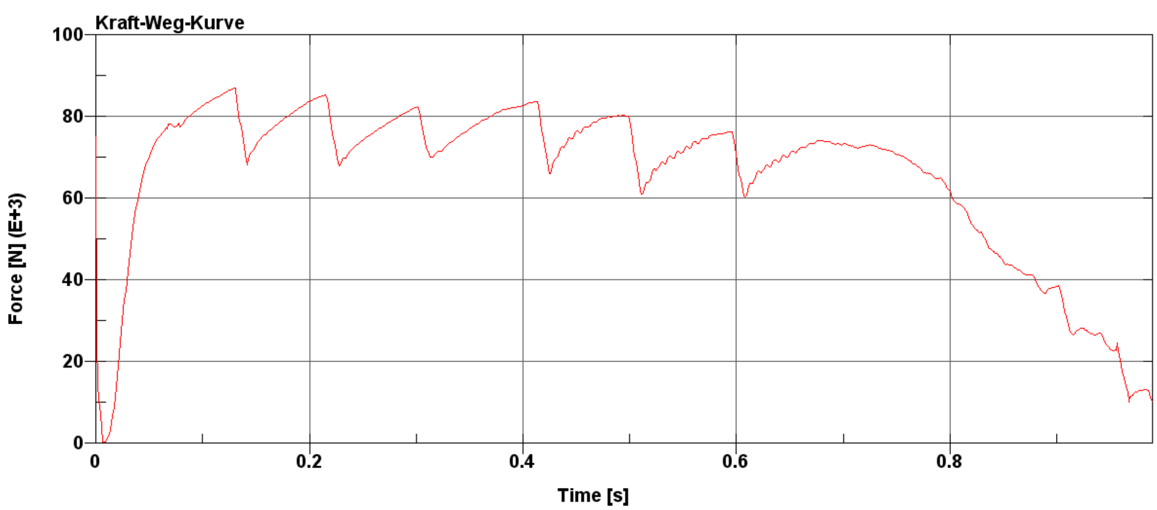

Figure 5 - Time-Force diagram of the FEM modeling

Static Tension Tests

The static testing improvement was focused on optimizing the brake`s material thickness, dimension of the material bridges and the design of the core.
For the implementation of the tests, there was used a horizontal tensile machine.

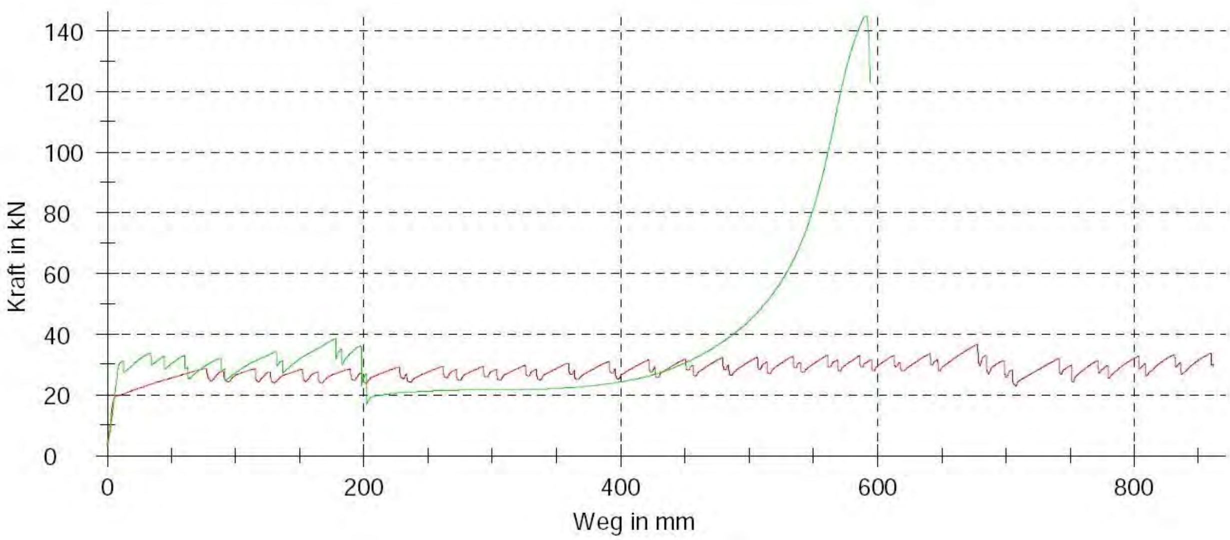

Figure 6 - Toughness diagram of the final prototype

The subsequent results were clear. If the material is thinner than 15 mm the unrolling disc will wind. If the predetermined breaking point is bigger than 3 mm, the variability of the force is too large and the whole system can start swinging. For the final step all variants of the core were theoretically assessed with a rating system. Then the four highest rated were produced and statically tested. The yin-yang core shape proved to be the most resistant to a failure load that was at least three times higher as the initial breaking load.

In the direct comparison of plasma cutting and drilling fabrication, a chipping technology showed to be more economical for the manufacturing process of the perforation. The laser cuts also have a negative effect on corrosion protection. Hot-dip galvanized zinc coating is limited by capillary action and cannot be deposited inside the narrow cuts.

The resistance force generated by a set of quasi-static tensile tests (rate: 100 mm/min), shows a small variation of less then 5%. In comparison with dynamic tests at 20 m/s block impact speed, the static tests generated the same resistance force value.

Dynamic Component Tests

As indicated by the numerical modeling, some uncertainties about the initial bending of the disc levers remained. To gain further reliable information on the general function of the brake element, a barrier was installed at a test site in Lungern, Switzerland. The test site (Figure 7) consists of foundation and anchoring points for a 30 m fence and an inclined cableway that allows 1:1-ETAG certification tests of up to 2000 kJ. The acceleration path of the cableway is inclined by ~ 40° and has a length of 40 m.

Figure 7 - Isofer rockfall barrier test site in Lungern, Switzerland

For the test series conducted in October 2012, an non-symmetrical barrier with one brake element for the upper and lower bearing ropes at on side of the barrier was erected. This setup was chosen to prevent any interaction between the brake elements. The installed brake elements, consisting of a drilled tear path and a non optimized core structure, were 60 mm thick (Figure 8a). As an additional safety measure, the brakes where bridged by overload cables. A series of tests with 320, 800 and 1600 kg blocks was shot. The forces in the bearing ropes were measured by load cells on each side of the brake elements with a frequency of 1200 Hz. Two high speed cameras with a frequency of 250 fps observed the motion of the discs.

Figures 8– Photos showing the tested brake element a) before and b) after the dynamic test.

The general performance of the disc was proofed by so called service level tests (SEL) that put a 30% load on the same brake element twice (Figure 8b). The response to the second impact was equal to the initial reaction of the brake. The consumption of brake-way and the time-force diagram showed an unexpected positive performance that underlined the potential of the invention.

For the analysis of an overload event, a 1,600 kg block was shot into the reconstructed barrier. After the brake-way of both elements was completely consumed, one brake failed at its core. Even so, the block was captured by the system and performance expectations were exceeded.

Overall, the former initial impact force peak, known from friction brakes, was nearly exceeded and the energy absorption of the prototype was higher than assumed. It showed a stable, very high plateau level during the entire dissipation process (Figure 9). The test also emphasized the need to improve the disc's core to increase the fracture resistance of the brake element at the end of maximum brake elongation, which is the main topic of the following step in development.

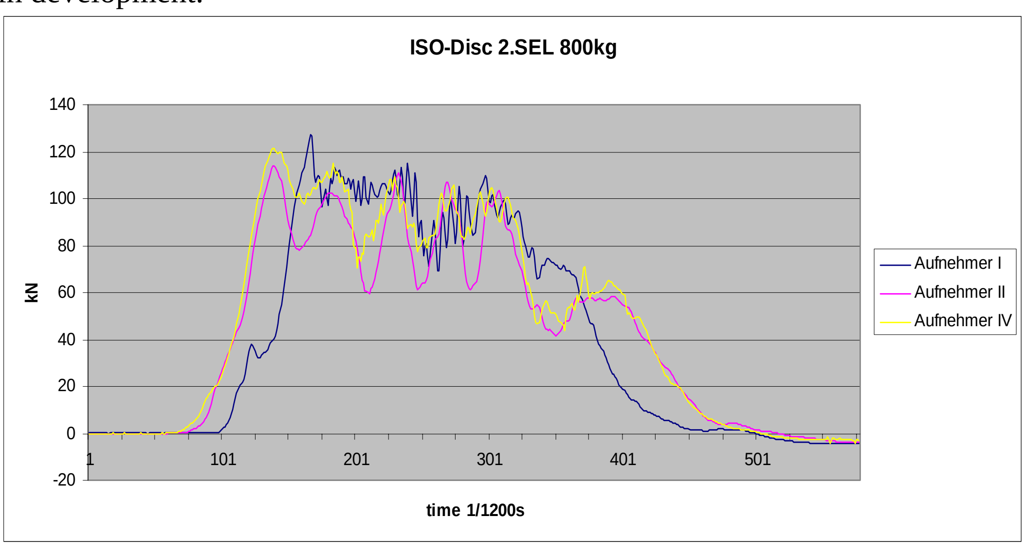

Figure 9 – Time-force graph for the dynamic test of the brake elements using an 800 kg projectile.

In a second dynamic test-series in February 2013, the focus was on the behavior of the disc-core and its interaction with the other system components. The same basic test set-up was chosen (Figure 10) and two different types of net were used. In addition, the test was run for the first time using a stack of discs (Figure 11) and the function of the brakes under below freezing conditions was observed.

Figure 10– Set up of the high speed camera for the second dynamic test

Figure 11 - Disc stack before (a) and after (b) the MEL impact

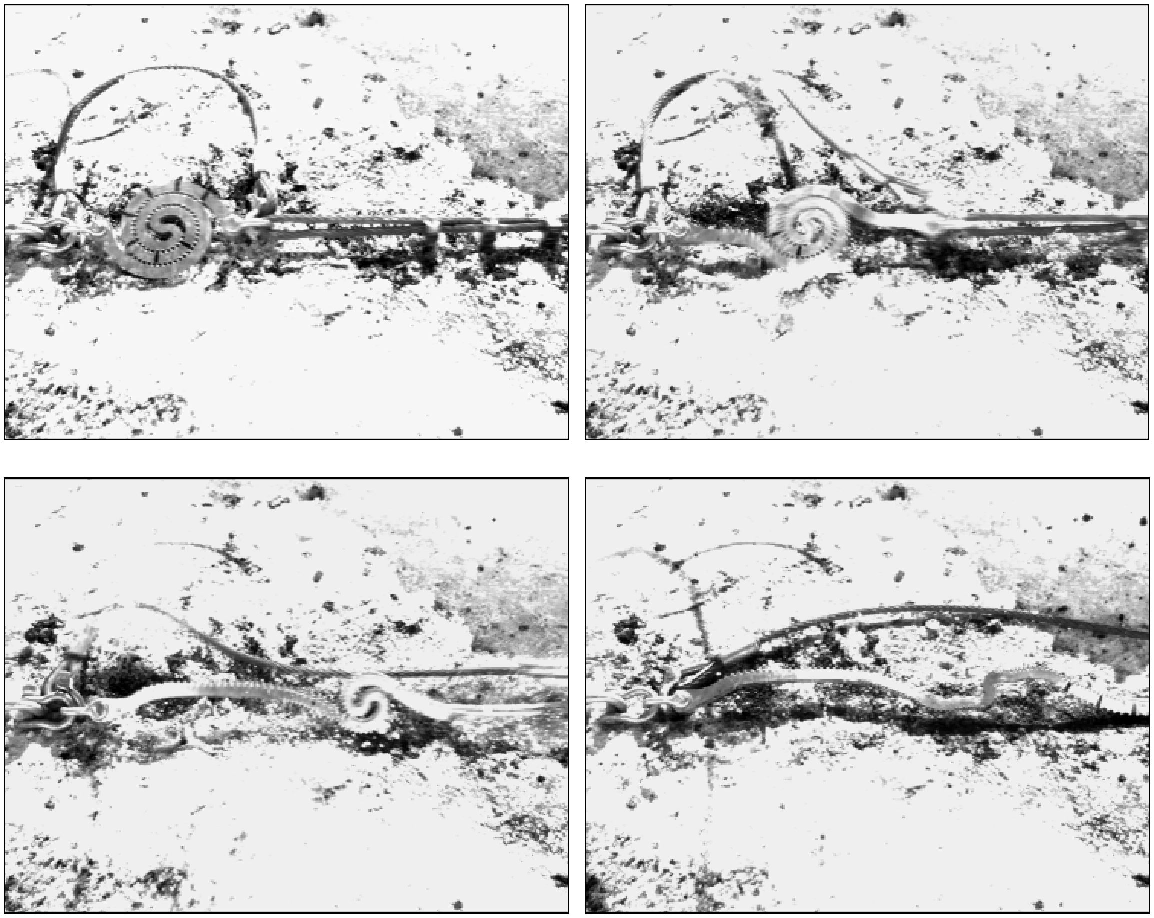

Figure 11 - Frame series of the unrolling disc and the core at maximum elongation

The output data of the test showed that in a double disc stack, both brakes are working synchronously and that the force level rises quasi-linearly with the number of discs used. Under dynamic loads, the optimized core caused the fatigue resistance to rise by at least 300%. Analysis of the ripping speed in comparison with the force on the bearing ropes and brakes illustrated the direct response of the brake to the dynamic forces applied to it. With the test series shot into the system using the same brakes, resistance against a series of impacts was demonstrated and the unrolling process started again when the applied load exceeded 50 kN (the same as for the initial impact). This final component test resulted in the ETAG certification test for the 250 kJ barrier system that is introduced later in this paper.

Economical Optimization

During technical optimization, the disc was standardized and production methods where improved. The approach of stacking discs turned out to be most economical with regard to production costs and quantities, transportation and inventory control. As a first step, an evaluation of the brake forces and the required brake-distances for the EOTA energy classes 1-6 was carried out. The numbers of discs derived from these evaluations are displayed in Table 2.

Energy Class	Brake Energy, total	Brake-distance, total	Resulting Number of Isodisk 25
0 (100 kJ)	75 kJ	1.5 m	4
1 (250 kJ)	190 kJ	2.8 m	8
2 (500 kJ)	375 kJ	4.2 m	12
3 (1000 kJ)	750 kJ	6.1 m	24
4 (1500 kJ)	1125 kJ	8.0 m	28
5 (2000 kJ)	1500 kJ	10.0 m	32
6 (3000 kJ)	2250 kJ	13.0 m	48

Table 2 - Calculated number of discs necessary for the main barrier classes

In cooperation with the company's steel fabricator, an optimum between production cost and performance was established. While the contour of the 60 mm thick second prototype was carved by a hydro jet cut, subsequent prototype discs where less thick and could be produced using a laser cutter. Additionally, to increase production speed the perforation was made using drilled holes instead of time consuming laser cut penetrations. The weight of a single disc was optimized to 6.4 kg.

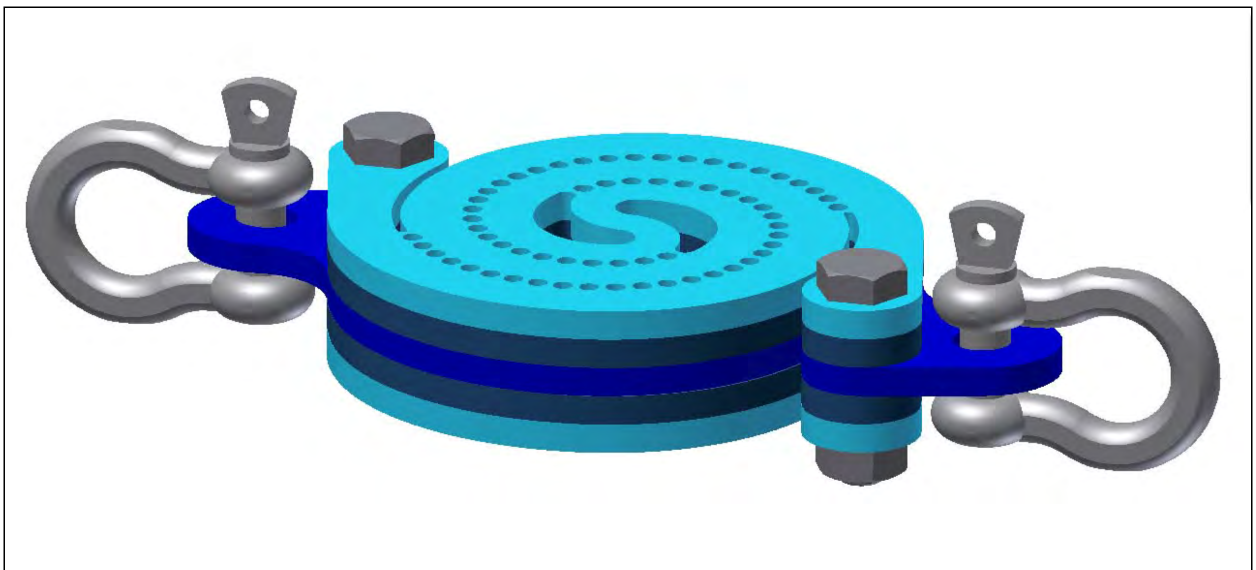

Figure 12 – Stack of discs with a center coupling disc

250 KJ ETAG 027 CERTIFICATION TEST

With completion of the final component test, the interaction of all system parts became the focus of final testing. A symmetrical barrier with a total of eight discs was installed at the test facility in Lungern. Other main components were the Flex Net and RHS hollow profile pillars.

With an MEL impact of an 800 kg block (Figure 13), all of the discs unrolled synchronously and showed similar elongations. The immediate response of the brake elements caused a soft deceleration of the block. The loads on the bearing ropes and retention ropes were limited to a maximum of 100 kN (Figure 14). All ETAG requirements for an A-Class certification were met.

Figure 13 – Impact of the 250 kJ MEL-Test

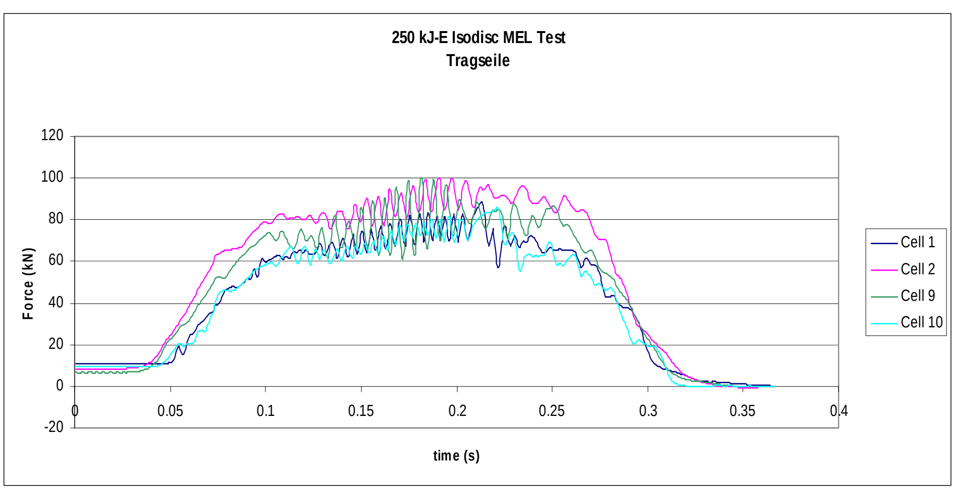

Figure 14 - Time-Force diagram of the MEL-Test

CONCLUSIONS

In a short time frame, it was possible to carry out a systematic process for inventing, testing, optimizing and establishing a cost efficient production process for a new brake element for rockfall catchment fences that utilizes a process that until now has not been implemented in a commercially available system; (i.e. controlled brittle failure of steel). All optimization steps in the process were reactive and where necessary, previously defined milestones initiated an additional optimization loop. The final disc shows clearly repeatable tensile results that vary by less than 1% and a fatigue resistance that is three times greater than the initial break force.

The innovative design of the Isodisk with stackable discs is economical because it optimizes production, transportation and inventory costs. This also reduces the ecological impact of the product.

The strong potential of the brake element, in interaction with the established Isostop barrier components, was verified by outstanding test results in the ETAG certification tests for the 250 kJ barriers. This new brake element shows promising benefits that present the core element for the future development of modern catchment fences and further natural hazard protection systems by Pfeifer Isofer.

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ONR 24810 – A Comprehensive Guideline for Building Better Rockfall Protection Structures

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ABSTRACT

The influence of the ETAG 27 Guideline for European Technical Approval of Falling Rock Protection Kits, published in 2008, has been relatively far reaching, including here in North America. ETAG 27 makes it possible to compare products, from different material suppliers, through standardized reporting of testing and material data. However, it does not consider best practices for the implementation or the evaluation of safety and maintenance requirements.

A new document published by Austrian Standards Institute – the Austrian national standards body, similar to ASTM and CSA – goes beyond ETAG 27, though in a much more broad spectrum including stabilization with anchoring and mesh/nets, embankments, and galleries. The document is entitled "ONR 24810, Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance", published in January of 2013.

Herein, the authors focus on summarizing the parts of the ONR specific to catchment fences beginning with the initial site investigation, which results in the input parameters for the numerical rockfall analysis. The semi-probabilistic verification of the design is then explained by the comparison of the impact parameters, such as energy and bounce height, with the resistance parameters of the catchment fence. Furthermore, helpful design and constructive rules regarding anchor design and fence layout are given. Lastly, maintenance and inspection schedules are presented.

INTRODUCTION

The publication of the ETAG 27 Guideline for European Technical Approval of Falling Rock Protection Kits in 2008 (*1*) was in response to the increasing use of flexible net catchment fences for mitigating rockfalls throughout Europe and the need for a unified standard. The document covers only the methodology by which systems are tested and how manufacturers must report material properties and system characteristics. It replaces national standards that had until then been enforced differently from country to country (e.g. *2*, *3*). Since similar national standards were not in existence in North America, the ETAG 27 guidelines have also become increasingly cited for projects that use rockfall catchment fences both in Canada and the USA.

A new tool for agencies, consultants and construction companies involved with rockfall mitigation was recently published by the Austrian Standard Institute, the Austrian national standards body similar to ASTM or CSA. This comprehensive document is entitled: "ONR 24810, Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance" (*4*). Unlike ETAG 27, it focuses not only on rockfall catchment fences but also on many other forms of mitigation including, stabilisation with anchoring and mesh/nets, embankments, and galleries. It does not cover system testing or material properties but instead concentrates on how mitigation structures are implemented, in particular the standardization of site investigation, design, construction and maintenance.

Only those sections of the ONR 24810 that pertain to rockfall catchment fences are discussed herein. The following themes will be summarized: Site Investigation, Semi-probabilistic Design, Anchor and Foundation Design, Constructive Rules, and Maintenance and Inspection.

Consequence Classes

A fundamental part of the ONR 24810 is its dependency upon consequence classes detailed in the European Norm EN 1990:2003 "Eurocode: Basis for structural design" (*5*). The consequence class is a qualitative rating in the case of failure of the system or component being classified with regards to the degree of loss of human life, and economic, social or environmental impacts. Three levels of consequence are defined as high, medium or low as per Table 1. They are arrived at by considering both the effects on the area of protection as well as the effects on the mitigation system's integrity which yield a global consequence class.

Table 1 - Consequence Classes

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, or economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g., a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g., an office building)
CC1	**Low** consequence for loss of human life, and economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g., storage buildings), greenhouses

In the ONR 24810, consequence classes are used to determine the required level of safety of components and characteristics of the planned mitigation structures, e.g. the factor of safety applied to forces used during design, a geometric coefficient applied to bounce heights, or the allowable opening of gaps in a fence after an idealized event. As the consequence level increases, so does the level of safety applied.

SITE INVESTIGATION

Site investigation requires both a desk and field investigation. The primary goal of the site investigation is to verify the hazard and collect information pertinent to the semi-probabilistic design parameters for the mitigation structures. The ONR 24810 explicitly notes that there should be no design of mitigation measures without conducting a thorough site investigation.

The desk investigation collects baseline information and identifies elements at risk and the areas of interest to protect prior to entering the field. It includes the review of historical data, databases, maps (e.g., topographical, geological, infrastructure, etc.) and other sources that help focus field investigations.

The field investigation is subdivided into three zones: initiation, transition and deposition. Each zone is investigated in an attempt to verify and expand information obtained during the desk investigation. Some examples of information collected for each zone are:

Initiation zone
Rock mass characterization, joint and discontinuity patterns and analysis, failure mechanisms, etc.

Transition zone
Morphology, dampening buffers, evidence of frequency, bounce height indicators, etc.

Deposition zone
Site morphology, relief (relative to initiation zone), identification of debris from previous events, evidence of frequency, bounce height indicators, accessibility (in particular for construction and maintenance), location of elements at risk, etc.

Using the information obtained, some preliminary analysis of the data is carried out in order to meet the goal of the site investigation, i.e. block size distribution, event frequency distribution, and bounce height distribution. Homogeneous areas are identified and a pre-selection of locations for mitigation measures are defined.

SEMI-PROBABILISTIC DESIGN PARAMETERS

After obtaining the necessary data, a series of steps are undertaken to perform a semi-probabilistic design of the catchment fence.

Design Block Selection

The selection of the design block is made in one of two ways: a simplified approach or standard approach. The simplified approach is used in the case that at least one of the following applies:

- Less than 100 blocks present in the deposition zone
- Less than 100 jointed rock bodies present in initiation zone
- Consequence class defined as CC1
- Event frequency falls under EF1 or EF2 (Table 2)

In this case, an expert can define the block based on their experience and information obtained during the site investigation.

Table 2 - Event Frequency

Event Frequency Class	Event Frequency n	Fractile for Design Block Size
EF 4 (very high)	$n \geq 10$ (≥ 10 events per year)	V_{98}
EF 3 (high)	$1 \leq n < 10$ (1 to 10 events per year)	V_{97}
EF 2 (low)	$0.03 \leq n < 1$ (1 event per year to 1 per 30 years)	V_{96}
EF 1 (rare)	$n < 0.03$ (< 1 event per 30 years)	V_{95}

In contrast, if none of the criteria for the simplified approach apply, then a standard approach to the block size design is required. This implies that the design block is the 98th fractile of the block size distribution recorded during the site investigation when the frequency class is rated as very high, or the design block is the 97th fractile in the case of a high frequency (as per Table 2).

Modelling of Energy and Bounce Height

State-of-the-art modelling techniques for trajectory analysis are employed using the data obtained from the site investigation and the design block. The results are verified with the site data to ascertain the realism of the model. The distributions of the modeled energy and bounce heights at the pre-selected location for the mitigation structures are reported and used for the verification of the mitigation design.

VERIFICATION OF DESIGN PARAMETERS FOR CATCHMENT FENCE

The basis for the verification that a particular catchment fence is an appropriate mitigation measure for a site follows the basic principle that the design values for the event are less than or equal to the design values of the resistance of the structure (i.e., $E_d \leq R_d$). Keeping to this, the verification of the energy capacity and bounce height are carried out independently. In addition, special performance criteria can also be implemented.

Energy

The verification of the energy capacity of a structure is carried out by comparing the design impact energy ($T_{E,d}$) to the resistance capacity of the structure ($T_{R,d}$). The design impact energy is given as Equation 1 and is equal to the 99th fractile of the energy distribution obtained for the location of interest ($T_{E,k}$) with a partial factor of safety ($\gamma_{E,kin}$), that is defined by the consequence class as shown in Table 3.

$$T_{E,d} = T_{E,k} \cdot \gamma_{E,kin}$$

Equation 1

Table 3 - Partial Safety Factor for Impact Energy			
	CC1	**CC2**	**CC3**
$\gamma_{E,kin}$	1.00	1.05	1.15

The resistance capacity is defined by the Maximum Energy Level (MEL) reported for a system by the manufacturer as per ETAG 27 ($T_{k, MEL}$) with a reduction factor ($\gamma_{T,R}$) applied as in Equation 2. The reduction factor is dependent on the consequence class given in Table 4.

$$T_{R,d} = T_{k,MEL} / \gamma_{T,R}$$

Equation 2

Table 4 - Partial Safety Factor for Resistance Energy			
	CC1	**CC2**	**CC3**
$\gamma_{T,R}$	1.00	1.05	1.15

The suitability of a system with regards to energy requirements is verified when Equation 3 holds true, i.e. the design impact energy is less than or equal to the resistance capacity. If the statement is false, a system with a higher capacity must be considered.

$$T_{E,d} \leq T_{R,d}$$

Equation 3

Bounce Height

The verification of the bounce height requirement is made by comparing the design bounce height ($h_{E,d}$) with the resistance height of the structure ($h_{R,d}$). The design bounce height is defined in Equation 4 as the 95th fractile of the bounce height distribution ($h_{E,k}$), taken at the upper surface of the block (i.e., **a half block height must be added**), for the location of interest with a geometric coefficient (α_1) applied that is given by the consequence class found in Table 5.

$$h_{E,d} = h_{E,k} \cdot \alpha_1$$

Equation 4

Table 5 - Coefficient of Bounce Height			
	CC1	**CC2**	**CC3**
α_1	1.05	1.10	1.30

The design bounce height is then compared to the available nominal heights of the system identified as a plausible system during the energy verification. Available nominal heights are governed by ETAG 27 and are based on the height of the system as tested, whereby:

1. The system cannot be manufactured below the tested height.
2. The system height can only be increased by 0.5 m if tested with a nominal height below 4 m.
3. The system height can only be increased by 1.0 m if tested with a nominal height greater or equal to 4 m.

The resistance height of the system is calculated in Equation 5, where the allowable nominal height of the system according to ETAG 27 ($h_{R,k}$) is reduced by a reduction coefficient (α_2) according to the consequence class in Table 6.

$$h_{R,d} = h_{R,k} / \alpha_2$$

Equation 5

Table 6 - Coefficient of Structure Height			
	CC1	**CC2**	**CC3**
α_2	1.00	1.05	1.10

The verification of the system with respect to height is then validated if the design bounce height is less than or equal to the resistance height as per Equation 6.

$$h_{E,d} \leq h_{R,d}$$

Equation 6

Performance Criteria

A last set of criteria is defined related to the effects of the MEL impact on the catchment fence. Where, for example, the residual height of a fence is reported and classified under the ETAG 27, the opening of gaps in the net near post locations are only reported but not evaluated. Gap openings such as this are a common occurrence in systems as elasticity of the net is limited in this area. These openings can allow subsequent material to pass through the system, and indicate a general elastic behaviour of the system. As such, the amount of allowable opening is defined according to the consequence class as indicated in Table 7. Other criteria under this category include what components are allowed to fail/rupture. This extends beyond ETAG 27 where elements such as nets, ropes or strands within ropes are allowed to fail, though they must be reported.

Table 7 - Optional Requirements for Rockfall Catchment Fences

Consequences Class	**Unacceptable Damages During an MEL Test**
CC3	- No opening of nets greater than or equal to **0.2 m** below the residual height, between the lower bearing rope and net. - No openings between the end posts and the net greater than or equal to 10% of the nominal height if the end fields are located within the hazardous area. - No rupture of the main nets, bearing ropes or retaining ropes **or the strands**. Single wires are allowed to break (as long as it is not through the entire strand). - A rupture of the sewing rope or component used to attach the primary net to the bearing ropes is **not allowed**.
CC2	- No opening of nets greater than or equal to **0.4 m** below the residual height, between the lower bearing rope and net. - No openings between the end posts and the net greater than or equal to 10% *of the nominal height if the end fields are located within the hazardous area. - No rupture of the main nets, bearing ropes or retaining ropes. - A rupture of the sewing rope or component used to attach the primary net to the bearing ropes is allowed if a new load bearing net border develops as non-positive connection to the bearing rope.
CC1	- No additional requirements. ETAG 27 certification sufficient.

* If the lateral openings are greater than or equal to 10% of the nominal height, the length of the line has to be extended by a half module length. If the end module lies outside of the hazardous area this condition can be neglected.

As an example, rockfall catchment fences tested that experienced either a rupture of the net or an opening of greater than 20 cm around the posts would receive full certification but would not be allowed to be used for projects having a high consequence class.

VERIFICATION OF ANCHOR AND FOUNDATION DESIGN

The design of foundation components is a somewhat contentious issue dependent on the project engineer's experience and local regulations. For our purposes, the design of anchor components is limited to rock and soil anchors that are bearing elements which apply both compression and tension forces into the ground and are hereafter referred to as micropiles.

Micropiles are further defined as having a borehole diameter of less than 300 mm and a reinforcement element (e.g., monobar anchor) diameter less than 150 mm. In Austria, the reinforcement element must have a national, i.e. Ministry of Traffic, Innovation and Technology (BMVIT), or a European, i.e. EOTA, approval. In addition, the following requirements apply when using micropiles:

- Minimum borehole diameter of 90 mm except in solid rock, with minimum 20 mm coverage of reinforcement element
- Minimum distance between micropiles is 1 m with the exception of base plate anchors
- Reinforcement element is centred in hole
- Minimum inclination 15 degrees from horizontal
- Injection begins from bottom of hole
- Micropiles that undergo primarily compression must use reinforcement tubes or concrete blocks or similar in the first 0.5 m for weathered or fractured rock or 1 m in soils
- The micropile is oriented to minimize shear loading on the anchor

As with the catchment fence, the verification is divided into two components: an effect side and resistance side.

On the effect side, the maximum force monitored during an ETAG 27 MEL test (E_k) is used for determining the design force (E_d). If multiple ropes are connected to a single anchor, then **the maximum forces from each rope are added in a scalar fashion**. A partial factor of safety (γ_E) equal to 1.5 is applied to this force (Equation 7).

$$E_{\mathrm{d}} = E_{\mathrm{k}} \cdot \gamma_{\mathrm{E}}$$

Equation 7

This method of adding forces is extremely important and often neglected resulting in under designed anchors. In many instances forces are added as vectors. If such a summation is used, then every anchor point must be considered individually with regards to the geometry of ropes and anchor positions. This is impractical, unrealistic and normally inefficient with regards to costs. If summed forces are given by manufacturers, they should clearly state how these forces were determined.

On the resistance side, two verifications are necessary: the cross section of the steel reinforcement element, and the verification of the surface between anchor grout and underground.

Verification of Steel Cross Section of Micropile

The resistance force ($R_{d,t}$) of the steel cross section of the micropile is determined by the product of the the cross section of the element and the characteristic yield strength divided by the product of a partial factor of safety ($\gamma_{s,t}$= 1.15, as per OENORM B 1997-1-1:2010 (*5*)), and a model parameter (η_{Mod} = 0.95) as shown in Equations 8 and 9.

$$R_{d,t} = R_{k,t} / (\gamma_{s,t} \cdot \eta_{Mod})$$ Equation 8

$$R_{k,t} = A_s \cdot f_{y,k}$$ Equation 9

Verification of Surface Between Anchor Grout Body and Underground

For the case that pre-production anchor pull tests are conducted, the characteristic value of pull out force ($R_{t;d}$) is defined by Equations 10 and 11. The value of pull out force is the lesser of the average pull out force ($(R_{t,m})_{mitt}$) divided by a distribution coefficient (ξ_1) or the minimum pull out force ($(R_{t,m})_{min}$) divided by a second distribution coefficient (ξ_2), where the distribution coefficients are defined based on the number of pretests as per Table 8. In both cases a partial factor of safety ($\gamma_{s;t}$) is applied.

$$R_{t;d} = R_{t;k} / \gamma_{s;t}$$ Equation 10

$$R_{t;k} = \min\left[(R_{t,m})_{mitt}/\xi_1, (R_{t,m})_{min}/\xi_2\right]$$ Equation 11

Table 8 - Distribution Coefficient Depending on Number of Pretests

n =	1	2	3	4	≥ 5
ξ_1	1.40	1.30	1.20	1.10	1.00
ξ_2	1.40	1.20	1.05	1.00	1.00

When no anchor pull tests are performed and values for the skin friction of the anchor grout surface are obtained from literature, then a model factor based on the consequence class is applied as per Table 9 in Equation 12.

$$R_{t;d} = R_{t;k} / (\eta_{P;t} \cdot \gamma_{s;t})$$ Equation 12

Table 9 - Modell Factors for Resistance of Foundation of Rockfall Catchment Fences

Resistance	Symbol	CC1	CC2	CC3
Micropile under axial pressure	$\eta_{p,c}$	1.25	1.25	1.30
Micropile under axial tension	$\eta_{p,t}$	1.25	1.25	2.50

The results of the verification are compared to available anchors and an appropriate selection and subsequent design is conducted.

CONSTRUCTIVE RULES

Some basic rules for the layout and construction of rockfall catchment fences are also defined by the ONR 24810. They are based on expert opinion and field experience, as described below.

Distance between catchment fence and object of protection
To ensure that the elements at risk are sufficiently far from the rockfall catchment fence, a factor of safety of 1.2 is applied to the maximum elongation distance as reported for the MEL test in the ETAG 27 documentation but where a minimum of the maximum elongation plus 1 m is observed.

Post spacing
It is not recommended to deviate from the approved tested post spacing for a system by more than ±2 m.

Row length without internal anchor
The length of a catchment fence without internal anchoring (i.e., directing the forces of the bearing ropes into the ground) shall not be more than 60 m.

End field placement
Since end fields are not tested for impacts, the last module should extend beyond the primary hazardous area. If the system has a tendency for the net to pull away from end post ≥10% of the residual height, then this is absolutely necessary.

Direct rock wall connection
There are two accepted scenarios for terminating a fence into a rock wall that differ in how the fence reacts to impacts in the end field, specifically the degree to which the net is pulled away from the wall. The accepted configurations are shown in Figures 2a and 2b.

Gully nets
Where gaps are present below the lower bearing rope due to undulating topography, the same net type must be used to fill the gap. No influence on the primary system is allowed (e.g., shortening of the elongation path, blocking of brake elements, etc.). Figures 3a and 3b show schematics of two potential solutions for gullies.

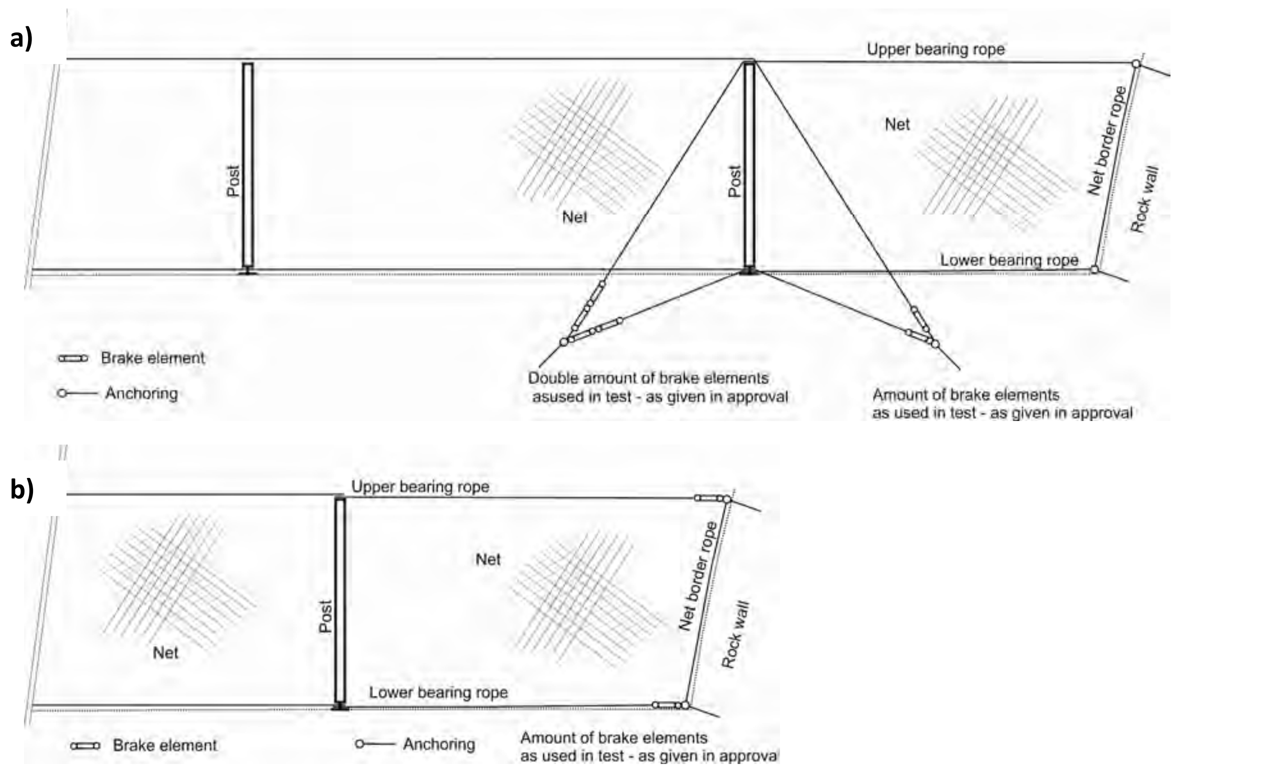

Figure 2 - Accepted solutions for the termination of a system into a rockwall where a) extra internal anchoring is used, and b) direct connection is used.

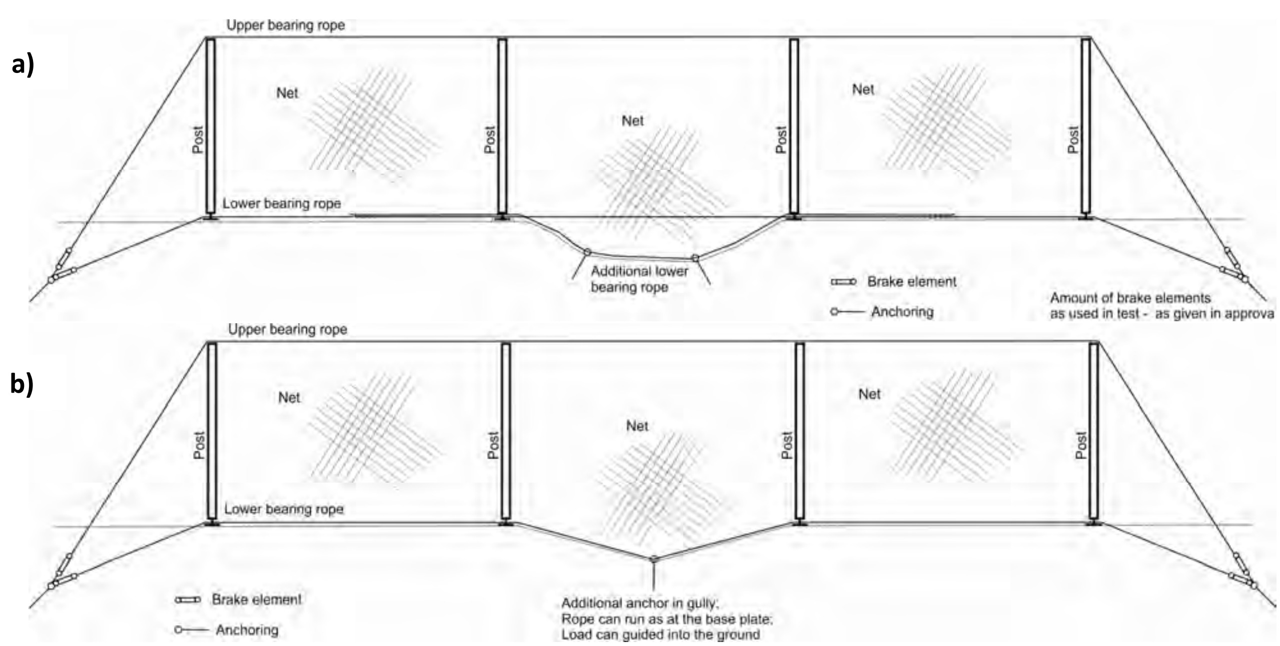

Figure 3: Solutions for gully nets where a) additional anchoring and an additional bearing rope are used, and b) additional anchoring with no additional bearing rope is used.

MAINTENANCE AND INSPECTION

Once a mitigation measure has been implemented, detailed documentation is required to establish a base-line of the structures. From this, the status of the system can be evaluated during future inspections in order to determine necessary maintenance.

The ONR 24810 covers the topic of maintenance and inspection in a general way that can be applied to all mitigation measures described in the document. The general methodology is laid out in Table 10 and consists of three primary inspection protocols: On-going inspection (LU-protocol), Control inspection (K-protocol) and Test inspection (P-protocol). A fourth type of inspection, Post-event inspection (SK-protocol) is a special case after an event has impacted the system. Examples of components of these protocols are limited to rockfall catchment fences herein.

Table 10 - Inspection types

Inspection type	Frequency	Responsability	Execution	Result
On-going inspection	yearly	obligated to maintain	by experts or trained personnel	LU-protocol
Control inspection	every 5, 7 or 10 years depending on consequence class	obligated to maintain	by experts	K-protocol
Test inspection	as needed	obligated to maintain	by experts or team of experts	P-protocol

LU-protocol
The on-going inspection is a yearly inspection conducted by experts or trained personnel. It includes checking brake functionality, elongation and residual capacity, net deformation and damage, damages to ropes, verification of nominal height, evaluation of debris in the system, etc.

K-protocol
The control inspection is conducted only by an expert on a schedule determined by the consequence class: every 10, 7 or 5 years according to a consequence class of low, medium and high, respectively. This protocol includes the LU-protocol but also evaluates possible corrosion of components such as brake elements, nets, ropes, posts and base plates, or any connecting elements. An evaluation of the foundation is also required where corrosion and deformation of micropiles are evaluated, as well as the state of erosion surrounding them along with the general condition of concrete foundations (e.g., evidence of cracking, spalling, flaking, corrosion of reinforcement elements if visible, etc.). Finally, a general evaluation of the state of the system compared to the most recent inspection report is conducted. Table 11 summarizes qualitative levels of system conditions with suggested actions and appropriate timeframes.

Table 11 - Qualitative Levels of System Status				
State Class	**Structural Safety**	**Fitness for Use**	**Time to Start Measure**	**Examples at Rockfall Catchment Fences**
1	given	given	long-term	no damage visible
2	given	given	long-term	minimal corrosion, minimal wear and tear
3	given	given	middle-term	plastic deformation of net, visible deformation brake element
4	limited	very limited	short-term	eroded or buckled micropiles, deformed posts, strongly deformed brake elements, decreased nominal height, rope ruptures, deformed shackles and wire rope clips, pulled micropiles, filled nets, broken welds
5	not given	not given		completely destroyed

SK-protocol

The post-event inspection is conducted by an expert and is in response to an event. It is independent from scheduled inspections and is used to determine the status of the system. It can result in the request for a test inspection.

P-protocol

This test inspection is conducted by an expert or possibly by an inter-disciplinary expert team. It is conducted on an as-needed basis when the status of a system or system component is identified by a previous inspection as indeterminable and which deems further, more detailed inspections are necessary. The nature of the test inspection will depend on the component(s) being inspected and may include more intrusive/involved test procedures to help determine the overall safety or state of the system (e.g., anchor pull tests).

SUMMARY

The ONORM 24810 describes a framework for the planning, implementation, construction and subsequent maintenance of rockfall mitigation measures. It includes methodology for the verification of suitability of a particular measure with respect to predicted event characteristics. Specifically regarding rockfall catchment fences, it draws on ETAG 27 documentation provided by catch fence manufacturers for the purpose of verifying that a particular system meets requirements determined during the site investigation and the engineering design. Constructive rules and maintenance routines are presented that help ensure a proper installation and the continued safe upkeep of the system.

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Rockfall Barrier Behavior Under Multiple Impact Events

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ABSTRACT

When a barrier is impacted by rocks, multiple components of the barrier are engaged to absorb the energy generated by the falling rocks. More often than not, rockfall events generate multiple rockfalls, that impact the barrier at different intervals. Rockfall barriers are generally very difficult to design considering that most information comes from few case histories, rigorous statistical analysis, and a knowledge of mechanical behavior of the barrier structure. With regard to the behavior of the barrier, the primary information available to the designer is provided by standard testing procedures like the ETAG 27, that defines a Maximum Energy Level (MEL) and Service Energy Level (SEL) capacity for the structure. The main question for the designer is: how does one correctly synthesize the data derived from geological and topographic surveys, the probabilistic analysis of the trajectories, and knowledge of barrier characteristics into the design?

This paper outlines some practical recommendations, that help overcome the main uncertainties affecting design reliability, to foresee and compensate for installation problems, and reduce maintenance costs. The goal is optimization of rockfall barrier designs considering their Service Energy Level (SEL) and Maximum Energy Level (MEL), as well as their behavior in cases where multiple impacts occur. The selection of a Rockfall Fence Kit, designed in accordance with full scale crash tests (ETAG 027), is recommended in order to understand and incorporate the values of loads and deformations acting on and through the fence kit during impacts.

KEYWORDS

Rockfall protections system, rockfall barrier, ETAG 027, new design approach, design, MEL, SEL.

INTRODUCTION

Rockfall barriers are generally designed and installed to catch rock masses that fall from a slope; and because their location is usually far from the rockfall source (detachment) area, they are classified as passive protection systems. These structures are widely used in different configurations depending on the impact energies they must withstand, their position on the slope, and the morphology of the slope.

The first systems adopted to protect mine sites, roads, and other infrastructure against rockfall phenomena were rudimentary wooden barriers. Afterwards, these wooden structures were replaced by rigid barriers composed of steel wire mesh, that began to be introduced in the market. The most significant progress took place in the early 1980s when the first probabilistic analysis simulation was introduced and the first full scale barrier tests were carried out in France, Swizerland and Italy. Nowadays, the concept of a deformable and dynamic barrier structure is at the base of the design of rockfall barriers, which are capable of absorbing energies up to 8,500 kJ (6.2E06 ft-lbf = 55900 lbs. at 55.6 miles/h).

In 2008, to ensure uniform performance and production quality of rockfall barrier structures, the European Organisation for Technical Approval (EOTA) issued a European Guideline ETAG 027 ("Guideline for European technical approval of falling rock protection kits"). Today this guideline has become the only test and construction framework used by manufacturers and it is also starting to be considered by designers who specify the performance of rockfall barriers. ETAG 027 (and the related European Technical Approval and the CE marking) represent a milestone for the rockfall barrier market, because it makes it possible to compare the performance of different fences, and it ensures the quality of the certified product. For these reasons, ETAG 027 presently constitutes the base for tenders around the world. Although rockfall barriers are in common use, often the designers do not have a clear overview of the technological limits of these structures, or why ETAG 027 performs the tests with a Service Energy Level (SEL) and a Maximum Energy Level (MEL). Unfortunately, ETAG 027 is a test guideline and not a design manual, and does not present information concerning the limitations of different fences or how to use barrier impact test results. In the background of this discussion there are several practical questions like: the effect of the site installation on a barrier's performance; the behavior of the barriers under extreme conditions; and the best use of the probabilistic rockfall simulation approach.

ROCKFALL PROTECTION KIT

The mentioned European Guideline defines a rockfall barrier as a kit composed of different elements that must be able to stop a block impacting against the installed kit.

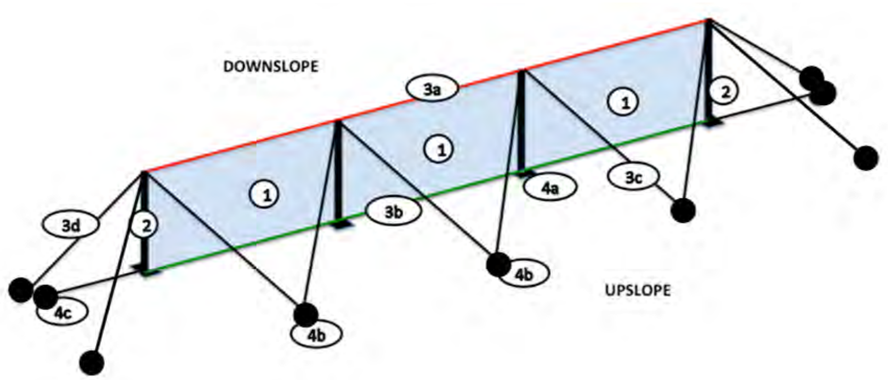

Figure 1: (1) functional module; (2) post; (3a) upper longitudinal cable; (3b) lower longitudinal cable; (3c) upslope bracing cable; (3d) lateral bracing cable; (4a) post foundation; (4b) upslope bracing cable foundation; (4c) lateral bracing cable foundation.

The kit is a sequence of functional modules (spans) composed of an interception structure (generally steel mesh) held by a support structure (generally steel posts) and a few connection components (cables and energy dissipater devices).

The barrier fence is anchored to the ground by foundations that transmit impact forces to the ground. Considering that barriers are installed on a wide variety of ground conditions, (loose soil, rock, etc.) the foundations are not part of the ETAG 027 guideline. Figure 1 illustrates a typical rockfall barrier configuration.

EUROPEAN GUIDE LINE ETAG 027

The ETAG 027 is the Guideline for European Technical Approval (E.T.A.) of falling rock protection kits. It defines the procedures for carrying out full-scale crash (impact) tests, and it is the strictest guideline in world on this subject. Moreover it establishes factory controls that manufacturers must follow regarding the materials used in fabricating rock protection kits.

ETAG 027 standardizes the procedure to carry out full-scale tests defining the following aspects:

1. Shape, minimum dimensions and density of the tested block: polyhedron characterized by a unit weight between 2,500 kg/m^3 (156 lb/ft^3) and 3,000 kg/m^3 (187 lb/ft^3); its diameter has to be at least 1/3 of the nominal height of the tested barrier
2. Dimension of the tested barrier: it must have at least 3 functional modules (3 spans)
3. Impact features: the block must impact the barrier in the center of the middle span
4. Minimum impact velocity of the block: no lower than 25 m/s (approx. 90 km/h (55.9 mi/h))
5. The test has to be carried out both at the Maximum Energy Level (MEL) absorbed by the barrier and the Serviceability Energy Level (SEL), which is equal to 1/3 of the MEL (i.e. for a 3,000 kJ (1,106 ft-ton) barrier: the MEL is 3,000 kJ (1,106 ft-ton) and the SEL is 1,000 kJ (369 ft-ton)). Those tests must be done on 2 different barriers A and B, which present the same energy level capacity as well as the same geometrical and mechanical characteristics:
   a) First launch at the MEL: on barrier A. To pass the test, the stopped block cannot touch the ground before the barrier reaches the maximum elongation
   b) Second launch at the SEL: on barrier B
   c) Third launch at SEL: on barrier B. This launch has to be done on the same barrier that received the SEL impact, b), without any repair, but only if the residual height of this barrier, after the first SEL impact is at least the 70% of the nominal tested barrier height (before initial impact). During this test the barrier simply has to withstand impact by the falling block

Figure 2: rockfall barrier with an energy capacity of 6,500 kJ tested according ETAG 027 Guideline. Before (left) and after (right) the MEL impact.

For the designer, the most important measures carried out are the following:

% Maximum dynamic elongation of the interception structure: maximum downhill displacement measured parallel to the reference slope during the impact

% Forces applied on the foundations

% Residual height (h_R): minimum distance between the lower and the upper longitudinal cable, measured orthogonally to the reference slope after the test and without removing the block from the interception structure

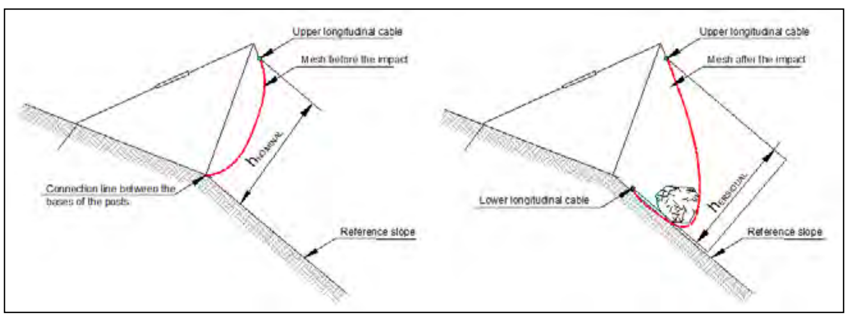

Figure 3: measurement of the nominal height (left) and the residual height (right) of the barrier during the crash test, according to ETAG 027

PRACTICAL CONSEQUENCES

Performance of the barriers

The test procedures set out above illustrate what the designers have to face when considering the barrier resistance results yielded by the crash test. Concerning the features of the impacts, the main difficulties in evaluating the results are the following:

% The crash tests are developed under ideal conditions: in the facility the barrier is assembled as a perfect plane, and the boulder hits in the center of the central module. But the impacts in the field can happen at any point on the barrier (on top, on the bottom, the corners, on the post, etc).

% Many times the impact on the barriers is due to multiple rocks, whereas the test is done with a single rock. Multiple impacts constitute one of the most severe conditions for a barrier, especially when they involve two or more functional barrier modules or spans.

Concerning barrier installation on a slope, the main difficulties are the following:

% The barriers are tested with 3 functional modules (typically 30 m – 98 ft.), whereas on the slopes they are often assembled with longer or shorter lengths.

% Most frequently, after the installation on the slope, the barrier is not perfectly planar, as it may be installed on a very irregular slope - so that the functional modules shows planar distortions (Fig 4), different foundation/post heights and planimetric deviations.

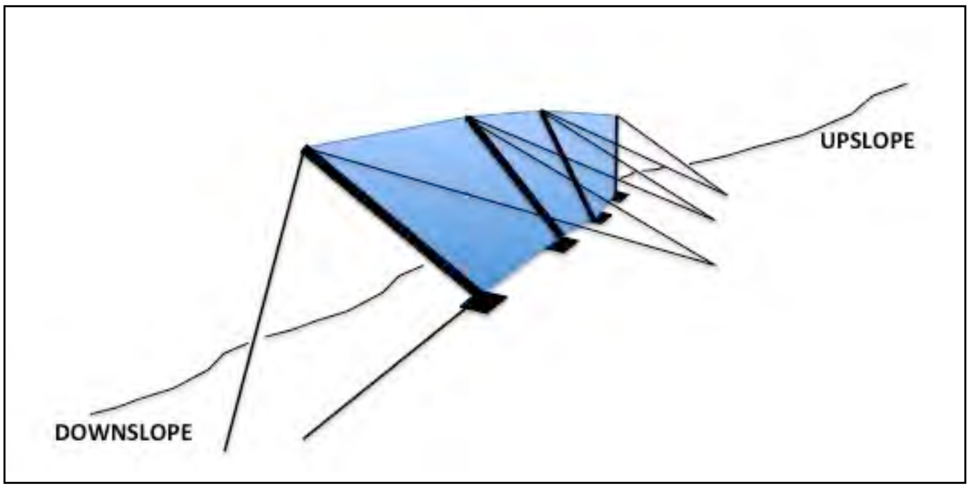

Figure 4: Example of non-parallel posts

For instance, a difference in post elevations modifies the barrier behavior because it induces anomalous stress conditions during the impacts and tilting of the posts (Fig. 5). For this reason tolerances and the operative procedures given by the manufacturer must be followed.

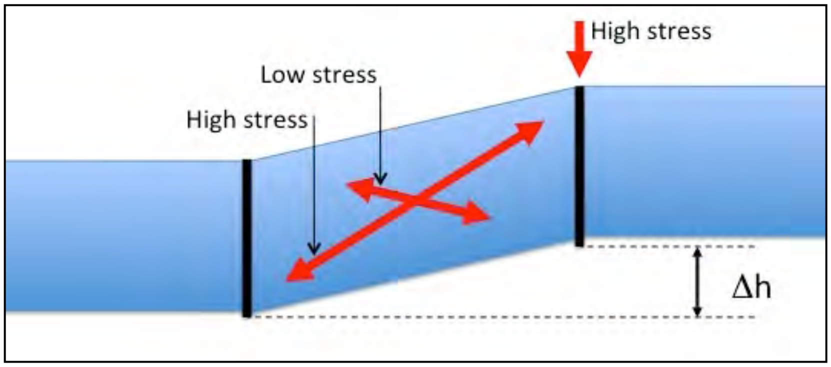

Figure 5: Consequences of different of level between the posts

Since the crash test cannot describe barrier behavior for all impact conditions, the test must be considered as an index test, and the rated energy capacity of the barrier must be considered as nominal. For this reason the designer should apply a reduction coefficient to the nominal energy level of the barrier. Moreover, the designer should take into account where and how the barriers will be installed.

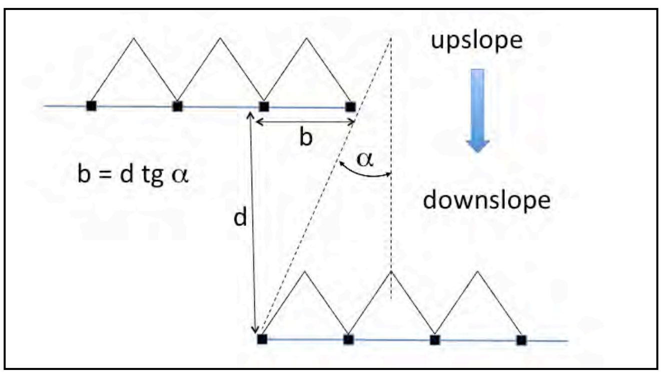

Figure 6: Procedure for the calculation of the spacing between barriers

For example, when two or more independent barriers have to work as one, the contiguous lateral modules should be overlapped because the lateral functional modules could represent a weak point. A simple and safe rule for the overlapping is the following: (a) the spacing between the two alignments must be enough to allow the deformation of the upslope barrier without interference with the downslope barrier (Figure 6); (b) the overlapping must be minimum half module-width. When the barriers are far from each other, the minimum overlap can be estimated with the formula (figure 6)

$$b = d \operatorname{tg} \alpha \tag{1}$$

Where b is the overlap length, d is the distance between the barriers, and α is the deviation angle of the trajectory from the maximum gradient. The deviation angle ranges between 10° when the shape of the boulders is spherical and the slope morphology is regular, and 45° when the shape is tabular and the slope is uneven.

Design impact

In order to minimize undesirable consequences, the designer should take into account that the crash test is an index test, and moreover that several uncertainties affect the trajectories calculation. A very interesting calculation approach is offered by the standard design Italian code UNI 11211:2012 that suggests a coefficient of safety aimed at increasing the estimated energy of impact.

ROCKFALL BARRIER DESIGN

The design of the rockfall barrier can be done considering the Ultimate Limit State (M.E.L. approach) or the Serviceability Limit State (S.E.L. approach) and introducing a safety coefficient discussed above to increase the driving forces on the barrier and reduce the resistance of the structure. The most interesting approach to designing rockfall barriers was proposed in 2006 by Peila D. & Oggeri C., and consequently implemented by the new Italian standard for "Rockfall protective measures" (UNI 11211:4 – 2012).

At the base of this innovative rockfall barrier design methodology there are rockfall simulations, which are normally carried out with probabilistic analyses. The input data for these numerical models are based on data developed by several in-situ surveys such as:

- Geomechanical survey: in order to define the detachment (source) area and the size and the mass of the unstable block/s
- Topographic survey: to define slope morphology, potential trajectories of falling boulders and the best location for the rockfall fence; and
- Geologic survey: to define the restitution coefficients of the soil in order to analyze the rebounds of the boulders during their falls on the slope

To validate a rockfall simulation, a back analysis is always recommended to compare the numerical results with reality. Once the rockfall simulation is calibrated, it is possible to start design of rockfall barriers following the UNI approach. It is recommended that these designs take into consideration both the maximum (MEL) and serviceability energy level (SEL), as defined by the ETAG 027.

The SEL criterion is normally used to reduce maintenance costs of the barrier, when the site is vulnerable to multiple impacts and a very low risk is allowable. This approach is obviously the most expensive, because it is necessary to use a barrier with a capacity 3 times higher than the minimum required. Still, it increases the safety of the protected zone significantly. A typical application of an SEL-design would be for a tunnel portal.

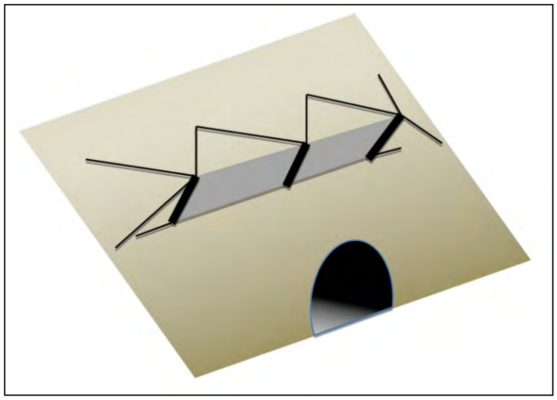

Figure 7: typical application with a rockfall barrier designed with the SEL approach: only low risk is allowed (e.g., tunnel entrance).

The MEL criterion is normally adopted when there is a low frequency of rockfalls or only one boulder is expected to fall, if maintenance can be easily done and/or if the risk level allowed is high. Using this approach, the initial cost of the barrier structure is certainly lower than the one designed for the SEL, but the maintenance cost can be higher and the safety level is lower. Typical uses of rockfall barriers designed at the maximum energy level could be for temporary works, or installations at the base of a re-profiled slope, as often happens in mining applications.

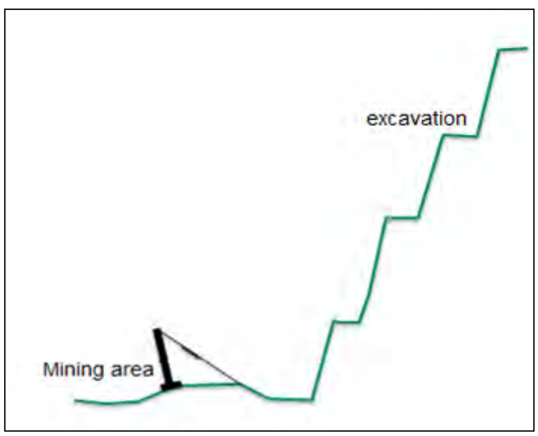

Figure 8: typical application with a rockfall barrier designed with the MEL approach: easy maintenance.

Energy of the barrier

The following steps are used to design a rockfall barrier according to UNI 11211:4 – 2012. The equation that is the basis of this design method is:

$$E_{sd} < E_{barrier} / \gamma_E \tag{1}$$

Where:
- E_{sd} = design energy level developed by the block against the barrier
- $E_{barrier}$ = energy absorbed by the barrier, as defined with the crash test carried out according to ETAG 027 (MEL or SEL)
- γ_E = safety coefficient related to the energy level adopted during the design ($\gamma_E > 1.0$ for MEL approach; $\gamma_E = 1.0$ for SEL approach) and also related to the length of the barrier ($\gamma_E > 1.0$ if the barrier is shorter than 30 m (98 ft); $\gamma_E = 1.0$ if the barrier is at least 30m (98 ft) long)

E_{sd} is defined with the classical formula of the kinetic energy increased by a safety coefficient ($\gamma_R \geq 1.0$), which considers the human risk. In the formula the spin effect of the falling rock can be neglected.

$$E_{sd} = (½\, M_d\, v_d^2)\, \gamma_R \tag{2}$$

Where:
- M_d = design mass of the block
- v_d = design velocity of the block

As per equation number 2, designers must define the design mass and velocity, which are defined as following:

$$M_d = (Vol_B\, \gamma)\, \gamma_{VOL}\ \ \gamma_\gamma \tag{3}$$

$$v_d = v_t\, \gamma_{Tr}\, \gamma_{Dp} \tag{4}$$

Where:
- Vol_B = volume of the design block
- γ =unit weight of the rock
- γ_{VOL} =safety coefficient related to the precision of the design block survey ($\gamma_{VOL} \geq 1.0$)
- γ_γ =safety coefficient related to the evaluation of the unit weight of the rock ($\gamma_\gamma \geq 1.0$)
- v_t = velocity calculated with the rockfall simulation and considering the 95° percentile of the velocities
- γ_{Tr} =safety coefficient related to the reliability of the rock fall simulation ($\gamma_{Tr} \geq 1.0$)
- γ_{Dp} =safety coefficient related to the quality of the topographic survey ($\gamma_{Dp} \geq 1.0$)

Height of the barrier

The minimum height of the barrier has to be defined considering the design height (h_d) plus a free zone (f_{min}).

$$H_{tot} = H_d + f_{min} \tag{4}$$

Where:
- H_{tot} = nominal height of the tested barrier according to ETAG 027
- H_d = design height of the trajectories defined with the following equation:

$$H_d \geq H_t \, \gamma_{Tr} \, \gamma_{Dp} + R_{block} \, \gamma_R \tag{5}$$

- H_{95} = height of the trajectories defined with the numerical simulations and considering the 95° percentile of the heights
- R_{block} = average radius of the design block
- γ_R =safety coefficient on the radius of the block ($\gamma_R \geq 1.0$)
- f_{min} = safety zone that cannot be impacted (usually $f_{min} \geq 50$ cm (2 inches))

Distance between the barrier and the protected zone

The minimum distance between the barrier and the structure to be protected is determined as follows:

$$D_A \geq D_b \, \gamma_d \tag{6}$$

Where:
- D_A = minimum distance between the barrier and the protected zone
- D_b = maximum dynamic deformation of the barrier, measured in accordance with ETAG 027 after the full-scale crash test at the MEL
- γ_d = safety coefficient related to the energy level adopted during the design ($\gamma_E > 1.0$ for MEL approach; $\gamma_E = 1.0$ for SEL approach), to the length of the barrier ($\gamma_E > 1.0$ if the barrier is shorter than 30 m) and to barrier-span impacted by the boulder ($\gamma_E > 1.0$ if the lateral span of the barrier may be impacted)

Design strategy

In order to minimize the effects of technological limitations of the fence, designers must evaluate the results of the full-scale tests in terms of residual height and maximum elongation. To increase the safety level after the first impact, a high residual height is strongly recommended, a low deformation of the barrier is also suggested when the distance between the fence and the protected zone is limited. Considering these two characteristics, Maccaferri has developed its rockfall barriers with the highest performance in these areas. Other important parameters to be considered during barrier design are the forces that the structure transmits to the foundations. These values are measured during the crash-tests in order to define the type and length of the barrier foundations, that are governed by ground conditions.

MULTIPLE IMPACTS ON ROCKFALL BARRIERS

Both the design procedures and the crash tests do not consider multiple impact events, even if they are quite frequent. A multiple impact happens when one or more functional modules of the barrier mesh are hit by two or more rigid bodies; and may also include the deformation of the structure and severe impacts by large deformable bodies involving a large portion of the mesh such as debris flows of snow avalanches in this category (Table 1).

Table 1 – Examples of multiple impacts

Case 1 - Example of performance Maccaferri barrier impacted by multiple blocks. Impact energy higher than the nominal of the barrier – Scilla (Reggio Calabria – Italy)

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Barrier characteristic	Value	Photo
Maximum energy absorption:	5,000 (1844 ft-ton) kJ	
Damages: 3 spans were involved in the impacts. One post was impacted and almost completely destroyed. Several energy dissipaters worked at their maximum capacity.. Notes: the barrier stopped approx. 50-70 m^3 (65-92 yd^3) of rocks. The central part of the barrier had a residual height of 3 m (10 ft) (50%), while the lateral spans had maintained a residual height of 5.5 m (18 ft) (90%).		

Case 2 - Performance of the 3,000 kJ (1,106 ft-ton) impacted by energy higher than the nominal one - Arnad (Valle d’Aosta – Italy)

Barrier characteristic.	Value	Photo
Maximum energy absorption:	3,000 kJ (1,106 ft-ton)	
Damages: 2 spans were involved in the impacts. One post was tilted downslope and the other one was bent. Notes: the barrier stopped approx. 30 m^3 (39 yd^3) of rocks. The biggest block had a volume of approx. 12 m^3 (16 yd^3) and it developed energy 1,000 kJ (369 ft-ton) higher than the maximum nominal (MEL) capacity of the barrier. This boulder impacted against the post. Minimum residual height measured = 50%.		

Case 3 - Performance of the 3,000 kJ (1,106 ft-ton) barrier impacted by an avalanche – Gitterberg (Austria)

Barrier characteristic	Value	Photo
Maximum energy absorption: Damage: 1.5 spans were involved in the avalanche. The lateral post was tilted downslope and several energy dissipaters worked at their maximum level. Notes: the barrier stopped an avalanche composed by rock debris and big trees. In the impacted span the minimum residual height was evaluated close to 50% of the nominal height, and the downslope deformation was approx. 3.5-4.0 m (11.5 – 13 ft)	3,000 kJ (1,106 ft-ton)	

Case 4 - Performance of the 2,000 kJ (738 ft-ton) barrier impacted a debris flow - Valsavaranche (Valle d'Aosta – Italy)

Barrier characteristic	Value	Photo
Maximum energy absorption: Damages: 4 spans of the longest barrier (80 m (262 ft) long) were impacted by a debris flow with a total volume of 200 m^3 (262 yd^3). Only 8 energy dissipaters worked at their maximum capacity. Notes: the barrier stopped the debris and it maintained its residual height, higher than 65%-70%. The downslope deformation was approx. 2.5-3.0 m (8 -10 ft).	2,000kJ (738 ft-ton)	

Where multiple impacts on the same functional modules of a barrier are concerned, Maccaferri has a great deal of experience based on MEL and SEL crash testing using the ETAG 027. Barriers tested by Maccaferri are tested to failure with a sequence of multiple impacts until the barrier screen is pierced. The impacts are located in the same position of the main impact (central position of the central module), where the mesh has already deformed, so that the measured residual resistance is related to the remaining capacity of the brakes and cables to dissipate energy. This procedure allows an understanding of how the barrier components interact, and the identification of the weak points in a system. By improving the resistance of the weak points, the performance of the final structure is improved.

Concerning impacts on different functional barrier modules, the field test analysis is quite complicated for several practical reasons, and the understanding of barrier behavior is mostly based on case histories. When the impacts involve two or more functional modules, the barrier is deformed at several positions along its length (Figure 9). In this case, the weakest points of the structure are the upper and lower longitudinal cables - since they are linked to the posts, after impact they no longer follow a straight line and in this condition the ability of the cables to slide in the post slots is reduced and the capacity to dissipate energy is dramatically lower. This unfavorable condition is emphasized if the barrier has been installed with anomalous geometry (as shown in Figures 4 and 5). Even if falling boulders can rarely impact them directly, the lower longitudinal cables lie close to the post footplates, which creates a stiffer point in the barrier. This exposes the lower cable to high stress conditions and a risk of breaking. On the other hand, the upper longitudinal cable can move a little bit with the posts, so that the risk of overstress is lower; but because of barrier height reduction, the exposure to direct impacts on the cable is high. A direct impact on the upper cable can cause cutting and collapse of the whole structure.

The case histories show that the barriers are effective also when the impact energy is higher the SEL. In some cases (Table 4) the energy level has been estimated to be several thousand kJ (ft-lb) greater than the nominal capacity; even if the structures were totally ruined (large elongation, posts severely bent, no possibility of cost effective maintenance), they were effective and not one boulder passed the barrier (Invernizzi and Giacchetti, 2010).

Figure 8 - Multiple impact event. The barrier with nominal capacity 5000 kJ, 6.0 m high, has been severely impacted by a large rockfall, with an estimated energy between 6500 and 9500 kJ. Despite the large deformation and the huge size of the blocks, the top and lower longitudinal cables have not been broken and the barrier retained almost 50% residual height.

We can say with certainty that dynamic barriers can withstand impact energies exceeding the nominal capacity, or multiple impact events, but we must be aware that in these non-standard conditions the desired performance cannot always be ensured. At present, the best way ensure the effectiveness of a barrier is a rational design process. Even if in design practice prediction of a multiple impact effect is really complicated, following the previously described design procedure and using the Service Energy Level should allow the correct sizing of barriers for high performance and low maintenance in critical situations.

CONCLUSIONS

In 2008, the European Organization for Technical Approvals (EOTA) issued a Guideline for Rockfall Protection Kits (ETA 027) in order to define a procedure to carry out the full-scale-crash tests on rockfall barriers and obtain the European Technical Approval (ETA) and the CE marking. This guideline should be considered as an index test that fundamental to comparing different barrier types and helping in barrier installation design. To consider all the technical limits of these structures in-situ, a new design approach has been introduced with the Italian Standard UNI 11211:4-2012 "Rockfall protective measure". It considers both the technical limits of the barriers and design difficulties by introducing safety coefficients for reduction of the nominal capacity of the barrier on one hand, and for the increasing of the driving energy on the other.

In order to install barriers in accordance with the geometric tolerances indicated in the manufacturers manual, the design should be very accurate and consider: the actual slope morphology; the optimum length of each barrier, and any required overlapping for two or more independent barriers to work as one.

Last but not least, in order to minimize the consequence of technological limitations, especially regarding multiple impact events, barriers should offer high performance, that is lowest elongation and greatest residual height, maximum energy levels (MELs) being equal. In this sense, even if ETAG 027 defines the best residual height (class "A") as equal to or greater than 50%, the experience of Officine Maccaferri has been that Class "A" barriers often do not offer enough safety margin in the case of multiple impact events. We have found that elongation is a very important parameter, especially where available space for barrier assembly is limited. To address this problem, Officine Maccaferri has developed barriers with structures able to distribute impact forces over a larger surface and reduce the total barrier elongation. Through ongoing testing and design, Maccaferri has developed barriers with better elongation and residual height retention for the world market.

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Use of Rockfall Rating Systems in the Design of New Slopes

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ABSTRACT

One typically uses different means to evaluate a highway rock slope depending on whether it exists currently or is in design. For example, the Rockfall Hazard Rating System (RHRS) and derivatives are commonly used to evaluate existing slopes and inform decision makers who are managing rock slope inventories. In contrast, kinematic and limit equilibrium analyses and methods based on observation and probability, such as Ritchey Ditch Criteria, Rockfall Catchment Area Design (RCAD), and the Colorado Rockfall Simulation Program (CRSP), are typically used to provide information for decision making when designing new slopes. Is there good reason for this difference? This paper raises this challenge and proposes that rating systems are not just good for existing inventories; they are good tools for design of new and rehabilitated slopes. Some of the challenges in using a rating system for design are addressed and the importance of distinguishing risk from hazard is highlighted. Finally, the paper demonstrates how rating systems can help us move towards and define a standard of practice for rock slope design in Colorado and other mountainous environments, and it discusses the challenge of establishing and applying an appropriate standard.

INTRODUCTION

This paper is written from the perspective of the highway industry though the points made are more broadly applicable and may have relevance to other owners of infrastructure and facilities, especially in mountainous terrain. Public highway agencies usually have a few goals that define their mission, often including the following:

- Provide safe highways;
- Provide highway systems that meet the broad range of user needs, ensuring consistent availability of transportation corridors;
- Provide highways with operation and maintenance costs that can be anticipated and planned for;
- Be good stewards of natural and scenic resources; and
- Be good stewards of public funds (financial resources).

Decisions regarding rock slopes should be and usually are based on these goals, satisfying each to some extent. It is recognized that these goals cannot all be optimized individually because they sometimes pull in different directions. Rather, there is a balance that is strived for that represents the optimum design for a project, a transportation corridor or system, and/or an owner.

One typically uses different means to evaluate a highway rock slope depending on whether it exists currently or is in design. The Rockfall Hazard Rating System (RHRS) (Pierson and Van Vickle, 1993) and many derivatives are used to evaluate existing slopes and inform decision makers who are managing rock slope inventories. In contrast, kinematic and limit equilibrium analyses and methods based on observation and probability, such as Ritchey Ditch Criteria (Ritchie, 1963), Rockfall Catchment Area Design Guide (RCAD) (Pierson et al., 2001), and the Colorado Rockfall Simulation Program (CRSP) (Jones et al., 1999), are typically used to provide information for decision making when designing new slopes. The thesis presented here is that rating systems are not just good for evaluating and managing existing inventories; they are good tools for design of new and rehabilitated slopes. Rating systems can help us define and move towards a standard of practice for rock slope design that is based on risk, and will help agencies balance their efforts on divergent goals. This is true whether the slope already exists or is in design. Throughout this paper RHRS, RCAD and CRSP are used to represent certain tools for convenience and simplicity. These are publically available in some form but this is not an endorsement of these products over others. Similar products could be substituted wherever these titles are used.

BACKGROUND

It is not practical to prevent all rocks on slopes (cut or natural) from falling, to prevent all falling rocks from reaching highways, or to immediately remove fallen rocks from highways. Therefore, rocks will impact vehicles, either moving or stopped, and vehicles will impact rocks. Programmatically, a certain low level expectation of this must be tolerated. Furthermore, highways below cut or natural rock slopes will have rock removal and repair as a maintenance need. In other words, it is not a question of “if” rockfall will occur; it is a question of how much is acceptable, or tolerable. One can measure this in terms of hazard or risk. Hazard and risk definitions vary but generally *hazard* is a measure of the likelihood of rockfall occurrence; whereas, r*isk* is a measure of likelihood and consequence of occurrence. Figure 1 shows an example where the consequence could be considered high.

Figure 1 - Rock slope at Glacier National Park where rockfall is expected to reach the travel lanes of the road (courtesy of Cornforth Consultants, Inc.).

Risk is the measure that best addresses our objectives because it includes consequences and can potentially be used to compare rockfall risks with other risks owned by the agency. Hazard is important to characterize because it must be represented in the calculation of risk, but knowing hazard alone only goes so far. Consequences must also be characterized. Consequences used in the calculation of risk include, for example: none, increased maintenance, public or private property damage, and injury or death, to one or many (both motorists and pedestrians).

The highway industry follows a loosely defined standard-of-practice, tempered with the specific needs of our projects, such as minimizing environmental impact and considering cost in proportion to the type and volume of traffic along the road. The standard of practice uses analysis methods that address hazard and consequence, but often not together, or in a systematic way. Risk is seldom explicitly addressed and, as such, is not part of a current standard.

Standards would allow us to explain to our multi-disciplinary teammates, project managers, partner land management agencies, and the public, in a consistent manner, why certain decisions are made. Standards would also explain that there is always maintenance and safety risk associated with rockfall, and would allow for characterization of that risk in a systematic way. In addition, they would also frame desired performance objectives in a manner allowing comparison to broader route or corridor objectives, including environmental, capacity, and operating cost issues. As such, the criteria used to define a standard should be with respect to risk; not factor of safety, percent retained in ditch, or hazard, so our goal is to use our analysis methods to provide us a measure of risk.

ANALYSIS METHODS

In current practice, various methods are used to analyze rockfall hazard and consequences. Each method has strengths and weaknesses, and in their own way contributes to an understanding of risk. To understand this contribution it is important to have a consistent definition of the failure event and the risk associated with it. If we define failure as the event of a rock starting to fall from a slope and evaluate the definition of "risk" the following relationships and influences are observed:

Risk = *f*{Probability of Failure, Consequence of Failure}

Probability of Failure = *f*(site conditions); which include geology, climate, presence of water, construction techniques, slope angle/aspect, reinforcement, retention, etc.

Consequence of Failure = *f*(proximity to people/property, potential energy of rock/debris mass); which are affected by catchment width/depth (effectiveness), height to failure, size/volume of rock, slope angle, surface attenuation, retention/attenuation measures, etc.

Figure 2 shows an example where steps are taken to reduce probability of failure. Figure 3 shows (a) a case where the ditch and run out area is far from the road, and (b) where a barrier is used on the same road to contain rockfall in the ditch. In both cases, the consequence of failure is reduced. With these definitions in mind, the common tools for analysis are described below and reference is made to how they contribute to understanding risk.

Figure 2 - Hand-scaling of a cut after construction to reduce probability of failure (hazard).

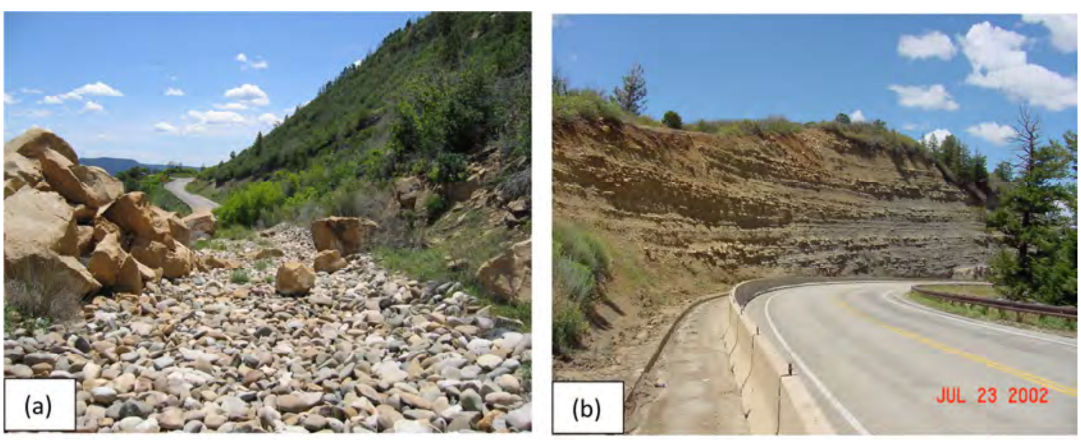

Figure 3 - Examples at Mesa Verde National Park where ditch effectiveness and distance from travel lanes effectively confines rockfall to the fallout area, reducing the consequence of failure.

Rock Slope Stability Analysis

Kinematic and limit equilibrium analyses can be used to calculate rock slope stability (Wyllie and Mah, 1998). These techniques are used to provide an assessment of hazard unique to a specific slope or site. Unfortunately, slopes are often found to be theoretically unstable or to have an unacceptably low factor of safety, when such is not actually true. This is because the analyses assume that discontinuities do intersect, are planar, and have largely frictional strength characteristics absent of any cementation or intact rock strength. These are reasonable and cautious assumptions given the uncertainty in the data that usually exist (mostly related to few, widely scattered measurements), but they combine to produce conservative solutions, not a best estimate of the average. Often it is assumed that kinematically feasible failures extend the full height of the cut; another cautious, conservative assumption, but not a best estimate of failure size and location, and therefore, not a good basis for estimating consequences which are related to volume and fall height.

Though an analysis of this type is usually deterministic and results in a factor of safety, there is an implicit relationship between factor of safety and probability of failure. In other words, these methods establish an estimated probability of failure implicitly and they could be modified to do so explicitly. The probability of failure is the measure of hazard and one of the two key inputs for calculating risk. These methods do not address failure consequence such as travel distance and bounce heights. As described above, they can be used to estimate volume and, through that prediction, a measure of consequence of failure, but these methods are generally best suited for analyzing failure probability and not consequence. A calculated factor of safety (or estimate of failure probability) is not an estimate or measure of risk because it does not address consequence.

Rockfall Catchment

These methods provide rational and statistical means of estimating ditch effectiveness and the effectiveness of other mitigation measures, such as fences, barriers and attenuators. They are usually used alone to calculate the percent of rocks that would reach the road if a given shape/size distribution were to fall, given a certain geometry of ditch and other mitigation measures. The ditch is then designed to meet a certain retention criteria. Other means of retention can be added to the design if needed, such as barriers, fences and attenuators. We have two types of tools in this area: those that are based on observation, such as RCAD, and those that are based on mechanical or numerical simulation, such as CRSP and RocFall (RocScience, 2012).

The RCAD empirical methodology is simple and powerful, yet results can be misleading if applied to conditions different than those from which the data were obtained (e.g., rock type and shape, slope geometry, slope-rock interaction). In the RCAD design charts for slopes of certain heights and slope ratios, each of the charts provides the percent retained per ditch geometry for a drop height equal to the slope height. Note, however, if some portion of the total rockfall hazard initiates lower on the slope those rocks have a higher percentage of being retained – as shown in the RCAD charts for shorter slope heights. Therefore, if an entire cut or a section of cut with similar characteristics is considered as a unit, it may be appropriate to explicitly state that the catchment design for a certain percent retained includes the integrated retention of rockfall for the entire slope – not just the retention based on the highest rockfall initiation. For example, consider a triangular-shaped cut with a maximum height of 24 meters (80 feet). If the catchment is designed for 95 percent retention based on the 24-meter RCAD design charts, the actual percent retained assuming equal likelihood of rockfall initiating anywhere on the slope could be over 99 percent. If 95 percent catchment retention is actually the performance target for the slope, the catchment area should be designed to about 60 percent retention based solely on the design charts. This type of integration is typically not done with RCAD analysis or with CRSP type analysis so reported analyses are generally for a 'design event', not a statistical measure of expected performance.

CRSP, RocFall and other analytical/mechanical methods have different limitations and do not exactly replicate the observations at the RCAD study quarry (where the RCAD data are absolutely correct), but they may offer the best way to extrapolate RCAD findings to different rock and site conditions and evaluate the importance of parameters not varied in the RCAD work, including variable slope materials and geometries. They also provide for the rapid assessment of retention for a defined distribution of rock sizes and shapes initiating over a delineated initiation area. The level of uncertainty in the predictions is considerable and care should be used in their application.

RCAD and CRSP are examples of tools used for consequence management – evaluating the outcomes of falling rocks, rather than the probability for rock failure resulting in rockfall – either in the design of new slopes, evaluation/maintenance of existing slopes, or analysis of specific rockfall events. Consequence is directly but not completely addressed by RCAD (and CRSP, etc.) because proximity is also dependent on average daily traffic, vehicle speed, sight distance, highway maintenance, roadway width, shoulder area(s), clear zones, etc., which are independent design considerations.

In summary, RCAD, CRSP, and other run-out and energy prediction tools are only effective in understanding and managing part of one of the key variables affecting risk: the consequence of failure. Estimating rockfall retention is only part of the process. Therefore, it is not good risk management to fix a standard retention criterion of, for example, 90 percent retained, as a goal for design and maintenance when what is desired is a certain acceptable level of risk.

Hazard Ratings

The RHRS and many State derivatives (Drumm et al., 2005; NYDOT, 2007), are used to evaluate rockfall hazard. For the RHRS, ten factors are scored on an exponential scale (of approximately 1 to 100) and summed to produce an overall slope rating that primarily indicates the likelihood of impact between a moving car and a fallen rock. A score in the range of 500 would typically indicate a very high hazard.

The RHRS is often used as though it were rating risk, not hazard, which is not too surprising because it includes factors that address both probability of failure and consequence of failure. In fact, defining failure as we have, as the event of a rock starting to fall, the RHRS has four of ten factors addressing hazard (failure probability) and six factors addressing consequence. The four factors primarily addressing the probability of failure are:

- Geologic Character Case 1 – Structural Condition/Rock Friction (failure along structural discontinuities);
- Geologic Character Case 2 – Differential Erosion Condition/Erosion Rate (failure due to erosion);
- Slope Water and Ice Conditions (climate-related slope water occurrence contributing to regular or seasonal rockfall); and
- Rockfall History (approximate frequency of occurrence).

The six factors primarily addressing consequence of failure are:

- Slope Height (potential energy contributing to severity of rockfall impacts and roll-out distance);
- Ditch Effectiveness (degree to which rock does not make it onto the travelway);
- Average Vehicle Risk (opportunity for vehicles to engage fallen rock or be struck by falling rock);
- Percent Decision Sight Distance (opportunity to avoid fallen rock on the roadway based on posted speed limit);
- Roadway Width (opportunity to avoid fallen rock within the travelway); and
- Block Size/Volume per Event (severity of impact to vehicles/structures, degree of roadway coverage with fallen rock, potential to close the roadway, etc.).

That the RHRS is referred to as a hazard rating has more to do with the fact that the risk is evaluated in a qualitative way, not in a way based on probability. For example, note that "Ditch Effectiveness", as one of ten equal factors, is only one-tenth of the 'hazard'. Thus, if a ditch is perfectly effective versus perfectly ineffective, it could make a difference of as little as 10 percent on the RHRS. With respect to public safety risk, this is counter-intuitive because if all rock is held within a ditch there is no risk of impact (or other safety consequences).

Nevertheless, the presentation here shows that we have methods that address probability of failure, methods that address consequence of failure and methods that address both probability and consequence. None of the methods are perfect but progress towards a goal of risk-based standards is best made from the methods that consider both probability and consequence of failure, which are rating systems like the RHRS. Note that rating systems also present a logical way to capture kinematic/limit equilibrium and fall trajectory and catchment calculations so the analysis tools and methods would still be an important part of the risk-based design process.

DESIGN OF NEW SLOPES

There are two types of "new" slopes often encountered. One is a significant widening or improvement of a cut that currently exists (Figure 4). This often results in an increase in cut size and height. The second type is a new alignment where no current exposure of the rock exists. In either of these cases the design could be developed to result in a level of risk that is set by the owner. The risk could be set to be equivalent to an average level of risk for a corridor or road system or, for example, it could be set to a lower level so that in time the risk posed by rock slopes system-wide would drop. If the risk assessment is quantitative, the risk could also be set with reference to other risks assumed by the owner. This approach is consistent with steps being taken in other parts of highway design where owners are balancing their investments in a corridor to lower risk for the whole corridor, not just for certain elements (e.g., improvements to safety geometrics).

Figure 4 - New slope created by widening and straightening to increase roadway capacity and traffic safety.

In order to move in this direction, the first step is to start using analysis methods that address both of the two key components of rockfall risk: hazard and consequence. As discussed above, the RHRS addresses both of these components and if it were used instead or in addition to kinematic and limit equilibrium analyses (which address probability of rockfall) and CRSP and RCAD (which address consequence of rockfall), for example, then one could solve for a rating that would be expected after construction. The rating would be a function of the geometry of the cut and the roadway design, the site conditions, and the construction techniques. The geometry of the cut and roadway template are known definitively in advance and the site conditions are predicted based on site investigation during the design phase. Construction techniques specified as part of the construction contract, as they typically are, allow one to predict the condition of the new slope after blasting and excavation.

There are some challenges to using rating systems to design new slopes. The primary challenges are with respect to giving an assessed RHRS rating to a slope that can't be observed because it doesn't exist yet, and using existing rating systems that are based on summations, not products, and thus don't capture the conditional probability that is required in a risk assessment.

For consideration of the first challenge, the RHRS factors are presented in Table 1 along with the general process for evaluating and scoring, as well as ways during the design phase that the design could be modified to change the RHRS score. As can be seen from the table, some of the factors are evaluated and scored based on line and grade on plan sheets and on routine information normally in the hands of the design team. Other factors provide an opportunity to incorporate the results of CRSP and RCAD, and of kinematic and limit equilibrium analysis methods, which are the more common analysis methods for new slopes. Figure 5 shows where observations and investigations, and certain specific analysis methods, contribute to the RHRS factors and also how the RHRS and other factors contribute to risk.

Other factors are going to require predictions be made based on limited information and site observation. Interestingly, this is no different than most geotechnical designs, wherein limited explorations are used to predict capacity and performance – it is just unusual from a rockfall perspective. It is easy to imagine how the predictions could be tested (re-rated) after construction to confirm what was discovered in construction was as envisioned in design.

The second big challenge to using the RHRS in this way is that RHRS is based on the summation of factor scores, not the determination of conditional probability. For example, a decision tree for the calculation of rockfall risk might be something like this:

(1) Probability of rockfall initiating = A (based on four RHRS failure factors, possibly supplemented by kinematic analyses);
(2) Given rockfall initiates, the probability that it is not retained by the ditch = B (based on three RHRS factors related to ditch effectiveness, possibly supplemented with RCAD/CRSP analyses); and
(3) Given that rockfall escapes the ditch, the probability that the rockfall and vehicle collide = C (based on three RHRS factors related to hazard avoidance by motorists).

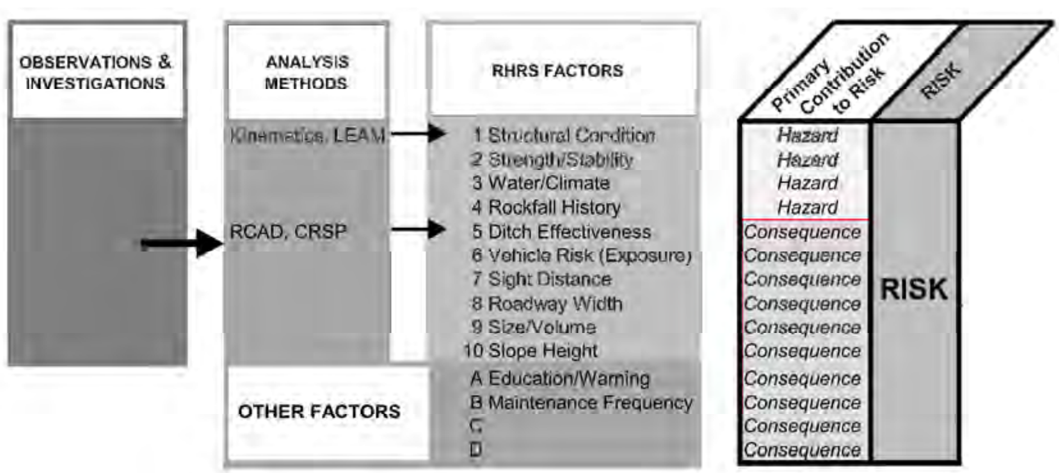

Figure 5 - The evaluation of risk through observations, analysis rating system and other factors.

In this example, B and C are conditional probabilities and the probability that rockfall and vehicle collide is the product, A x B x C. From this it is clear that if the ditch was essentially 100 percent effective the risk of a rockfall – vehicle collision is effectively zero. This challenge is most notable for the ditch effectiveness factor but one can envision its impact on other factors as well. Ideally, this challenge would be dealt with by converting the RHRS to a system based on multiplication of factors from one based on summation. Until that can happen it is suggested here that other criteria be used in addition to a RHRS rating to evaluate suitability of a rock slope design. For example, the owner might specify that regardless of the calculated RHRS rating for the slope in design, a certain percent retention from the maximum slope height is required.

Table 1 - Evaluation of RHRS scores in the rock slope design phase.

RHRS Rating Element	Evaluation and Scoring Option	Alteration Options
Consequence Related Elements		
Slope Height	• Based on planned road grade, slope angle and topography	• Alter road grade or slope angle • Add benches or measures such as bolts, mesh or attenuators to change effective height
Ditch Effectiveness	• Use CRSP or RCAD to evaluate effectiveness in terms of percent retained	• Change ditch geometry • Add barriers
Average Vehicle Risk	• Rate based on design sources	• Design to prevent traffic slowing and increase speed limit
Percent Decision Sight Distance	• Rate based on roadway design	• Work with geometrics and clear zones to increase decision sight distance • Reduce speed limit
Roadway Width	• Rate based on roadway design	• Consider shoulder or non-travel lanes as available for retention.
Block Size/Event Volume	• Use adjacent sites for reference, rock cut mapping • Borehole information • Kinematic or limit equilibrium analysis	• Mesh, bolts or other measures to reduce the size or volume of material that could fail • Specify scaling
Probability of Failure Related Elements		
Geologic Character Case 1 Structured Rock	• Use adjacent sites for reference, rock cut mapping • Borehole information • Kinematic or limit equilibrium analysis	• Alignment changes
Geologic Character Case 2 Differential Erosion		
Water and Ice Condition	• Use adjacent sites for reference, rock cut mapping • Borehole information	• Install drainage
Rockfall History	• Base on regional experience and construction method • Borehole information	• Specify construction method, scaling

BENEFITS OF RATING SYSTEMS

Rating systems embrace the other analysis methods often used in design and they address risk because they capture both hazard and consequence. Their use gives an owner the ability to manage risk and gives the potential for establishing a standard of practice, even if only loosely defined.

Risk Management

Risk management is an ultimate objective of an owner whether looking from the perspective of public safety, performance of the system or financial and/or natural resource stewardship. However, getting to a comprehensive suite of risk management tools is a long term objective for most and not something that can be done right away. Additionally, one needs to consider subjective elements in the formulation as well. For public owners there certainly are public tides that need to be heeded and there are ranges in the tolerability of risk. For instance, rockfall fatalities comprise only about 0.005 percent of highway fatalities nationally yet they have a public interest that far exceeds that, perhaps because of expectations we have set for roads free of these risks coupled with the often dramatic nature of fatal rockfall events.

On the other hand, managing agencies may have greatly diverging risk tolerances from the expectations of the traveling public, as well as within and amongst interagency entities. For example, the consequences of rockfall may be far greater when considering the direct and indirect costs of road closures, including maintenance, repair, alternative route capacity, socio-economic impacts, etc., and these may be the broader-view risks to be managed by public highway agencies. In contrast, natural resource management entities may consider preservation of the corridor viewshed a priority, resulting in a higher risk tolerance for rockfall and justification for routine roadway maintenance and repair expenditures and inconvenience.

Moving rock slope design to a framework that estimates risk will help owners set priorities based on these costs. The initial steps proposed in this paper will not result in a quantitative risk calculation that could be compared to other risks on the system, such as pavements and bridges, but it is a step in that direction. This type of analysis is also a positive step in that it will allow owners to evaluate the risk reduction benefits of some measures and design alternatives with respect to their life cycle costs and broader corridor management objectives. We are starting to build more and more mitigation measures directly on rock slopes and are getting information not just on how they can reduce the hazard and the consequence of rockfall, but on how long they last and how much they cost to own and maintain.

Standard of Practice

A precise standard of practice for rock slope design and rockfall mitigation will be difficult and perhaps impossible to define, even using risk as its measure, as there are many intangibles. Nevertheless, it is envisioned that if designs and assessments of existing slopes are evaluated with respect to risk in consistent ways, a band of practice can be established as shown in Figure 6. Quantifying slope performance somewhere within this band establishes the standard of practice for a given roadway or section and will assist owners in balancing the five goals listed in the introduction to this paper.

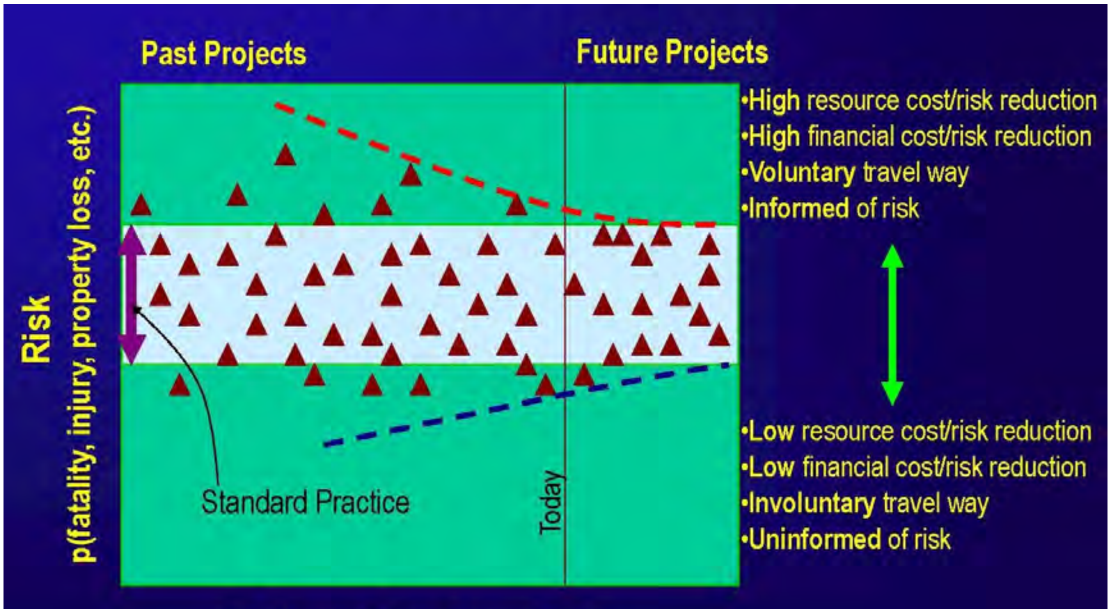

Figure 6 - The evolution of a band of standard practice.

An established range of practice would be useful for owners and responsible professionals engaged with the owners in setting expectations. Establishing the current range of practice, as well as future targets for slope performance, allows owning agencies to manage internal expectations as well, identifying when competing mission priorities fall within or outside accepted performance levels. The proposed risk-based approach to designing new slopes or mitigating old ones would provide a framework for setting a standard for a particular route or region and in evaluating alternative designs, such as ditch width versus scaling or mesh installation in a rational way. Having a standard for a given project will help the appropriate allocation of resources targeted at route-appropriate performance objectives and avoid "worst-first" management of existing slopes and over-/under-design of new or rehabilitated slopes.

CONCLUSIONS

One typically uses different means to evaluate a highway rock slope depending on whether it exists currently or is in design. For example, the Rockfall Hazard Rating System (RHRS) and derivatives are commonly used to evaluate existing slopes and inform decision makers who are managing rock slope inventories. In contrast, kinematic and limit equilibrium analyses and methods based on observation and probability, such as Ritchey Ditch Criteria, Rockfall Catchment Area Design (RCAD), and the Colorado Rockfall Simulation Program (CRSP), are typically used to provide information for decision making when designing new slopes. The shortcoming of all of these methods is that they only address probability of failure or consequence of failure. As such, if they are used alone or not in some consistent combination, they lead to new slopes that are not designed on the basis of risk.

Existing rockfall hazard rating systems such as the RHRS and others that have been developed similarly, or from the RHRS, provide an opportunity to design new slopes on the basis of risk. These rating systems currently have their own limitations in that they are based on the summation of factor scores and this prevents them from being used to actually calculate risk based on conditional probabilities of events, such as rockfall initiating, then escaping a ditch, then coming in contact with a car. Nevertheless, with some additional design criteria to be used in conjunction with a targeted rating value, these systems could allow owners to design new slopes to meet certain approximate risk standards. The FHWA is interested in exploring this idea further and in exploring the development and deployment of true risk-based rating systems for the future.

It is unlikely that this development will result in a singular expectation of risk associated with rockfall on highway slopes, or a precise standard of practice because of the many factors considered in design. These factors were all part of the recent reconstruction of Guanella Pass Road in Colorado and the varied rockfall mitigation measures that were included (example shown in Figure 7), and they include the goals to:

- Provide safe highways;
- Provide highway systems that meet the broad range of user needs, ensuring consistent availability of transportation corridors;
- Provide highways with operation and maintenance costs that can be anticipated and planned for;
- Be good stewards of natural and scenic resources; and
- Be good stewards of public funds (financial resources).

It is expected that the development of a risk-based design approach will result in improved communication of the desires and expectations of highway owners, highway designers, and highway users. Such an approach will facilitate management of the performance of a system of highways – something of great interest to highway owners. It is also likely that other public and private entities with interests on or near rock slopes will find this of value for the same general reasons.

Figure 7 - A rockfall fence installed on Guanella Pass Road.

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The Engineering Geologist and Transportation

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ABSTRACT

THE ENGINEERING GEOLOGIST AND TRANSPORTATION

Transportation engineering geologists are called on to perform various duties for a public agency or consulting firm. Traditionally, many of these groups were named “Soils and Geology” units and were staffed by personnel with an engineering geology background. The geotechnical branch of civil engineering gained strength during the 1970’s and now many of the groups are staffed by both engineering geologists and geotechnical engineers. The tasks and responsibilities between the two professions are sometimes blurred.

The responsibilities of Engineering Geologists within the transportation industry vary as widely as the geology of the 50 States. Their principal responsibilities include exploration and classification of earth materials, geologic mapping, geomorphology, geologic hazard identification, groundwater, geologic processes, rock discontinuity characterization. Problems can arise when engineers with little or no background or education in geology perform these tasks. Many geotechnical engineers have never had a university level course in geology.

Transportation engineering geologists should have a role in the planning or NEPA process, identification of geologic hazards, route selection, bridge foundations, subsurface characterization and location of materials, slopes, especially rock slopes.

Highway engineering involves many aspects of geology. Applying the principals of geology should make for less risk during construction and better, longer lasting, trouble free highways. The tasks an engineering geologist performs in highway engineering should be better defined given the evolution of the practice.

INTRODUCTION

During the last 100 years the engineering geologist has been an integral part of the highway construction process. History has shown that geologists working with highway engineers have provided the best solutions for the traveling public.

Most Departments of Transportation and geotechnical consulting firms have maintained a Geotechnical Engineering or Soils and Geology Section. The size as well as functions these groups perform can vary widely. Some are led by geologists, some by engineers. Recently, many agencies as well as consultants have downsized and are struggling to maintain a highly experienced and skilled staff of engineering geologists.

We are entering a new era with smaller highway budgets, legislative action for lower taxes and smaller government. The traditional funding of highways through user paid gas taxes raises less money each year as many states tax per gallon has remained the same for decades. The present political climate along with a recession, has left many states with little budget for new highway construction, opting to utilize the limited funds for maintenance.

The State of the Practice

The endangered practice, many of agency and consultant staffs now employ less geologists than in the past. The retirement of the interstate builders is complete, staffs are shrinking, with a new focus on safety and capacity improvements. New deliver methods such as design build and Public Private Partnership (P3) promising faster cheaper are beginning to be the norm. Much of Highway Geology work is now conducted by the "Geotechnical Engineering community", a mix of geotechnical engineers and geologists. As far as highway geology goes, some of the traditional geology work may now be performed by geotechnical engineers, who may or may not have a geology background. The advent of readily available information on the internet and various software packages have made for some questionable practice.

There are geotechnical engineers without a single class in geology, writing geology sections of reports, geologic history and setting, and interpreting rock mass classifications. Geologic information such as maps and reports are readily available on the internet. The advance of technology. software and "apps" have made available a vast quantity of information and the ability to input data and export an answer to a problem. The trouble may be that the user may or may not know if that answer is within the realm of correct possibilities As more and more highway geology work is outsourced to "geotechnical consultants" the more it has a chance of being performed by a team without an experienced geologist. Request for proposals from agencies should favor qualifications of consultants with transportation engineering geologists on staff. Perhaps the scope of work should include the requirement of certain reports and documents be sealed by a licensed geologist in appropriate states. .

Engineering Geologists should remake their workplace, adapt to new practices and specify what work should be within their control.

The Engineering Geologist

The practice of highway geology has changed over the years due to multiple factors. Large Soils and Geology departments grew out of the construction of the Interstate High System during the 50's, 60's and 70's. This was the great development age for geologists as can be noted by the early founders of the Highway Geology Symposium.

Geologists have enjoyed many opportunities working with various engineering disciplines as part of the highway design team. Many prominent geologists have made their mark on highways as well as the Highway Geology Symposium.

There are differences between geologists and engineers, as there is substantially more room for interpretation when it comes to geology. Geology is part art and part science which can lead to various shades of gray. Earth materials are very different than engineered materials. Concrete and steel perform in a very predictable manner. Rock on the other hand, can be very different. Samples from the same formation, in very close proximity to each other can vary highly in character. It is up to the geologist to interpret and determine the actual rock quality and predicted behavior.

Functions of the Modern Engineering Geologist

Highway Planning

First and foremost, the geologist should be involved with the planning and location of a highway. Of course, this varies with the complexity of the geologic environment - a straight stretch of highway on relatively flat ground with constant geology does not require the same effort as one with complex geology, steep grades, complex rock cuts and deep fills.

The geologist should be engaged with the National Environmental Policy Act (NEPA) document. Signed by President Nixon in 1970, the act acknowledges and includes the study of impacts to the environment by Federally funded projects. The products of the NEPA process are the Draft Environmental Impact Statement (DEIS), Final Environmental Impact Statement (FEIS) and Record of Decision (ROD). Sections of the documents include studies of the natural environment including general geology, geologic hazards, groundwater, mineral resources, past and present mining and locations of hazardous materials. The geologist should be included in the research and writing of these documents.

Identify Geologic Hazards

The environmental document will usually include a study of highway alignment alternatives. The study, performed by a variety of disciplines, analyze the positive and negative effects of each alignment in order to select the preferred alternative.

The geologist should be included in the analysis of alternatives and consider items including but not limited to the following;

- landslides,

- areas of instability,
- sinkholes, karst,
- groundwater,
- location of aggregate,
- ground favorable for excessive settlement,
- unfavorable rock orientation,
- deposits of unsuitable material,
- location of mines and abandoned mines.

If many of these problems are identified in the study phase, appropriate measures can be undertaken during design and construction and long term maintenance problems avoided.

Explorations

Explorations are perhaps the biggest responsibility of the geologist, and starts with a literature search and continues with a study of available maps, aerial photographs and satellite imagery. Based on the preliminary data, the soils type including glacial, alluvium, colluvium, and residuum can be identified as each has its own general behavior and distribution.

The second step includes an exploration plan by test pit, auger borings or coring. The geologist should be involved with the directing the drilling and sampling, examination of samples and logging of the subsurface materials. The geologist should complete the field and final logs and stratify the subsurface information from the borings. The geologist should also select the samples for further laboratory testing.

Aggregates – Sources and Quality

A large percentage of the cost of highway construction is aggregate for structural concrete, paving and drainage. Modern mix designs demand a durable and available aggregate. Sources of suitable aggregates especially near in urban area can be a challenge since a majority of the cost is transportation. It is important to have a geologist perform explorations and locate suitable reliable sources.

Classification of Roadway Excavation and Material Use

The application of geology in cases of classification of roadway materials has been diminished by the use of "unclassified excavation". A proper understanding of the geologic setting of the site is necessary to determine general uses and performance of soil, especially in subgrades and slopes. Traditionally, the geologist, in association with the highway engineer performed extensive investigations to identify the type and extent of soil and rock material expected to be encountered on the project. Materials, their location, suitability and uses in the grading were identified and quantified.

The location, type and character of the bedrock to be excavated was identified along with properties that enabled the bidding contractors to better estimate costs. Also important is the ability to identify and analyze the bedding, jointing and orientation of rock discontinuities. The analysis of the discontinuities is very important in designing a stable slope without long term maintenance problems.

For igneous rocks, that includes degree of weathering, depth of weathering, spacing of joints and hardness. For sedimentary rocks a geologist familiar with the formation can safely predict the difficulty of excavation as well as proven backslope design. It was also possible for both the agency and the contractor to estimate if the rock could be excavated without blasting or the type of blasting that would be anticipated.

This type of characterization seems to have ended in an attempt at avoiding contractor claims. The use of unclassified excavation now places all the risk on the contractor, eliminating the practical usefulness of the characterization, and quantification of the earth materials. The use of unclassified excavation also seems to be used unilaterally without the analysis of project complexity, project costs or project risks. The tradeoff in passing the risk to the contractor, in the author's opinion is higher contract costs of excavation. Contractors now do not have the benefit of understanding the geology as interpreted by the geologist and are left to their own interpretation. Hence, the risk the contractor takes is passed along as higher costs, while never completely eliminating the risk of a claim.

Landslides

Highway construction may contribute or initiate a change in conditions that upset temporary stability. The solution often lies in properly diagnosing the causes and conditions that created the slide. Slides may be caused by removal of a toe of a slope, by loading with a fill, by change of drainage, or by vibrations such as blasting or removal of vegetation.

Two general types of landslides are most commonly encountered in highway construction, changes in the groundwater condition of the presence of weak layer such as clay or bentonite that softens when wet. Detailed geology of the area of the landslide should be investigated, stratification and orientation of rock discontinuities should be identified. A dip toward the highway cut can be disastrous.

Groundwater

The highway geologist should also have an understanding of groundwater and the role it will play. Water in the subgrade can lead to a variety of major pavement problems. Water in slopes can lead to instability

Bridge Foundations

Usually the geologist isn't given much choice, and has to fit the bridge and foundations in the given location. The geological history or the valley and conditions at the site are therefore important. A geologist can help to predict the depth and character of the sediments overlying the bedrock. They can also predict the character of the bedrock and assist in the selection of preliminary foundation types. Basic steps of an investigation of the geology of a site may enable the designers to avoid future of trouble and maintenance.

P

Post Design – Construction Services

The geologist is often engaged in post design services. Many resident inspection opportunities are available for the following items

- Grading Inspection – Control and acceptance of general grading,
- Rock slopes, rock reinforcing, rock fall remediation, underdrains.
- Bridge Foundations
    - Inspection of spread footing and confirmation of material and bearing values
    - Large excavations, mapping and evaluation of rock quality and discontinuities. Repair recommendations for deficiencies
    - Inspection of Drilled Shafts and log of excavated material, confirmation of bearing material and bearing values assigned with the design.
    - Inspection of driven piling – confirm pile capacity and bearing material.
    - Micropiles – confirm bearing material and capacity
    - Retaining walls – confirmation of bearing material
    - Ground improvements – inspection of soil mixing, geopiers, wick drains, etc.

Summation

Geologists should be enlisted during various phases of transportation planning, design and construction. Geologists working alongside engineers have produced the best solutions for the travelling public.

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New Design Software for Rockfall Simple Drapery Systems

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ABSTRACT

Rockfall drapery systems are commonly used as simple, fast and economical measures to control rockfall trajectories on very steep slopes. The systems basically consist of a steel mesh attached at the slope crest with a longitudinal cable fixed by means of a suitable number of ground anchors. The effect of this kind of intervention is to control the trajectory of falling rocks, which then fall to the bottom of the slope with a with slower velocity, or are stabilized in place. They can be used on any kind of slope to protect sensitive targets in the mining industry, roads and railways, and inhabited areas.

The falling blocks, typically smaller than 0.6 - 1.0 m in diameter, pile up into a trench (or into a "pocket of mesh") at the bottom. In comparison to other types of rockfall protection measures, the simple drapery is cheaper, and its maintenance is easier. On the other hand, it cannot be considered a remedy for shallow instability because it can only control the trajectories of falling rocks and facilitate their collection at the slope toe.

The design of simple draperies requires the analysis of several factors such as slope features (height, gradient, morphology), the geological and dynamic features (nature of the ground or rock, type of instability, erosion problem, blocks size), the environmental condition (presence of vegetation, aesthetic concerns), the installation problem (access to the slope, safety for the workers, safety for the surrounding areas) and finally the performance required (temporary or permanent intervention, required maintenance, cost). Finally the most problematic design-step is the choice of a suitable mesh, the top longitudinal cable, and the top anchor type. Because of the highly variable nature of rockfall behavior, these structures cannot be standardized - they have to be analyzed and designed for each application.

Maccaferri has developed a new software application (MacRO 2) with a practical tool to define the mesh and the related supporting structure consisting of up-slope cables and anchors. The software, based on an approach proposed by Muhunthan B. et al. (2005), allows designers to size the top longitudinal cable, the anchors, and select the appropriate mesh drapery and establish for maintenance procedures. Even if the method seems quite simple and rough, it is effective and lets the designer correctly select drape materials and the geometry to be used on the systems. This paper analyzes the conditions for a simple drapery installation, the main steps used for the calculations, and presents a case study at a Mine in the U.S.. Nevertheless, even if the software allows for a quick and simple calculation approach, onsite observations are always recommended in order to achieve a good design, with the ultimate goal of protecting property and human lives.

KEYWORDS

Rockfall, rockfall protection system, design, mesh, drapery system, MacRO 2, Rock Mesh HR.

INTRODUCTION

The natural processes of weathering, increased by climate change, generate geological instabilities, which frequently expose populated areas and infrastructures to a wide range of shallow instabilities varying from erosion to rockfalls. Shallow instabilities should not be underestimated because they frequently cause rockfall events. Due to the fact that they happen with a high frequency over large areas, the probability of rock strikes and accidents is elevated. In this situation, the design must consider the efficiency of a remedial solution in terms of performance and low maintenance costs. The rockfall mitigation solution is divided into two different design approaches related to their means of stabilizing the slope area:

Active Protection Systems: are applied directly on an unstable zone in order to prevent or control the movement of the shallow instability. The most common solutions inside this category are the following:

- *Soil Nailing*: is to improve soil stability by inserting reinforcement bars in the soil in a regular pattern, the nails are then grouted and fixed soundly to the ground for their entire length (nailing). The frequency and the length of the nails can be calculated in accordance with FHWA, EN 1997 1 or BS 8006. The ground surface is reinforced with a structural facing which can be flexible (steel mesh) or rigid (shotcrete)
- *Pre-stressed Soil Anchors (tie back anchors)*: pre-stressed anchors are installed in a shallow instability to modify the internal stability since an external force is applied to tie the instability into the slope area.
- *Secured Drapery System*: composed of an anchor system spaced at regular intervals where the rocks are held in place by a surficial structural, flexible (steel mesh), or rigid (shotcrete) facing interconnected to ground anchors.

Passive Protection Systems: are not implemented at the source area, but rather mitigate the hazard of instabilities by affecting the trajectories of falling rocks or arresting or reducing the falling rock velocities. They are generally applied far from rockfall source areas. This category includes the following solutions:

- *Simple Drapery System*: consisting of a steel mesh drape system, secured at the top of the slope with ground anchors and steel wire rope cables.
- *Rockfall or Debris Flow protection Barriers*: structures composed of posts, cables, energy dissipaters and interception structures (steel or wire mesh) capable of arresting and containing falling rocks. The barrier is also composed of elements to anchor support cables, post foundations, and ground anchors;
- *Hybrid Barriers*: structures composed of posts, cables, energy dissipaters, and a tail of mesh designed to reduce the energy and the velocity of falling

rocks which are driven into the slope by a steel drape system reducing energy through ground collisions; and

- *Rockfall or Debris Embankments*: a gravity or mechanically stabilized earthfill structure forming a steep berm to contain falling rocks or debris, generally installed at the toe of a slope

Simple drapery system

A simple drapery system consists of a rockfall steel mesh installed along the face of the slope. As mentioned before, the drapery is hung as a curtain (figure 1), suspended by longitudinal ropes and anchors at the crest (Rc). Anchors are positioned along the crest (A_C) and toe (A_T) of the slope and their distance depends on the design and the prevailing rockfall conditions at the site. They are commonly located in a line and are fitted with suitable connections (often eye nuts, or plates, or similar) to accept the crest rope (R_C). Once the crest anchors and the upper longitudinal cables are installed, the mesh can be fixed to them and left free-hanging all along the slope.

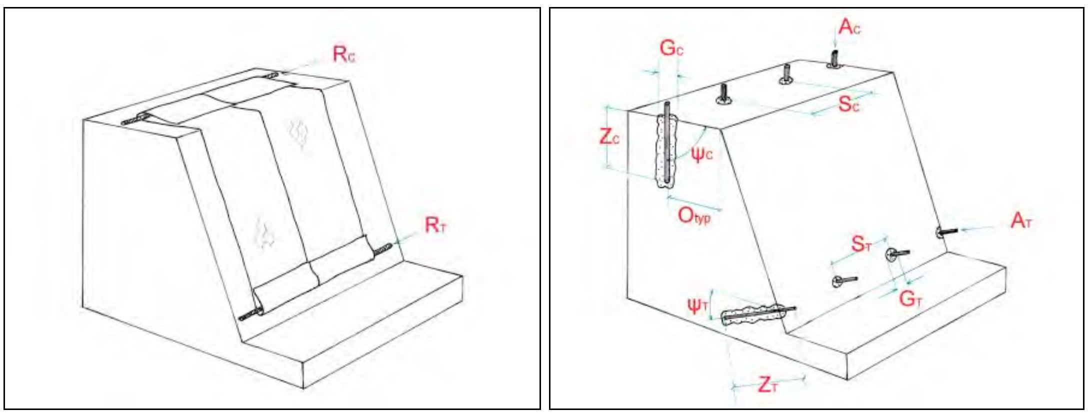

Figure 1 – Scketch of a simple drapery system application (left) and disposition of the anchors (right)



Figure 2 (left) – Mesh installation at the crest (left)
Figure 3 (centre) – Debris accumulation at the toe of the simple drapery system
Figure 4 (right) – Reunion Island (Fra) – more than 40,000 sqm of drapery system were installed in a rocky slope higher than 150 m

The steel mesh can be fixed as well at the bottom where runout space is limited, so that the falling debris can pile up into a pocket (figure 3). In order to reduce the stress on the mesh and reduce the costs as well, the mesh at the toe of the slope can be unsecured; in this case, a catchment trench or a fence is required to collect the fallen debris. This type of system is usually installed on a large rocky slope (figure 4), where the secured drapery systems are not cost-effective, or where the rockfall barriers and rockfall embankments cannot be installed because the slope morphology is either too uneven or too steep.

DESIGN: PRELIMINARY REMARKS

In order to design the most cost effective and suitable mesh system, the designers must first analyze the main factors affecting the effectiveness of the mesh.

First of all, the stress applied on the mesh and the performance of the simple drapery system largely depends on the slope morphology. For example, for a very uneven slope, the drapery system may only be in contact with the slope on the crest area and convexities, whereas the debris can freely run down into the gullies and concavities. In this situation the drapery has a negligible capacity to control erosion, and the falling rocks can reach higher velocities. The installation of the drapery then requires particular care to maximize the contact between the ground and the steel mesh, or the slope must be preventively re-shaped and scaled.

Another important factor affecting the selection of the mesh is the existing rock slope instability. If erosion is the main problem, typically on a gentle slope, the appropriate selection of drape system should have a small mesh opening and enough weight to maintain constant pressure on the ground surface. When there is contact between the mesh and the ground, the drapery is quite effective in erosion control and allows both the re-growth of the vegetation and the confinement of large boulders. If the slope is vertical, the drapery must be stronger to absorb impacts and funnel falling debris to the toe of the slope. In cases of large blocks (i.e. in the basalt cliffs), a “dynamic” drapery, like cable panels or ring nets, should be considered, whereas in cases of small blocks (i.e. thin layered limestone cliff) lighter draperies, like steel composite Rock Mesh or double twist wire mesh could be suitable.

Other important design factors are the expected life span of the drapery and its maintenance costs. Concerning the life span, designers should consider exposure to atmospheric conditions (i.e. salt spray or wind), and abrasion due to movement of falling debris. If the drapery is applied for temporary protection, as in the mining industry, a light corrosion protection could be enough. If the application must be permanent or it is close to aggressive environments (i.e. seaside), a stronger corrosion protection is required. In the last case, the designer has to plan for maintenance suggesting the maximum size of the debris pocket acceptable for the mesh.

MACRO 2: CALULATION APPROACH

The design of simple drapery depends on different variables related to the geometry of the slope, the type of the mesh, and the assumed debris accumulation at the base of the system. One of the available references to give as a design guideline for these applications was prepared by the Washington State Department of Transportation (Muhunthan et al. 2005).

Using this study and the results obtained from several laboratory and field tests, Maccaferri has developed new software (MacRO 2) able to perform stability analysis for the selected mesh, the diameter of the crest wire rope cable and the steel and geometric (diameter and length) characteristics of the crest anchors. If time and money are not a problem, a complex numerical analysis with very precise data from the field could be completed, but this is not practical for every project, especially if the system has a modest size and has to be done in a short period of time (emergency protection). MacRO 2 allows designers to have a quick and reliable solution for design. The design procedure that is the basis of the software is simple, but it gives reliable results considering the low level of accuracy generally available from the input data.

Mesh design

The simple drapery system is a passive system capable of controlling rockfalls and containing the debris at the bottom of the slope. It is designed considering all the different components able to transmit loads on the mesh per linear of slope section:

1) The proper weight of the selected mesh
2) The weight of the debris accumulated at the toe of the slope
3) External weight like the snow or ice accumulation on the drapery

These three loads may be described with the following formulas, based on the research from the U.S. Department of Transportation FHWA (note: formulas 1, 2 and 4 are multiplied by unit length for simplification).Total load due to the mesh (W_m) has to be defined:

$$W_m = \gamma_m H_s / \sin\beta \, (\sin\beta - \cos\beta \tan\delta) \, g \tag{1}$$

Where:
- γ_m = steel mesh unit weight
- H_s =total height of the slope
- β = inclination of the slope
- δ = friction angle between mesh and slope
- g = acceleration of gravity

It is possible to identify the load transmitted from the debris to the mesh (W_d) as follows:

$$W_d = \frac{1}{2} \gamma_d H_d^2 (1/\tan B_d - 1/\tan\beta)(\sin\beta - \cos\beta \tan\varphi_d) g \quad (2)$$

Where:

- γ_d = debris unit weight
- H_d =debris accumulation height
- φ_d = debris friction angle
- B_d = debris external inclination value (Muhunthan equation) :

$$B_d = \arctan[H_d / (T_d + H_d / \tan\beta)] \quad (3)$$

- T_d = debris accumulation width

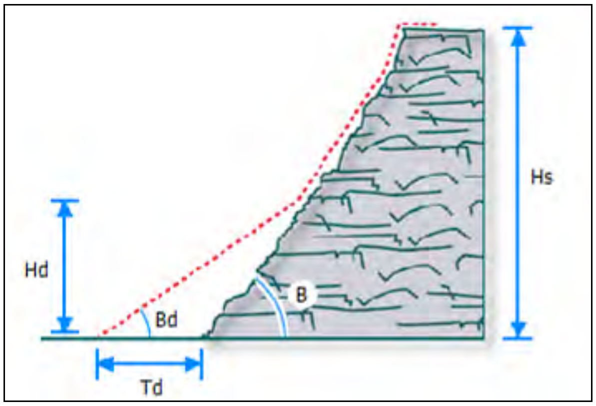

Figure 5 – Geometrical input data to calculate the load on the mesh due to the debris accumulation

The last load acting on the mesh is due to the snow thickness above the mesh (W_s). It is considered that for slope with an inclination (β) higher than 60 degrees this load is neglected since the snow cannot be accumulated.

$$W_s = \gamma_s t_s Hs / \sin\beta (\sin\beta - \cos\beta \tan\varphi_s) g \quad (4)$$

Where:

∃ γ_s = snow unit weight
∃ t_s = snow thickness
∃ φ_s = friction angle between soil and snow

To design the drapery system at a limit equilibrium state, three safety factors have to be introduced in the calculation to increase the acting forces and decrease the resisting one:

Safety factor reducing resisting forces:

∃ γ_{mts} = safety coefficient which reduces the tensile strength of the mesh (≥ 1.0; from the in-situ and laboratory tests, this factor would not be lower than 2.0)

Safety factors increasing acting forces:
∃ γ_{vl} = safety coefficient for the variable loads, like the snow thickness and the debris accumulation (≥ 1.0; suggested value according to the Euro Code = 1.5)
∃ γ_{pl} = safety coefficient for the permanent loads, like the drapery (≥ 1.0; suggested value according to the Euro Code = 1.3)

The acting and resisting forces at the limit equilibrium state can be calculated introducing the partial safety factor coefficients listed above:

The total stress on the drape (S) will be:

$$S_w = (W_d + W_s)\,\gamma_{vl} + W_m\,\gamma_{pl} \tag{5}$$

The serviceability tensile strength of the mesh (R_m) is calculated as:

$$R_m = T_m / \gamma_{mts} \tag{6}$$

Where:
T_m = ultimate longitudinal tensile strength of the mesh (defined by laboratory tests)

The design is satisfied if:

$$R_m - S_w \geq 0 \tag{7}$$

Thus, the safety coefficient of the mesh equals:

$$FS_{mesh} = R_m / S_w \geq 1 \tag{7.a}$$

Cable design

The mesh is secured on the crest with a wire rope cable connected to ground anchors. To design the wire rope cable, the maximum load acting on the drapery (defined above) and the spacing between the crest anchors is used to calculate the deformation and the stress distribution within the rope. This method uses the principle of the catenary loading to verify that the tensile strength of the cable is sufficient to support the total weight of the system: $W_m + W_d + W_s$.

The cable is verified if the following equation is satisfied:

$$T_{wlc} - F_{cbl} \geq 0 \quad (8)$$

Where:

∃ T_{wlc} = cable working load limit [MLT^{-2}]:

$$T_{wlc} = T_{cbl} / \gamma_{cbl} \quad (9)$$

∃ T_{cbl} = ultimate tensile strength of the designed rope (varies with steel grade, wire rope construction and the diameter)

∃ γ_{cbl} = safety coefficient decreasing T_{cbl} ($\geq$ 1.0)

∃ F_{cbl} = max tensile strength acting on the cable (calculated with catenary theory)

Thus, the safety coefficient of the cable is:

$$FS_{cable} = T_{wlc} / F_{cbl} \geq 1 \quad (8.a)$$

Moreover, using this theory it is possible to define the maximum length of the rope and its maximum sag between two anchors.

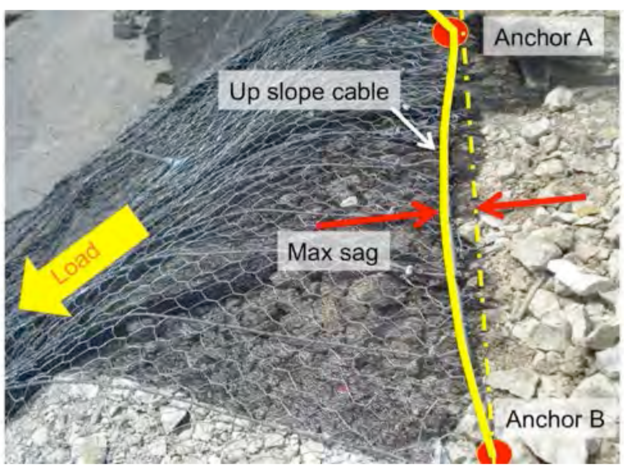

Figure 6 – Example of the deformation of the crest wire rope cable between two anchors (A and B) calculated by the Catenary theory

∃ T_m = ultimate longitudinal tensile strength of the mesh (defined by laboratory tests)

The design is satisfied if:

$$R_m - S_w \geq 0 \quad (7)$$

Thus, the safety coefficient of the mesh is equal to:

$$FS_{mesh} = R_m / S_w \geq 1 \qquad (7.a)$$

Anchors design

Anchor's design may be divided into 2 different steps:

1. The first step is for designing the anchor diameter taking into consideration the shared load transmitted from the system, composed of the mesh + wire rope cable
2. The second is designing the minimum anchor length, which depends on soil or rock characteristics

Evaluation of the anchor diameter

Using catenary theory it is possible to determinate the maximum force acting on the intermediate and lateral anchors. These two forces have to be related to the working capacity of the designed anchors:

$$S_{bar}(j) - N(j) \geq 0 \qquad (10)$$

Where:

∃ $S_{bar}(j)$ = working shear resistance of the anchor j :

$$S_{bar}(j) = (Y_{bar}(j) / \gamma_{st})\, 3^{-1/2} \qquad (11)$$

∃ $Y_{bar}(j)$ = yield load of the steel bar j :

$$Y_{bar}(j) = ESS(j)\, \sigma_{adm}(j) \qquad (12)$$

∃ ESS(j) = effective area of the steel bar j [L^2]:

$$ESS(j) = \pi / 4 \{[fe(j) - 2\, fc(j)]^2 - fi(j)^2\} \qquad (13)$$

∃ $\sigma_{adm}(j)$ = yield stress of the steel of the bar j

∃ fe(j) = external diameter of the steel bar j

∃ fc(j) = thickness of corrosion on the external crown of the steel bar j

∃ fi(j) = internal diameter of the steel bar j

∃ γ_{st} = safety coefficient for the steel strength of the bar (> 1.0)

∃ N(j) = force that the cable and the mesh develop on the anchor j (calculated with the catenary solution)

∃ j = position of the anchor: intermediate or lateral

Thus, the safety coefficient of the different cable may be calculated as follows:

$$FS_{anchor}(j) = S_{bar}(j) / N(j) \geq 1 \qquad (10.a)$$

Evaluation of anchor length

The evaluation of the anchor length takes into account the following:
1. The anchor plays an important role because it has to support the entire system. Its length must be deep enough to reach the stable section
2. The steel bar and the grout are exposed to weathering influences (ice, rain, salinity, temperature variations, etc.)

The minimum theoretical length is derived by the equation:

$$Lt(j) = Ls(j) + Lp \qquad (14)$$

Assuming:
Ls = minimum foundation length [L]:

$$Ls = P / (\pi \, \phi_{drill} \; \tau_{lim} / \gamma_{gt}) \qquad (15)$$

Lp = length of hole with plasticity phenomena in firm part of the rock mass

With
- ϕ_{drill} = diameter of the drill-hole
- τ_{lim} = adherence tension between grout and rock
- γ_{gt} = safety coefficient of the adhesion grout – rock
- P = maximum pullout forces depending on the cable load (Figure 7)

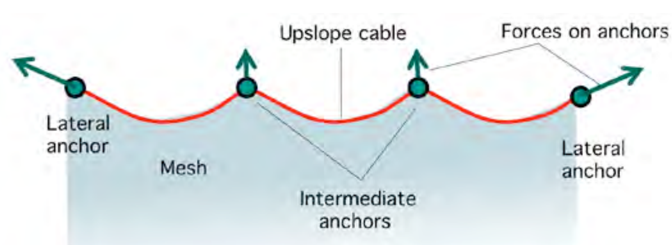

Figure 7 – Scheme of the load on up slope anchors

The minimum length of the anchor that was determined by the formulas will need to be verified onsite. The final suitable length of the bars has to be evaluated during drilling in order to verify the exact nature of the soil and be confirmed with pull out tests.

TYPE OF MESH

Today, the market offers a wide portfolio of rockfall draperies: single twisted or double twisted wire meshes, steel composites mesh with wire rope cables and wires, cable meshes, cable panels and ring panels. To define the drapery to be used, the designer should take into account different aspects of the material:

- No unraveling phenomena if a part of the mesh is cut (i.e. a wire, a cable or a connection element): single twist mesh should be rejected;
- Resistant to dynamic impact: ring nets or cable panel are the most suitable;
- High tensile resistance: it depends on the input parameters, but generally no lower than 50 kN/m;
- Capacity to transfer the load to the anchors: meshes with vertical support rope included are the most appropriate;
- Easy installation.

The following table summarizes the main meshes available on the marked giving also the principals characteristics.

Table 1 – Rockfall mesh features – Note: (*) Value from literature; (**) Average value defined by Officine Maccaferri Full Scale Test carried out in Fonzaso (BL – Ita) on a sample 2.0x2.5m (6.6 ft x 8 ft), restrained on 4 sides; (***) supposed values

Type of Mesh	Longitudinal Tensile Resistance (*)	Dynamic Resistance (**)	Unraveling when one or more wire/cable fail	Photo
Double twist mesh	Up to 100 kN/m (6,854 lb/ft) (usually 60 kN/m) (4,112 lb/ft)	Up to 15 kJ (3.69 ft-ton) (usually < 10 kJ)	No	
Single twist mesh	Up to 150 kN/m (10,281 lb/ft)	< 10 kJ (3.69 ft-ton)	Yes	
Steel composite: cables woven in a double twist mesh	Up to 180 kN/m (12,337 lb/ft)	15 to 20 kJ (5.53 – 7.38 ft-ton)	No	

Cable panel	Up to 250 kN/m (17,135 lb/ft)	20 kJ (7.38 ft-ton)	No	
Ring net	Up to 350 kN/m (23,989 lb/ft)	> 50 kJ [***] (18.45 ft-ton)	No	

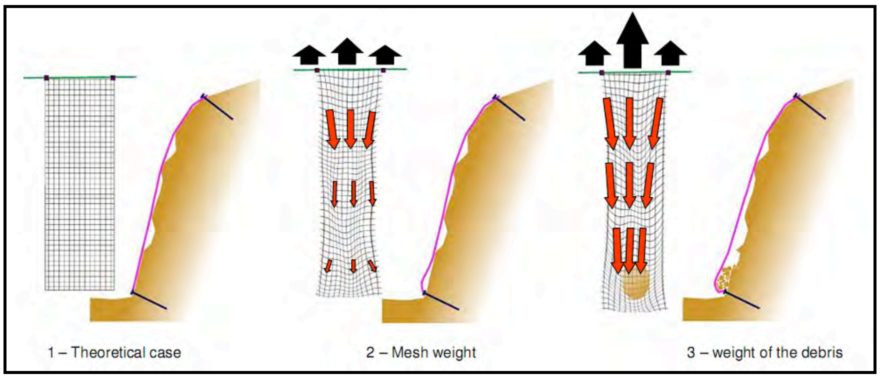

Figure 8 – comparison between the theoretical and the real case after the mesh installation

From figure 8, it is highlighted that the higher stress is on the mesh (black arrow), which deforms transversally (narrow neck) and stretches longitudinally (elongation). Using the Rock Mesh composite, the forces acting at the bottom of the system are directly transferred to the interwoven cables which reduce the load on the mesh increasing the reaction on the top anchors.

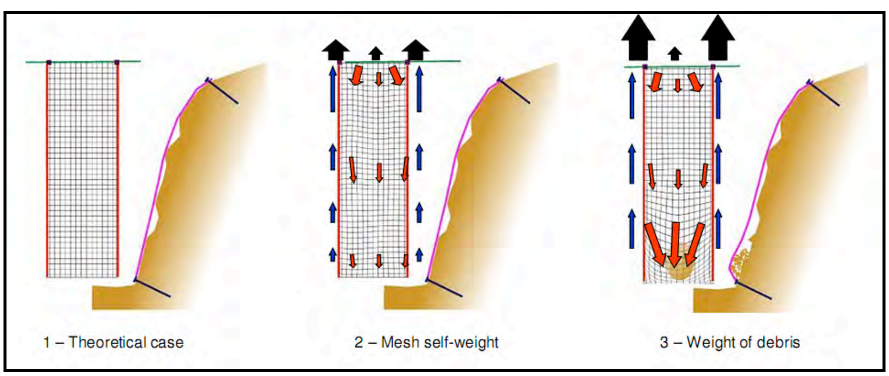

Figure 9 – comparison between the theoretical and the real case after the installation of the Rock Mesh HR"

The previous figure illustrates that Rock Mesh allows higher loads to be supported by the drapery system with less deformation on the mesh and lower loads on the crest line ropes due to the direct connection of the load supporting integrally woven steel ropes to the crest line anchorages.

CASE HISTORY

Simple Drapery Systems are commonly used to protect workers, building and infrastructure from rock fall hazards. This case study will describe a typical application of this mitigation measure in an urban area, Peerless Park, Missouri. The solution was designed to control rockfalls with a system that would be easy to install, economic and low maintenance, and of course reliable. The design was determined considering 2 main aspects: the visual impact needed to be kept at a minimum and the selected system had to have low maintenance requirements.

Figure 10 – General overview of the protected slope

The height of the slope was around 20 m (65 ft) and the falling block size was supposed to be from 15 to 30 cm (6 to 12 inches). A simple drapery system was the best solution with the toe of the upper section secured in this case because:

∃ The very limited room at the toe of the slope was not suitable for a rockfall barrier;

moreover from a numerical simulation the trajectories were too high to be intercepted by a standard fence due to the limited catchment area;

∃ No room for a rockfall embankment;

∃ The cost of a secured drapery was too high in relation to the saving of the maintenance costs.

More than 1,500 m^2 (16, 2000 ft2) of Rock Mesh M4000 was installed to protect the car wash workers and their patrons. The bluff is separated into two tiers and measured approximately 55 m (180 ft) in length and a total height of 27 m (90 ft). The installation of the mesh was separated into an upper and lower section to blend better with the surroundings, regardless of its expense, rather than running the mesh from top to bottom. The rock mesh drapery systems were installed on each tier. The rock anchors were drilled 6’ in depth at 24’ centers, the length of the bluff, and grouted in place.

Rock Mesh is a woven composite product made of steel ropes and a double twist wire mesh woven together during the manufacturing process. The metallic cables are used in place of the conventional selvedge wire to increase the connection strength and to transmit the load of the debris directly to the top-anchors in order to reduce the stress and the deformations of the mesh. The hexagonal double twist mesh provides high resistance to the impacts of rocks, avoiding the unraveling in the event of wire breakage.

Figure 11 – Detail of the cable of the Rock Mesh and the secured toe of the upper section.

For the upper and lower section, two different design approaches were used. For the upper section, the toe of the drapery was secured to contain the rocks within the mesh and to avoid rocks falling over the lower section where the lower section the mesh was not secured at the toe. According to MacRO 2 calculation, the upper section had an extra load on the mesh due to the rock containment compare to the lower section that was not secured and free to fall into a catchment area.

CONCLUSION

Simple drapery is an effective rockfall protection system for rock slopes. This type of solution is economical, easy to install, and has a low level of maintenance. It is recommended in areas where other mitigation systems (i.e. pinned drapery or rockfall barriers) cannot be applied because their cost and/or the morphology of the site are not suitable.

Based on the researches done by Muhunthan et al. 2005 and the in-situ and laboratory tests, Maccaferri has developed a calculation approach (MacRO 2) able to design all the components of the drapery system, such as the mesh, crest cable and support anchors.

The latest advancements in mesh, marked “Rock Mesh”, is a new concept of mesh to be used as a simple drapery system in order to reduce the stress acting on the mesh, and consequently the maintenance costs, even if the amount of debris volume potentially accumulated at the base of the slope is larger.

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Maud Farm Road Investigation

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Abstract.

This paper presents the findings of an extensive site investigation into the causes of embankment settlement over five flexible pipe culverts ranging respectively in 36, 24, 36, 36, and twin 48 inch diameter. The site location is on an east−west County Road 131 near Maud, Oklahoma. The project was developed and designed by the Bureau of Indian Affairs (BIA). The issue here concerned a lawsuit brought by the BIA against the contractor in which the BIA wanted to know if the contractor could be held libel for the embankment settlement. At stake was contact retainer held by the BIA in the sum of $358,000 against the contactor.

The site geology consists of very shallow alluvial soils and/ or residual soil underlain predominately sandstone and sandstone and interbedded shale in the narrow drainways. The embankment was constructed from roadway cut sections containing residual sandy and clayey soils underlain by sandstones and sandstone interbedded with shale. The site landscape is one of shallow rolling hills.

The field investigation consisted of a total of 15 piezocone soundings at the site. Soil properties of the embankment material, the underlying shallow alluvial and/or residual soil, and underlying geology were inferred from the piezocone tip resistance (q_c) and friction ratio (R_f). Three piezocone soundings were made in a staggered pattern at each of the five pipe locations in as close a proximity to the pipe centerline as possible.

The analysis used software for the analysis of buried structures, Cande−2007 Update Release 7/31/2011, Version 1.0.0.7. This software uses a finite element mesh analysis. A detailed analysis revealed that the settlement at each pipe location was due to deformation below the pipe grades. The piezocone tip resistances in the embankment indicated a very stiff material and did not support the BIA claim that the contractor was responsible for the subsidence above the pipe culverts. The analysis showed that the settlements were the result of vertical pressure against a yielding base, a concept borrowed from theoretical soil mechanics.

Introduction

The paper presents the findings of an extensive site investigation into the causes of embankment settlement over five flexible corrugated pipe culverts ranging from 24 to 48 inch diameter. The paper will discuss in detail the first pipe investigated, structure number 6. The site location starts at the intersection of SH9A and extends east approximately two miles to the intersection of E−W 131 and N−S 353 southeast of town of Maud in Seminole County, Oklahoma, see Figure 1.

The request for this investigation was made by the Bureau of Indian Affairs (BIA) for the Eastern Oklahoma Region located in Muskogee, Oklahoma. The request was initiated by a telephone call on December 14, 2011from Mike Ollar, the BIA Construction Engineer, who explained the general scope of investigation. The person of contact with the BIA during the course of this study with the project details was Kirk Carson, BIA Design Branch Chief.

The project was developed and designed by the Bureau of Indian Affairs (BIA). The BIA uses the Oklahoma Department of Transportation (ODOT) design standards and the specifications for highway construction in their roadway projects.

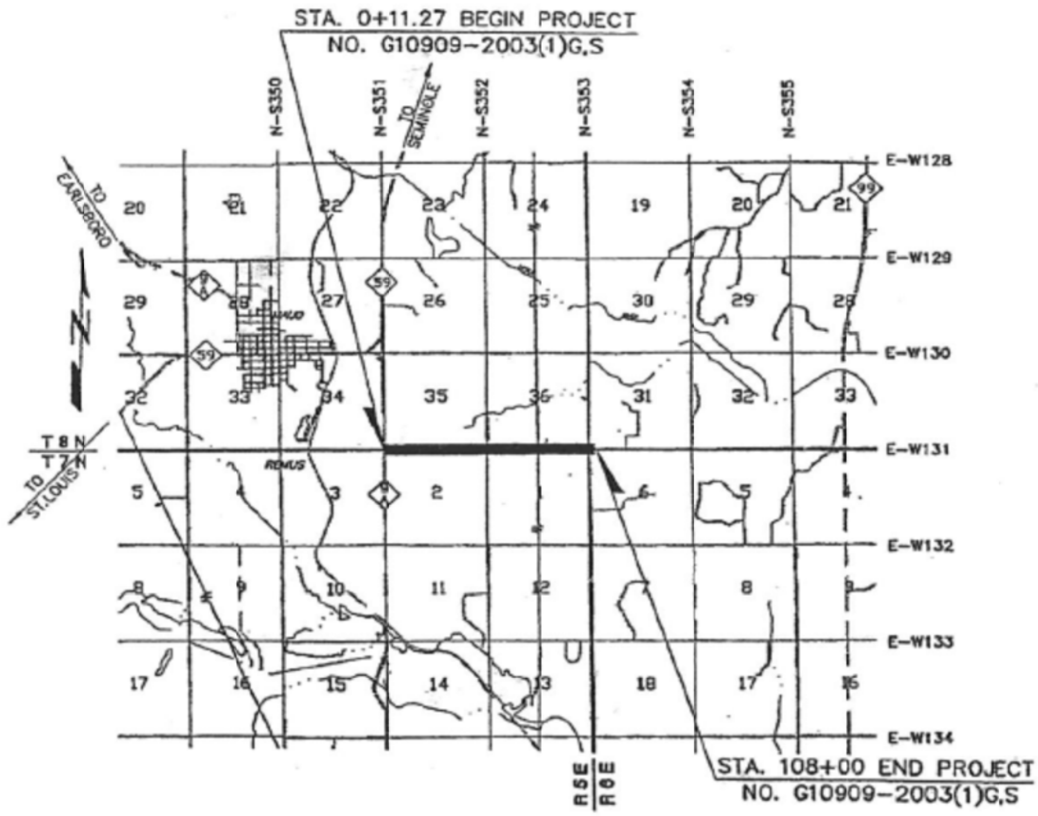


Figure 1. Site location.

The issue here concerned a lawsuit brought by the BIA against the contactor. The reason for the investigation was that the BIA wanted to know if they could negotiate a settlement with the contractor to keep the contract retainer, a sum of $358,000 dollars, held by the BIA to offset their anticipated repair costs of the settlements over the pipe culverts. The BIA's intent was to show that the contractor was negligent in the compaction of the earthwork, and thereby through his negligence the settlement problems in the embankment occurred. The settlements were significant enough having caused several accidents with one resulting in personal injury and other complaints concerning a rough ride to be lodged with the BIA.

Site and Subsurface Condition

The new E−W 131 alignment crosses over a shallow hill and rolling topography with a 24 foot wide asphalt concrete pavement and 2 foot wide asphalt concrete shoulder in cut and fill grading sections. The typical section for the pavement and side slopes is presented in Figure 2.
The project was started in July 2005 and completed in May 2007. Pipe structure number 6 with a respective description of (152 LF 36   Skew 30^{o}) is reviewed in this paper. It was learned late in the investigation that for pipe structure 6 that the skew had to be increased based on a field decision due to a drainage way channel change. This field change necessitated an increase in the pipe length of 30 feet, and a field splice consisted of a corrugated band being placed over a torch

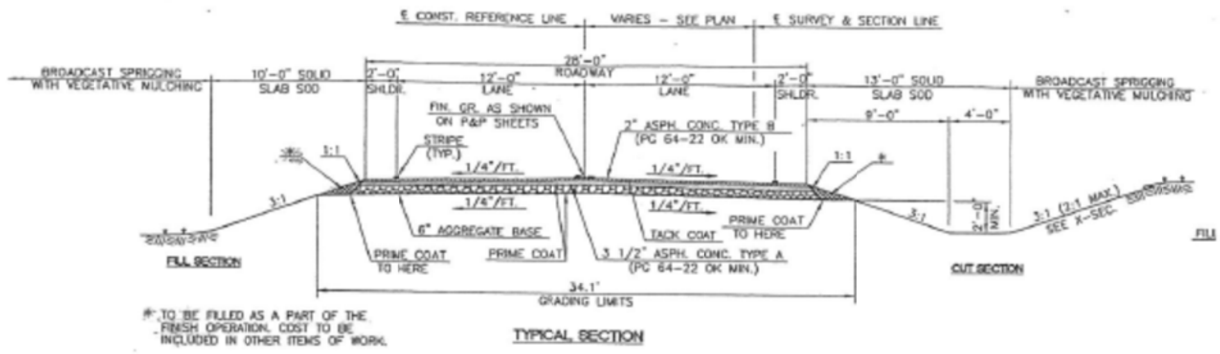


Figure 2. Typical pavement section for the E–W 131 alignment.

cut joint between the two pipe lengths. This change was made on the south end of the pipe requiring a cut into the back slope at station 22+49 with the pipe trench and compaction thereof not in close compliance with the ODOT construction requirements. The location of this pipe extension is approximately from the middle of the embankment side slope to the pipe inlet. There was a collapse of the flexible corrugated pipe culvert within the pipe extension, see Figure 3. The setting for the site conditions at the project site and approximate location of structure number 6 is shown in the photograph in Figure 4

Soils and Geologic Statement

Surface Soils

The soil series underlying pipe structure number 6 is the Grainola–Lucien complex according to the Web Soil Survey 2.3 (1). The Grainola soils in this complex make up 60 percent of the map unit, and the Grainola soil series is a residual clay soil developed from shale bedrock. The Lucien soils make up 25 percent of this complex, and the Lucien soil series is a shallow residual soil that develops from sandstone bedrock.

Geology

The Seminole County Soil Survey (March 1979) (2) does not provide any geologic maps or geologic description of Seminole County and lists only the underlying geologic descriptions associated with the listed soil series.

According to the (ODOT) Division Three, (Red Book) (3), the underlying geology for this site location is the Vanoss Unit (IPv) of Permian geologic age. This unit consists of alternating moderately soft to moderately hard sandstones, conglomerates, shales, and a few thin limestones. The shales are multicolored, soft to moderately hard. The sandstones and conglomerates in the middle of Seminole County are thicker and locally are so arkosic that at first glance they resemble granite. The total thickness of the Vanoss Unit is irregular in Seminole County and varies from 140 to 500 feet. The unit outcrops in a two to seven mile–wide, north–south band in eastern edge of Pottawatomie and across western Seminole Counties.

According to the Hydrological Atlas 4 (4) the geology at this site location is the Vanoss formation (IPv) and consists of alternating layers of red–brown to

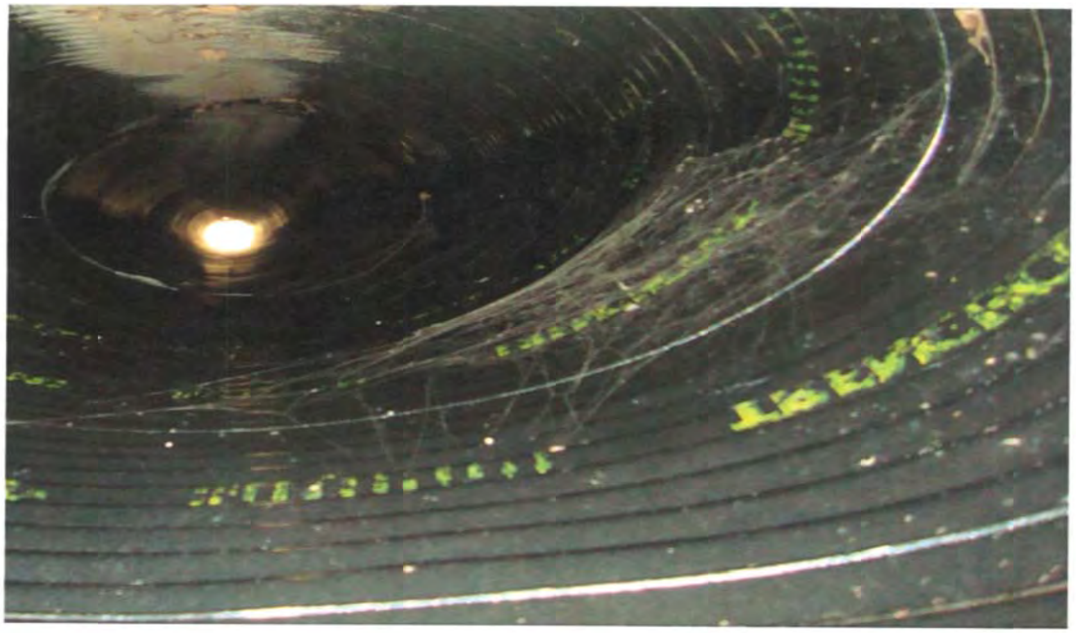


Figure 3. Collapsed portion of structure number 6.

Figure 4. Site of structure number 6 looking East in the drainage way.

gray shale and orange–brown fine–grained , cross bedded sandstone. The formation grades southward into arkosic sandstone and conglomerate.

The Oklahoma Geological Survey has a definitive current publication in Bulletin 74 (5). The geologic description presented above in the ODOT Division Three, 1968 (Red Book) above was taken from the Oklahoma Geological Survey Bulletin 74.

Field Investigation

The field investigation consisted of a site inspection, a level survey of the pipe location, and three piezocone soundings at the pipe location. At pipe structure number 6 location, an elevation level survey was made along the center line and the right and left white strip lines at a two foot spacing. The elevation level profiles were designated as the following: a.) Profile A at the right strip line, b.) Profile B at the center line of survey, and c.) Profile C at the left strip line.

The level survey was made at two foot intervals along Profiles A, B, and C, see Figure 5. The settlement analysis at pipe structure number 6 was made for Profile A which had the largest settlement of the three profiles, see Figure 6. Note that the length of the settlement profiles is approximately 70 feet in length for Profile C. The maximum settlement and swale lengths recorded at structure number 6 are presented in Table 1.

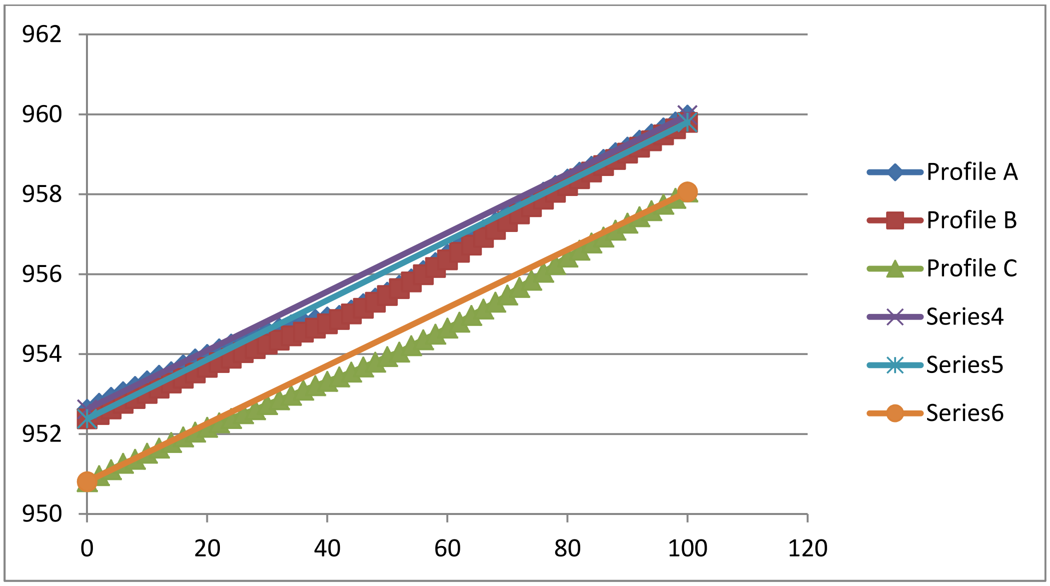


Figure 5. Elevation level surveys at pipe structure number 6.

Based on the initial site visit, three piezocone soundings (CPTU) were made at the pipe structure number 6 location. The plan location of the three CPTU soundings is shown in Plate 1 in Figure 7. The CPTU soundings were performed by Terracon Inc. Tulsa office personnel for pipe structure number 6 on August 20, 2012. The piezocones soundings were advanced by the hydraulics of a Dietrich rubber−tracked mounted drill rig according to the current ASTM D5778 standard test procedure (6). The piezocone used in the soundings was a 10 cm^2 base area cone with the pore pressure element in the u_2 position. Observing the performance of all the Terracon CPTU cone soundings, I would judge that the ASTM standard test procedure was followed with due care.

Embankment Section

A review of the plan cross−section on plan sheet 20 from the plan set indicates that the embankment height above pipe structure number 6 at the center line of survey was 19.1 feet. The depth to the bottom of the pipe structure grade line at the center line of survey was 21.1 feet. The grading section was 34.1 feet wide, and the embankment side slopes were at a 3:1 slope ratio.

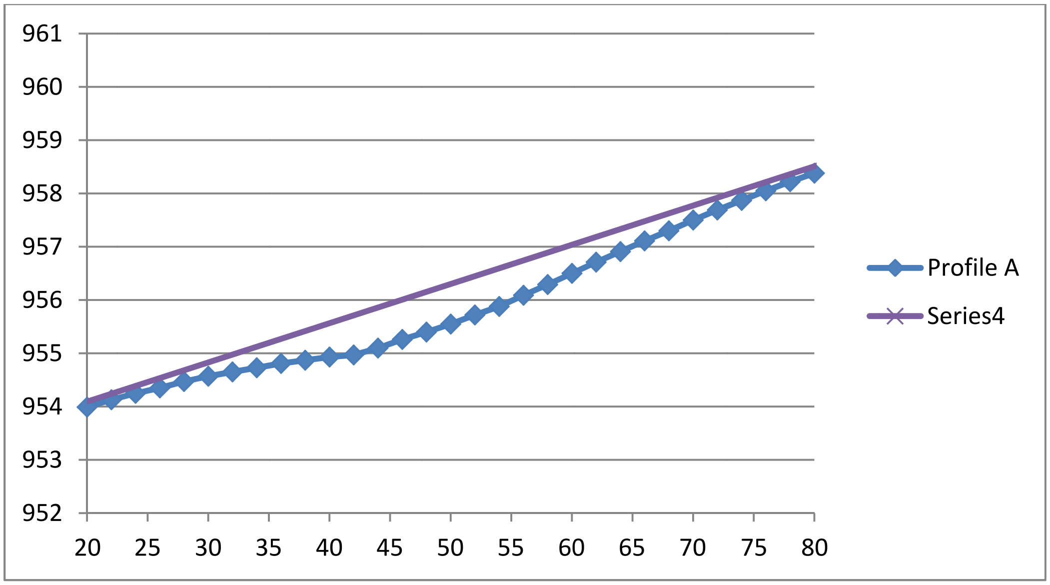


Figure 6. Elevation level survey for Profile C at pipe structure number 6.

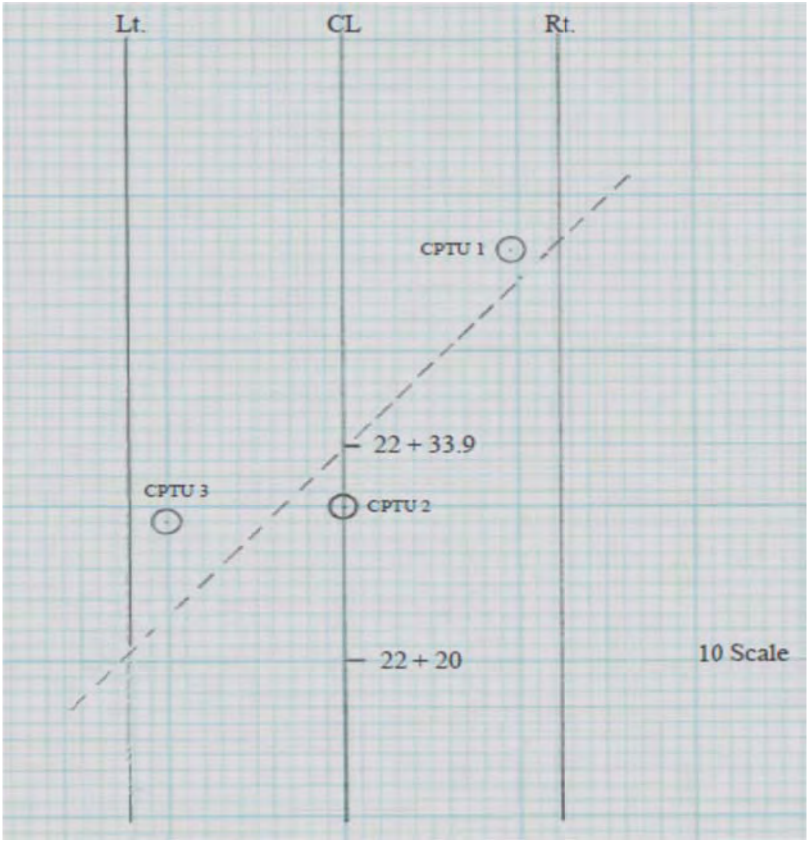


Figure 7. Pipe structure number 6 piezocone sounding locations.

Table 1. Maximum profile settlement.

Structure Number	Maximum Settlement, Feet (Inches)/ Swale Length, Feet		
	Profile A	Profile B	Profile C
6	0.758 (9.096)	0.643 (7.716)	0.530 (6.360)
	60	60	70

The standard for pipe installation is shown in Figure 8. The normal application of this ODOT standard is to construct the embankment up to the top of the initial embankment line shown in Figure 8 and then sub excavate down to the pipe grade before proceeding placing the pipe.

CPTU Soundings

At the pipe structure number 6 location, the BIA personnel cored the pavement, and then they removed the aggregate base course material from the core hole. The Terracon personnel measured the core thickness and the depth of the augered out aggregate base layer in the core hole before starting the CPTU soundings. CPT sounding 1 shown in Figure 9 is typical of the site condition at the pipe structure number 6 location. The CPTU soundings 1, 2, and 3 indicate that the embankment material consists of predominately clay soil, but has a variety of soil layers with depth. All soundings were hydraulically pushed to a point of refusal, and in all cases penetrated below the flow line of the pipe structure. Notable about the CPTU logs are the following: a.) multiple thin soil layering, b.) the inclusion of organic material in thin to thick layers, c.) layers of relatively soft soil material at varying depths, and d.) Terracon used a moving average interpolation of the tip resistance (q_c) with depth.

Based on the CPTU sounding 1, 2, and 3 log interpretation of the CPTU log tip resistance (q_c) and friction ratio (R_f), the soil materials underlying the pipe flow line are variable at pipe structure number 6 as shown in Table 2. In CPTU soundings 1, 2, 3, the cone and rods indicated the presence of a water table. All CPTU soundings appear to have missed intercepting the granular bedding backfill as planned.

The circumstances surrounding the project construction were discussed in an office visit at the BIA headquarters in Muskogee, Oklahoma on June 14, 2012 and on site during the CPTU soundings between August 21−23, 2012. Key factorsdiscussed that helped to formulate the analysis were the following: a.) the pipe was laid down as specified in the trench excavation detail in Method No. 1 Trench Excavation in Embankment Sections in the Pipe Installation Standard SPI−3 shown in Figure 6, b.) variable embankment soil layers but predominately clay as indicated by the CPTU soundings, c.) the length of the elevation profiles at each structure

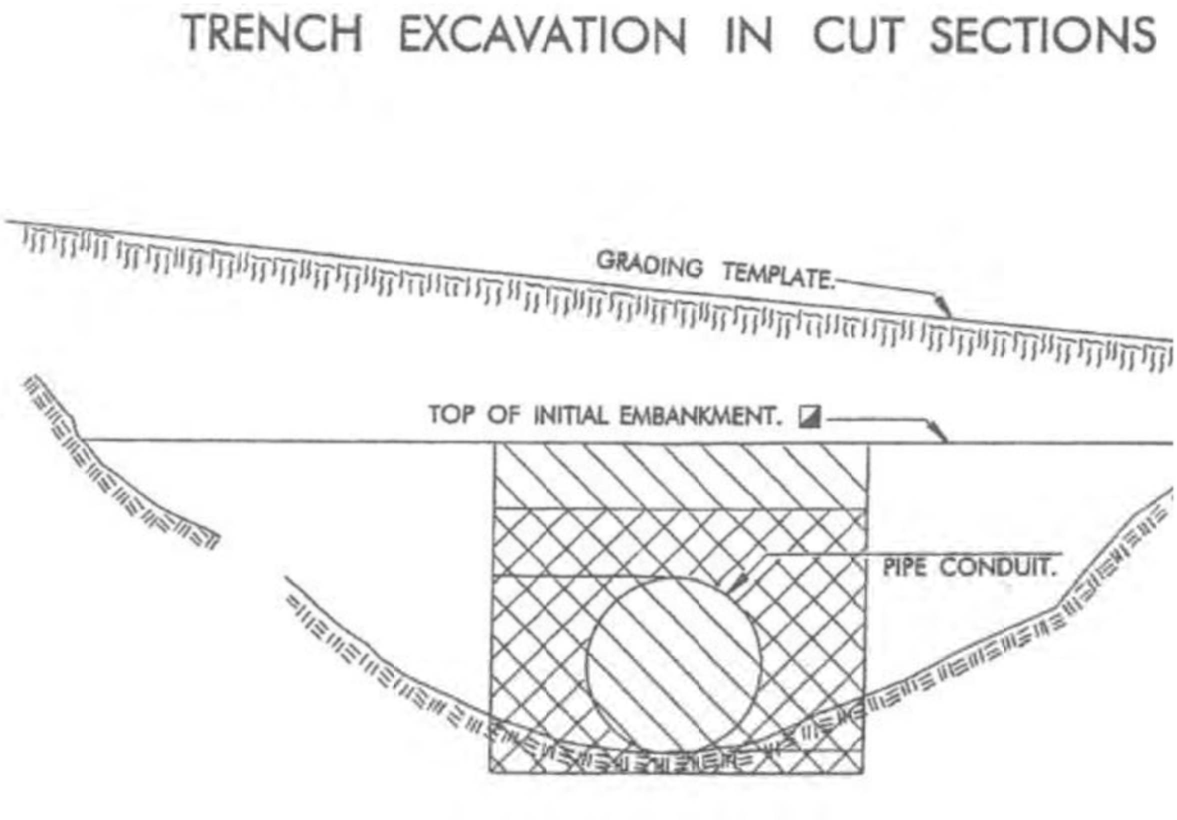


Figure 8. ODOT pipe installation standard SPI−3 detail. Discussion and Problem Model

Table 2. Table 1. CPTU q_c and f_s interpretation at pipe structure 6.

Reference Station[1]	CPTU No.	Average q_c, Tsf	Depth Below Flow Line to Parent Geology, Ft.	q_c Log Interpretation
22+20	1	20.61	2.17	Residual clay lensed with silty sand underlain by sandy silt, clayey silt, silty sand
"	2	34.31	2.50	Residual clay with few lenses of sandy silt and silty sand
"	3	40.93	2.63	Varying thin layers of sand to silty sand, clayey silt, silty clay.

1. Center line station of pipe structure 6

location were substantially longer than what would have been anticipated the collapse of the prism load above the pipe structure, d.) the time frame over which the settlement has occurred as defined in the scope of work, e.) and the comment by Mark L. Bush, BIA Land Surveyor, that all surface elevations have continued to be change.

The model identified in the Method No. 1 Trench Excavation in Embankment Sections in the Pipe Installation Standard SPI−3 in Figure 8 is called a negative projecting conduit. The negative projecting conduit is very favorable in the construction of highway culverts under embankments, since the load produced by a given height of embankment fill is generally less than that for a positive projecting conduit. For the following soil/pipe interaction analysis, the assumption is made that the pipe structures were constructed in trenches as detailed in Pipe Installation Standard SPI−3 with the exception of the 30 foot extension of pipe structure 6.

Soil/Pipe Interaction Analysis

The software used for the analysis of buried structures was Cande−2007 Update Release 7/31/2011, Version 1.0.0.7 (7). This software uses a finite element mesh analysis. This computer software was used in the analysis mode and can estimate the deformation of the soil surrounding a pipe structure with depth. This soil deformation is depicted in a finite element analysis as deflection in the nodes in the finite element mesh. At pipe structure number 6 the CPTU soundings 1, 2, and 3 were used to develop the depths and soil properties for the Cande analysis. The soil property input data used for each pipe structure for the Cande analysis is presented in Table 3.

Table 3. Soil property input for Cande analysis[3].

Pipe Structure	Pipe Diameter, Inches	Embankment Fill		In situ Soil		Pipe Backfill	
		δ[1]	D[2]	δ[1]	D[2]	δ[1]	D[2]
6	36	111[4]	444	116	833	120	3000

1. δ unit weight pcf.
2. Young’s modulus psi.
3. Poisson’s ratio assumed to 0.30 embankment fill and in situ soil and 0.35 for granular backfill.
4. The unit weight of 111 pcf was the average unit weight of the lean clay with sand and sandy lean clay based on a Summary Table of Proctor test results provided by BIA.

The depths are recorded in feet and the soil properties, in situ density (δ) and Young’s modulus (D), were estimated from correlations of the CPTU tip resistance (q_c). At each pipe structure location the analysis was ran using the properties developed from the CPTU soundings as well as those from the conservatively selected soil parameters from the Cande−2007 data base. For properties developed from the CPTU soundings, the methodology used at each pipe structure was to select average representative values of the (q_c) of the embankment fill and underlying residual soil materials for the Cande analysis.

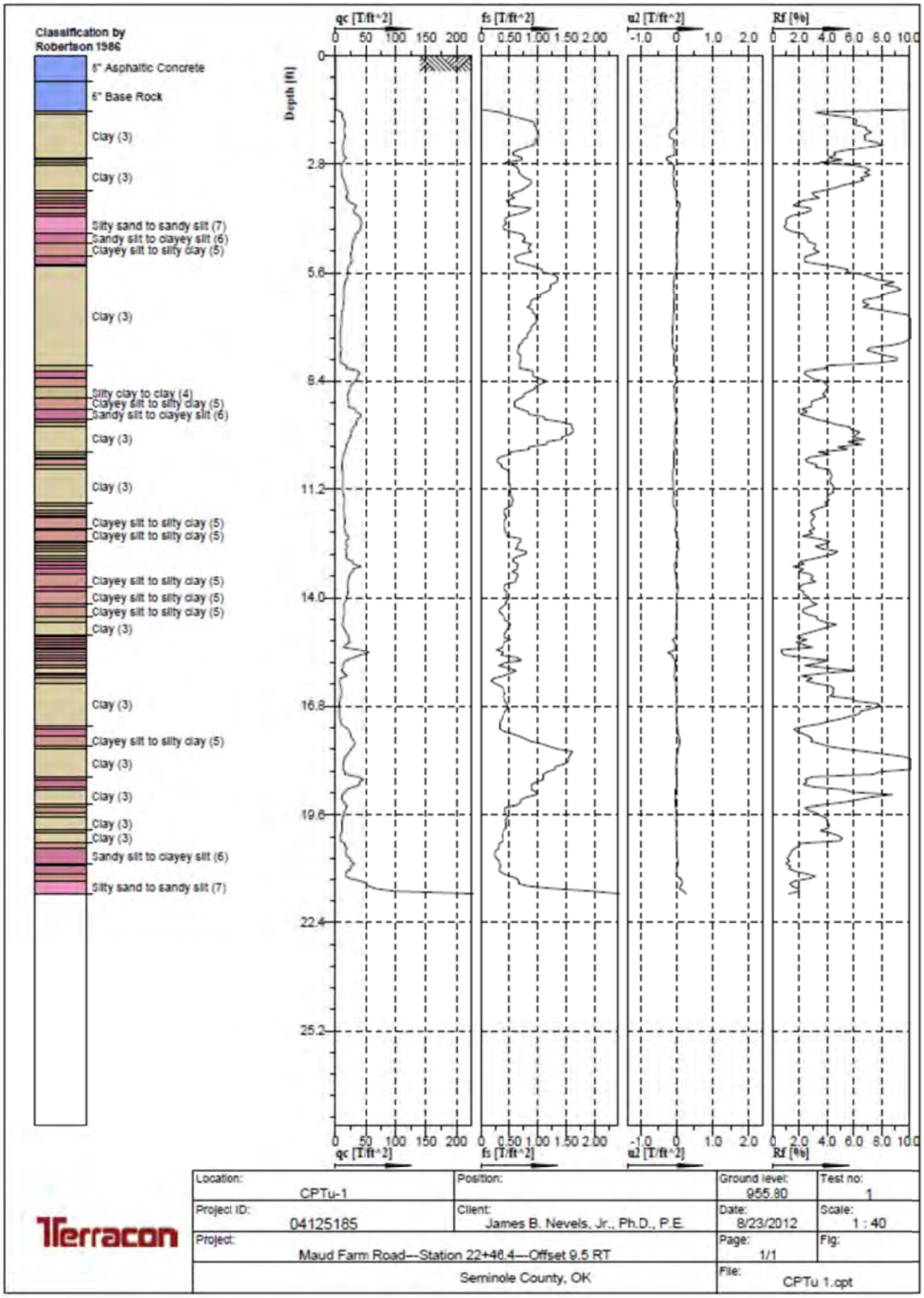
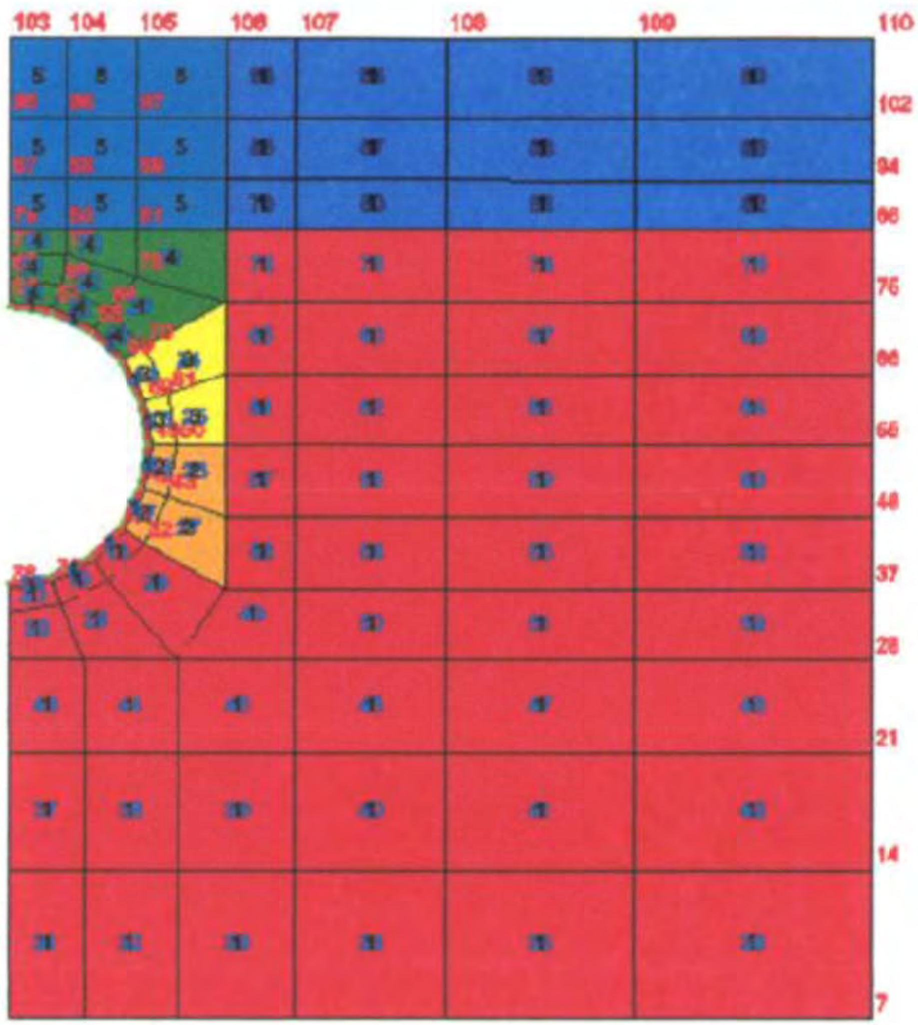


Figure 9. CPTU 1 sounding at pipe structure number 6.

The analysis results consist of finite element mesh plot showing the five color coded components of soil loading surrounding the pipe structure and the deflections of the earth material with depth plot. These plots are generic in appearance in that the pipe dimension, embankment depths, and in situ foundation depths are factored into the model for analysis. Two models were used in producing the finite element mesh plot and the node deflection plot, and they were respectively the isotropic linear elastic and the overburden dependent models. In the isotropic linear elastic model soil properties correlated with the CPTU sounding q_c values were used, and in the overburden dependent model canned soil properties found in the Cande 2007 software were used based on soil types determined in the CPTU soundings, see Figures 10 and 11 for the.

Settlement Analysis

The anticipated discovery in this analysis process that might implicate that the construction was at fault for the settlement above the pipe structures would have been the movement of a soil prism above a pipe structure downward as depicted in Figure 12 for the inverted arch action (8). In the finite element mesh deflection plots presented in Figures 10 and 11, the opposite trend is



seen

Figure 10. Pipe 6 finite element mesh Overburden Dependent, Profile C, canned properties.

seen throughout the deflection plot figures. A settlement analysis such as the FHWA Embank computer software (9) to check for the prediction of a one−dimensional consolidation of a compressible soil layer underlying the embankment loading was ruled out. The reason being is that the CPTU tip resistance (q_c) in this case indicated very stiff residual soils. These residual foundation soils are considered relatively incompressible based on the high q_c values seen in the cone soundings and, from experience we know that these residual soils are over−consolidated due to desiccation typically in the range of 3.0 to 4.5 Tsf for residual soils of Permian geologic age. Also the residual silty sands and clayey silts can be over−consolidated due to desiccation as well.

The question then as to what mechanism could have caused the settlement came to light during a conference call at 10:00 AM on October 02, 2012 when it was learned that the project was constructed during a relatively very dry period, and after project completion the site experienced extensive rainfall. As stated above the in situ moisture content of these shallow residual soils in the drainage ways at these structure locations can vary depending on the climate and time of year.

It is believed that the cause of settlement seen in the pavement surface is the result of vertical pressure against a yielding base. This concept is borrowed from theoretical soil mechanics and has a precedent in the subsidence above culverts and tunnels in previously reported case histories (10) (11) and is depicted in Figures 13 and 14. In case history accounts such pressure is applicable to flexible pipe structures, conduits, and tunnels, see Figure 15.

The vertical pressure produces a displacement (ρ) only and to this must be added the embankment overburden pressure (γH), see Figure 13. In Figure 13 the zone of plastic flow is delineated by a set of slip lines. Referring to Figures 12 and 13, if the yield proceeds far enough and the distance D is small enough then one or more of the slip lines may propagate to the ground surface. The strip ab relates to the base of the trench excavation detail in Method No. 1 Trench Excavation in Embankment Sections in the Pipe Installation Standard SPI−3 in the plan set plus a wetted zone on either side while the remainder of the base is rigid, see Figure 6. The depth of the strip includes the very top of the ground surface upon which the pipe is placed plus thin underlying soil layers. The yielding is the result moisture intrusion and the softening of the of the ground surface upon which the pipe is placed and the consolidation of underlying thin layers of clay, silty sand, clayey sand, and clayey silt, see Table 1. The softening at very top of the ground surface upon which the pipe is placed results from the moisture infiltration into the granular bedding, and outward from there the where moisture spreads laterally away from the pipe and downward along the pipe extent. The source of this moisture is from rainfall occurring at the site which generally finds its way to drainage ways and the pipe structure locations. An estimate of the approximate rainfall at the site from the start of the project (July 2005) to July 2012 is based on the monthly rainfall summaries at the closest Mesonet site at Bowlegs, Oklahoma, see Figure 16. As can be seen the area received a substantial amount of rainfall occurring in the closing months of the project and thereafter.

The result of such yielding along these slip lines is the eventual subsidence over the pipe structures, and this is due the shearing along slip lines in a plastic flow, see Figure 14. An approximation of the total width of the zone of greatest subsidence (B_E) is made by the approximate trench equation given in Table 3. The B_T and B_E values for the plan pipe trench width and height from the Pipe Installation Standard SPI−3 presented in Figure 8 and for a five foot wetted zone either side of the plan pipe trench for varying values is presented. The real

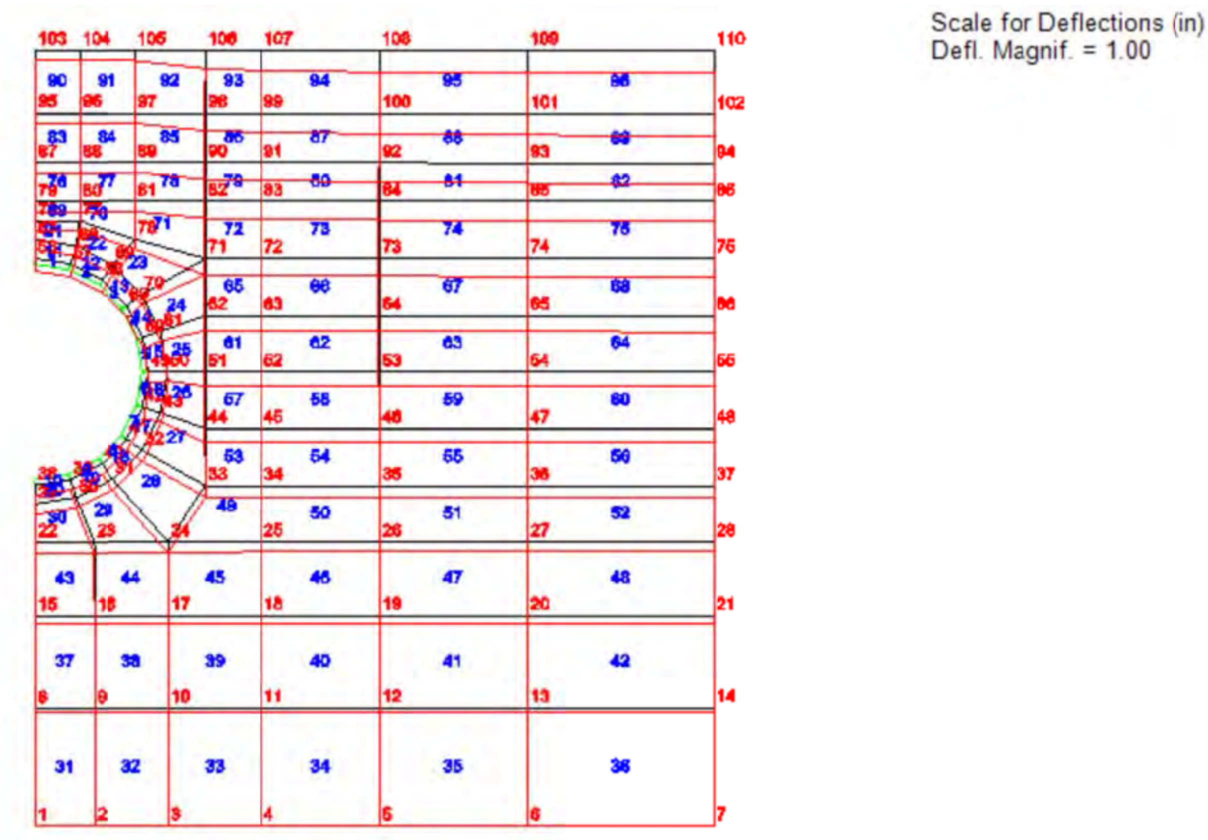


Figure 11. Pipe 6 deflectionplot, Overburden Dependent, Profile C, canned properties.

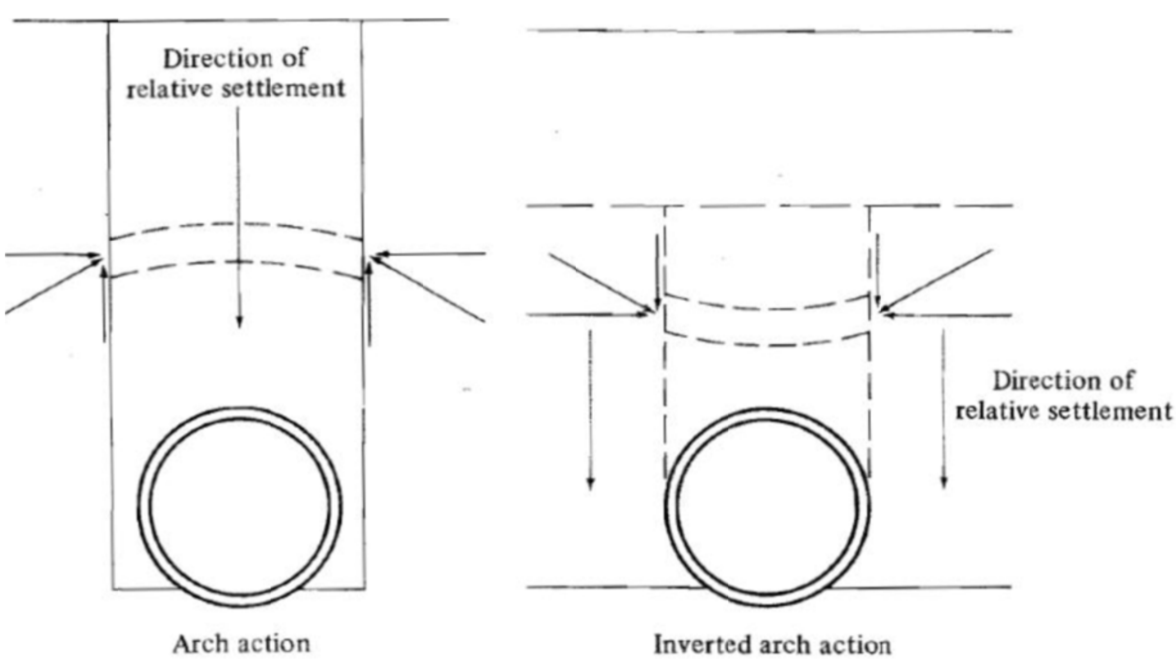


Figure 12. Direction of relative movement between overlying prism and adjacent soil prisms.

shear zone indicating plastic flow in embankment fill extends beyond the limits of the hypothetical trench (B_E).

Conclusion

The conclusion reached after a thorough review of the finite element mesh and deflection plots is that the red mesh deflection contours reflect away from the soil prism immediately above the pipe and extend even below pipe grade line and are indicative of settlement from another reason. Had red colored node contours reflected downward and into the soil prism immediately above the pipe, then an argument could be possibly be made that the pipe installation could have had something to do with the settlement presented in the elevation survey for Profiles A, B, and C shown in Figures 5 and 7.

Based on the Cande analysis made and all the supporting evidence, the settlement is thought to be the result of pressure on a yielding base. The red mesh deflection contours do indicate yielding at and below the pipe grade line which supports the support the concept, refer to Figure 11. The rainfall over the period following the construction was the major contributor to the problem. In the Cande analysis, the overburden dependent model was accurate than the isotropic linear elastic model, and the Cande canned soil properties were more representative than those selected from the CPTU sounding correlations with soil properties. selected from the CPTU sounding correlations with soil properties.

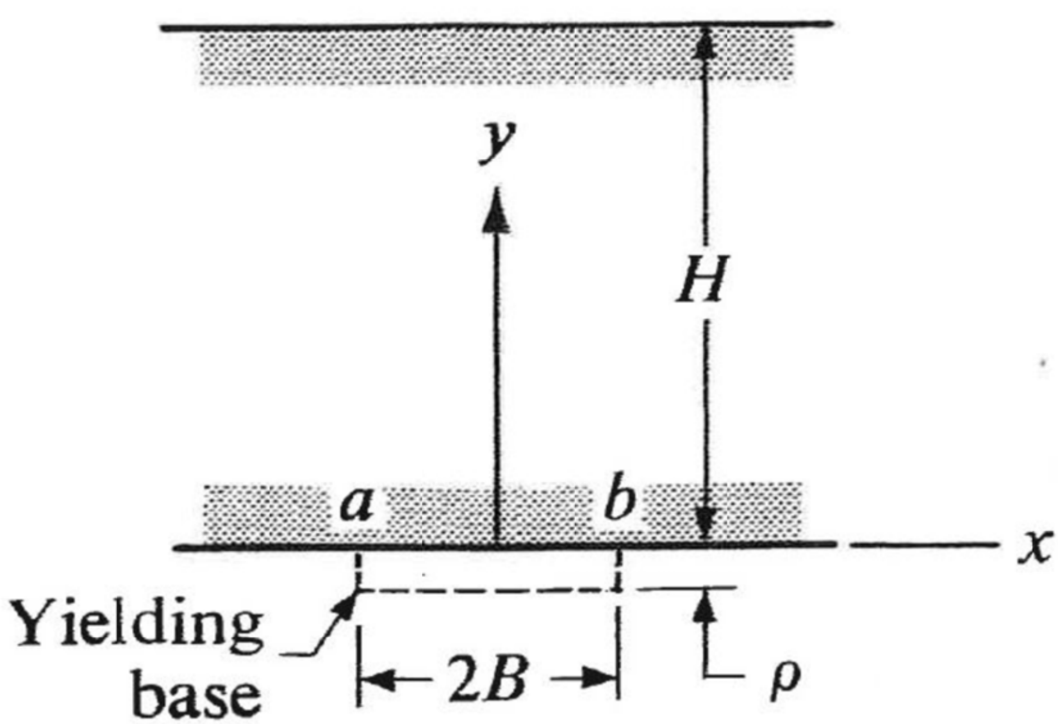


Figure 13. Yielding base free body diagram.

The equivalent trench width shown in Table 4 ranging from the plan standard trench width to an estimated five foot wetted zone either side of the pipe trench width comes close to matching the largest extent of the settlement shown in Profile C in Figure 6. The problem is indeterminate in that the deflection of the shear zone that may vary induces a vertical subsidence adjacent to the subsiding structure surface. Table 3 further suggests that moisture from the granular backfilled trench most likely does spread laterally away from the pipe and downward along the pipe extent to form a strip ab shown in Figures 13 and 14. In summary, the investigation indicates that the contractor cannot be held accountable for the settlement of the embankment.

Table 4. Estimate of equivalent trench

Pipe Structure	Pipe Diameter, Inches	Width of Trench, Feet		Height of Trench, Feet	Equivalent Trench, Feet, B_E[1,4]						Maximum Node Displacement, Inches
		B_T[2]	B_T[3]	H_T	$\phi = 0$	$\phi = 10$	$\phi = 20$	$\phi = 0$	$\phi = 10$	$\phi = 20$	
6	36	5.50	15.5	5.75	18.5	16.42	14.59	26.75	24.74	23.19	3.624 (OD)

1. $B_E = B_T + 2H_T / \tan[(45 + \phi/2)]$
2. B_T = Estimated lateral width based plan trench width in the Pipe Installation Standard SPI–3 in the plan set and Figure 28.
3. B_T = Estimated wetted zone of 5 feet either side of the plan trench width in the Pipe Installation Standard SPI–3 in the plan set and Figure 28.
4. Equivalent Trench estimated for varying ϕ values.

Recommendations

The BIA was advised that to bring a lawsuit against the contractor and argue against refunding the contract retainer, a sum of $358,000 dollars, held by the BIA to help pay for the repair of the the roadway subsidence over the five pipe culverts would not be in their best interest. The contractor cannot held accountable for soil movement at or below the pipe grade line. Another question by the BIA was how long would this subsidence last? A time rate of settlement from consolidation theory is not exactly an appropriate measure for the prediction of the length of time for total amount of settlement, since the subsidence is due to soil movement and/or shearing along slip lines. There is no known method for predicting the total time of settlement for a yielding base. From a practical stand point, the repair of the five designated embankment settlement locations above pipe structures needs to proceed forward, and an asphalt level up is the appropriate recommended maintenance repair technique.

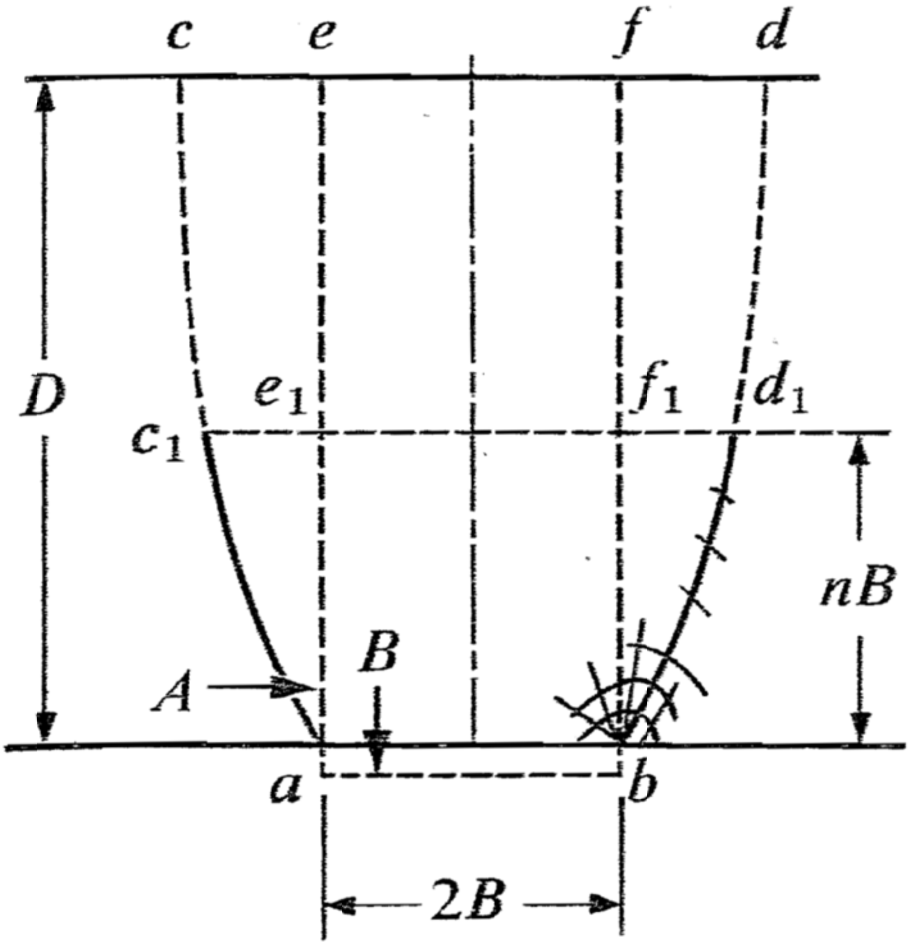


Figure 14. Slip lines associated with a yielding base ab.

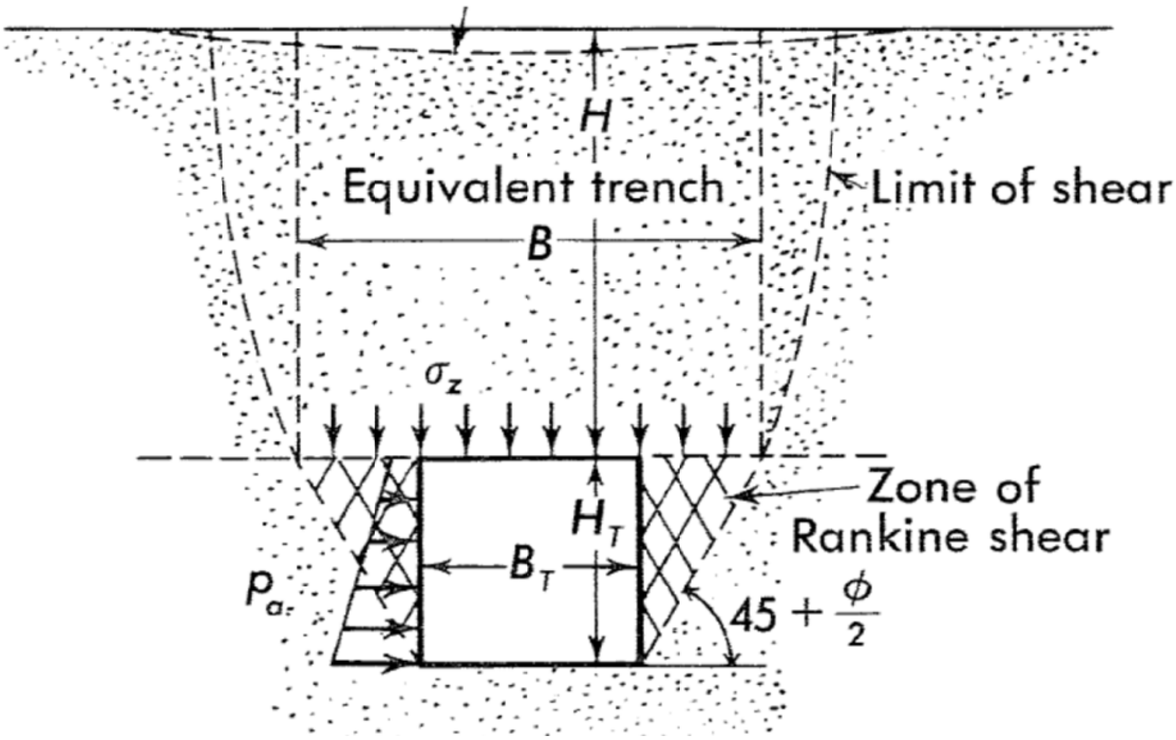


Figure 15. Plastic wedges and lateral earth pressure on a deflecting buried structure.

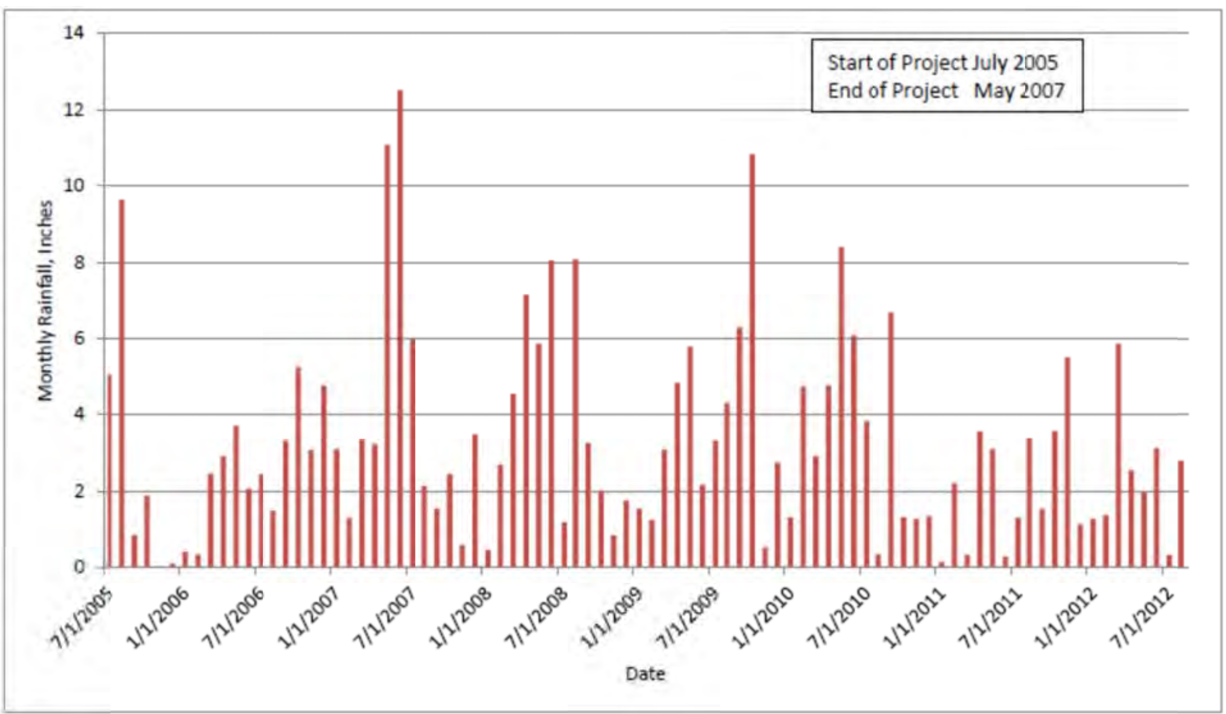


Figure 16. Rainfall distribution from the Bowlegs Mesonet site in Seminole County.

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Emergency Slope Stabilization,
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ABSTRACT

In response to flooding/ scour damage from Tropical Storm Irene in late August 2011, the New York State Thruway Authority and Golder prepared emergency slope mitigation designs for two slope failure areas in the southern embankment beneath the Catskill Creek Bridge on I-87 south of Albany. Regional catastrophic flooding occurred in the region on August 28, 2011 from Tropical Storm Irene. Based on United States Geological Survey (USGS) stream gauging data, the water level in Catskill Creek rose at least 25 feet during flooding from Irene. Following this event, Thruway personnel inspected the bridge foundations, and discovered recent scour of embankment fill, riprap and other soils surrounding the piers north and south of the streambed. The scour included loss of riprap and soils adjacent to the east footing of Pier 3 on the south side of the northbound truss. The scour compromised the pier foundation as well as a large portion of the slope supporting the southeast approach of the northbound structure.

Shortly after discovering the damage, site visits were conducted to initially evaluate the scour damage adjacent to the pier and collect site geologic/geotechnical field data. During one site visit, a larger landslide failure surface was noted, along with tension cracks at the head of the southern bridge approach embankment. To evaluate potential mitigation approaches, the project team reviewed site geology and geotechnical conditions using the original highway/bridge design borings; conducted back-analysis of the failure modes to estimate geotechnical conditions; developed conceptual slope mitigation concepts, inclusive of the Thruway's design for oversize riprap for scour mitigation; developed a soil nail – tensioned mesh system to retain both soil scour areas and the toe of a riprap repaired slope (used only in areas where a stable riprap slope design could not be used to avoid encroaching on the stream channel); developed special provisions; and prepared a design report. Mitigation construction was conducted between November 2011 and May 2012.

INTRODUCTION

The Catskill Creek Bridge (the Bridge) is a 600-foot long steel arch bridge spanning Catskill Creek at approximately Mile Post 113 of the New York State Thruway (Figure 1). The Bridge carries four lanes of the roadway and respective shoulder lanes for both northbound and southbound traffic. The Bridge trusses are set on eight large concrete piers socketed into bedrock. The piers are staggered and do not lie within the streambed. The bridge was designed in 1952 and constructed in 1953. The northern abutment and the eight piers are founded on bedrock with two footings each, socketed 1 to 6 feet (ft) into bedrock. The southern abutment is supported on deep individual buttress footings bearing on rock. In 1992, the Thruway conducted a bridge rehabilitation program which included the addition of a soldier pile and lagging wall to the southeast approach of the abutment, and a reduction in grade of the existing rockfill embankment from 1 horizontal : 1 vertical (1H:1V) to 1.5H:1V.

Figure 1 - Site location map.

Regional catastrophic flooding occurred in the region on August 28, 2011 from Hurricane Irene. Based on United States Geological Survey (USGS) stream gauging data, the water level in Catskill Creek rose at least 22 ft during flooding from Irene. Following these events, Thruway personnel inspected the bridge foundations, and discovered recent scour of embankment fill, riprap and other soils surrounding the piers north and south of the streambed. The scour included loss of riprap and soils adjacent to the east footing of Pier 3 on the south side of the northbound truss (see Figures 2 and 3).

Figure 2 - Scour damage below Pier 3.

Figure 3 - Scour of embankment causing slope failure at toe.

EXISTING CONDITIONS

Geology

Regional geologic mapping indicates the bedrock consists of limestones of the Lower Devonian-aged Helderberg Group (Figure 4; Fisher et al., 1970; Raytheon, 1996). These rocks were previously mapped as the Devonian Alsen Limestone (Ruedemann, 1942). More recent mapping indicates these rocks consist of the Coeymans/Manlius, Kalkberg, New Scotland, and Becraft Formations, which have been deformed by broad open folding and subject to low angle thrust faults (Marshak and Engelder, 1987). The limestone consists of medium to light tan-gray to rusty gray (weathered), medium to dark gray (fresh), moderately weathered, fine to medium grained, thin to medium bedded (0.25 to 3 ft), moderately jointed, fossiliferous grainstone, with dark gray to black chert nodules to about 2 inches in diameter, occurring in discontinuous

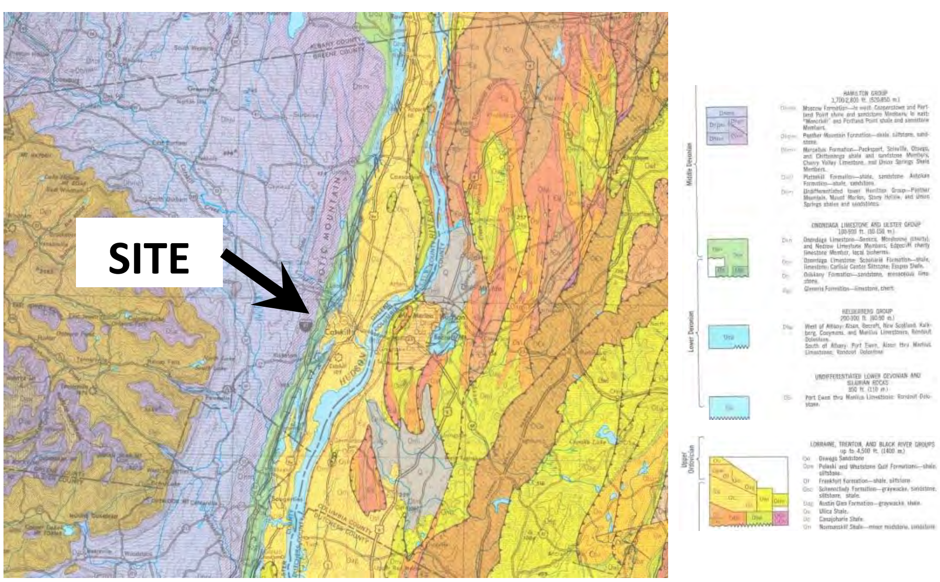

Figure 4 - Regional bedrock geologic map (Fisher et al., 1970).

beds. The strata dip gently to the south-southwest at roughly 5-10 degrees. Solution weathering along two sets of near vertical orthogonal joints has produced karst features such as caves observed in the north side of the creek, and voids encountered in core drilling for a prior scour analysis study (Raytheon, 1996).

Regional surficial geologic mapping indicates surficial soils on the south side of the creek consist of lacustrine delta sediments (coarse to fine gravel and sand, stratified, generally well sorted, deposited at a lake shoreline); and lacustrine silt and clay (generally laminated silt and clay, deposited in proglacial lakes, generally calcareous, with the potential for land instability; Caldwell et al., 1991). Glacial striae occur on limestone bedding surfaces, and potholes are common within the creek bed.

Pier Arrangement

The bridge is founded on eight (8) piers, each of which consist of two vertical reinforced concrete columns supporting a horizontal reinforced concrete beam (Figure 5). Four piers support the northbound lanes (Piers 1, 3, 5 and 7), and the other four piers support the southbound lanes (Piers 2, 4, 6 and 8). The piers are staggered by about 30 ft to account for the skew across the creek. Each pier column is supported on a concrete footing on bedrock. In 1992, the Thruway conducted a bridge rehabilitation program which included the construction of a soldier pile and lagging wall on the southeast approach of the south abutment.

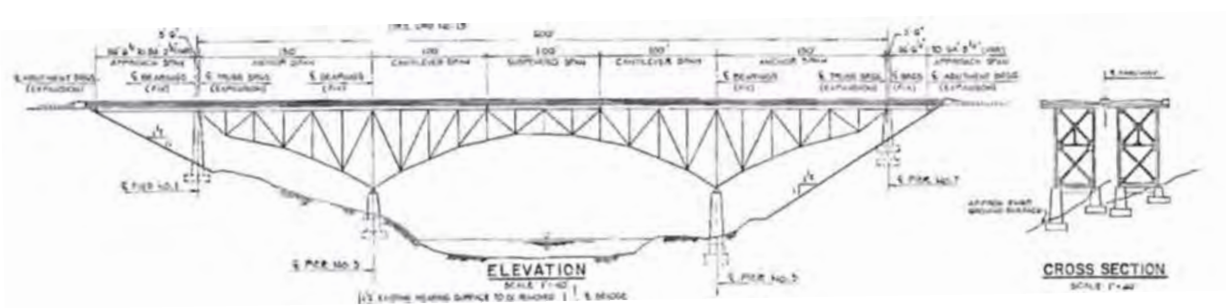

Figure 5 - Pier arrangement of bridge, view looking upstream (north).

Field geologic mapping for the initial Bridge design indicated the presence of a north-trending fault zone beneath Piers 5 and 6 on the north creek bank (De Leuw and Brill, 1953). The vertical orientation and other details of the fault are not shown on the design drawings, and we did not observe indications of this fault zone on the south side of the crossing were not observed.

Scour At Pier 3

Based on the Thruway site photos and inspection records, flooding from Hurricane Irene created a washout of fill and natural slope materials adjacent to and downstream of the east footing of Pier 3 (Figure 2). Exposed materials consist of rocky fill material, composed of silt, sand and angular dark gray limestone boulders to about 1 ft maximum dimension. This material was likely used as backfill following excavation for and construction of Pier 3. Additional materials, such as bedrock exposed nearby and naturally occurring soils were also eroded presumably by scour action during flooding.

Subsurface Conditions

The project borehole logs from 1952 indicate the original subsurface soils consisted of brown, moist to wet silt, with trace to some clay, and trace to some gravel to a maximum thickness of 11 ft. The two logs from 1960 (D.H.-1 and D.H.-2) indicate fill materials consisting of boulders had been placed on the upper portion of southeast abutment slope to a depth up to 16 ft. In this area, medium to stiff, brown, wet varved silt overlying brown, wet, very stiff bouldery glacial till lies beneath the fill materials. Other fill materials lying beneath the bridge near Pier 3, as described in the borings drilled in 1991, consist of medium stiff to stiff, brown to black, coarse to fine gravelly sand fill (SW), overlying stiff, brown, silt and coarse to fine gravel fill (GM). These materials overlie medium stiff, reddish brown to gray, clay with little coarse to fine gravel (native soils).

The 1990 borehole logs indicate the bedrock consists of gray to dark gray, fresh, hard, laminated to very thinly bedded, fossiliferous limestone, with less than 5% disseminated pyrite. Rock quality designation (RQD) ranged from 20% to 100%. The 1952 logs indicate voids up to 3.5 ft long were encountered during coring.

Groundwater was not noted on the 1952 or the 1990 borings. In the 1960 borings, groundwater level was not noted; however the sample descriptions indicate that samples were wet below approximately 10 ft of depth. Based on these descriptions the groundwater depth is estimated to be 10 ft near the top of the slope. Localized perched groundwater emanates from a spring between the overburden and bedrock surface beneath the southbound lanes at approximately elevation 100, which was used to estimate the groundwater level mid-slope. Lush vegetation indicates this spring may be perennial.

Slope Failure Downstream of Pier 3

During a site visit on September 27, 2011, a landslide scarp within the fill materials east of the south abutment was observed (Figure 3). Further investigation revealed several tension cracks above the scarp and below the abutment, as well as additional scarps adjacent to the soldier pile and lagging wall. The upper-most scarps and tension crack were within 5 ft of the base of the soldier pile and lagging wall, and displacement of the sliding soil exposed a fiber optic line and associated junction box. Fallen trees at the scarp with dead vegetation indicate the failure may be relatively recent, possibly occurring due to scour of the slope toe during Hurricane Irene flooding. Estimated displacement of the landslide at the recent scarp is on the order of 20 ft down slope. Tension cracks associated with this slope failure were recovered by the Thruway surveyors for inclusion on the site survey map.

Flood Levels

Gage height data from the USGS gage on the Catskill Creek in Catskill, New York (Gage No. 01362090) indicate the water level in the creek rose from about 3 ft to over 26 ft stage height on August 28, 2011 during flooding from Hurricane Irene, and rose from about 4 ft to about 18 ft stage height on September 8, 2011 during flooding from Tropical Storm Lee. Observations of tree damage during the site visit on September 27, 2011 indicate the water level reached at least 20 ft above the exposed shoreline during Irene flooding. For the analyses of high water conditions, a conservative value of 25 ft above the surveyed shoreline elevation was used for back analysis calculations of the failure mechanisms.

GEOTECHNICAL PARAMETERS

Interpreted Bedrock Contours

Prior test boring information was used to develop an interpretation of the bedrock topography. Many of the borings logs contained station and offset and bedrock elevation information, which was used to plot the location and elevation of bedrock observations on the topographic survey map. In addition, as-built records of the pier footings, which included the surveyed locations and elevations of bedrock outcrops, were used to supplement the boring information. Because borehole data are lacking in the slope failure area downstream of the Pier 3 scour, the bedrock contours in this area were estimated from historic topographic maps, site photos of the exposed rock slope downstream, and from slope failure back calculations. The interpreted bedrock structure was used to estimate the top of bedrock in the stability analysis profiles.

Back Calculation of Parameters Under Water Conditions

Due to limited time, budget and access issues, a geotechnical investigation including test borings and laboratory testing to determine geotechnical parameters was not possible. Instead parameters were developed through back calculation. Using the topographic survey information, the interpreted bedrock contours, the limited historic borings in the area to determine subsurface soil conditions, and the estimated water table during the flood, a stability model of the slope in in the south end was generated. The resulting model consists of a layer of rock fill over the entire slope and a thinner layer of till directly above the bedrock.

Observations made during the site visit, including the scarp and tension cracks at mid-slope and at the top of the slope, and evidence of movement near the toe, indicate that the slope was destabilized during the flood and had a factor of safety below 1.0. Starting with typical strength values for the rock fill and the till, the strength values were adjusted until the factor of safety of the model was at or just below 1.0 (Figure 6). Using the adjusted strength values and reducing the creek level to estimated normal conditions resulted in a factor of safety of 1.016 (Figure 7), which indicates that the slope is barely stable and is consistent with the apparent lack of any movement under low water conditions.

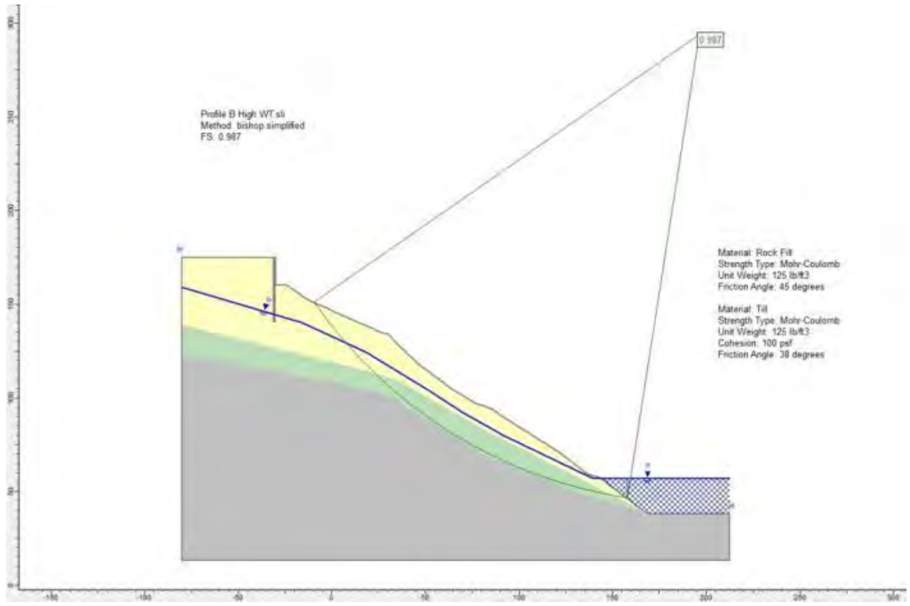

Figure 6 - Stability analysis of Area 1 under high water conditions.

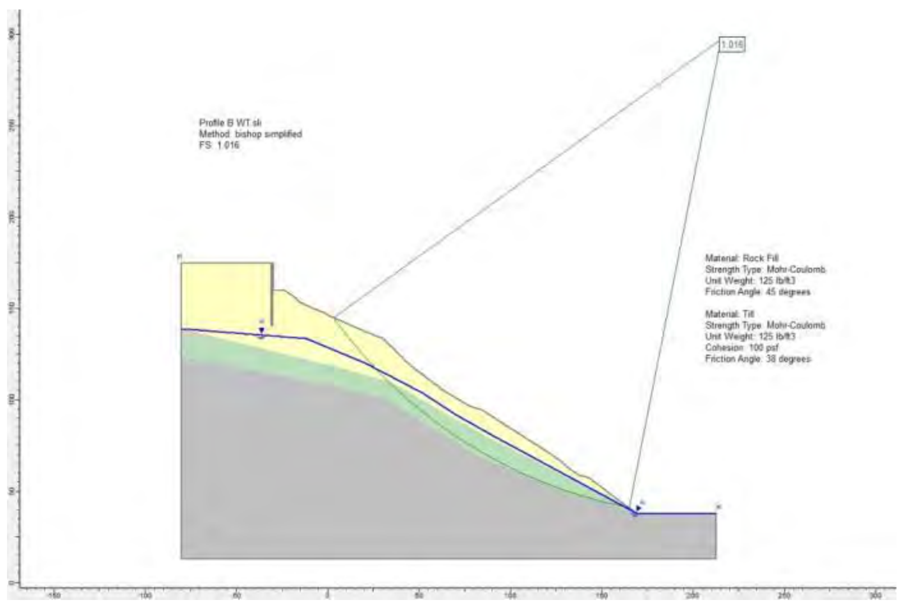

Figure 7 - Stability analysis of Area 1 under normal water conditions.

Stability modeling indicated that the failure surface is defined by a circular failure with a large radius, and extends all the way through the surficial rock fill and into the lower till, running just above the bedrock surface for much of its length. This is confirmed by the observed site conditions of the bulging toe and tension cracks at the middle and top of the slope. It also indicated that possible stabilization methods would need to include either substantial removal of material at the top of the slope to reduce the driving force, or creation of a substantial structure at the toe to increase the resisting force.

ALTERNATIVE DESIGNS

As shown on Figure 8, the project is divided into three areas:

- Area 1 is the furthest downstream section of the project and ranges from baseline station 0+00 (ft) to approximately station 1+56. This area is approximately the width of the large slope failure at the downstream limit of the project.

- Area 2 is the middle section of the project and ranges approximately from baseline station 1+56 to station 3+50. This area lies between the large slope failure at the downstream limit of the project and the scour below Pier 3 at the upstream limit of the project.

- Area 3 is the furthest upstream section of the project and ranges approximately from baseline station 3+50 to station 4+40. This area includes the creek bank scour adjacent to and immediately downstream of Pier 3.

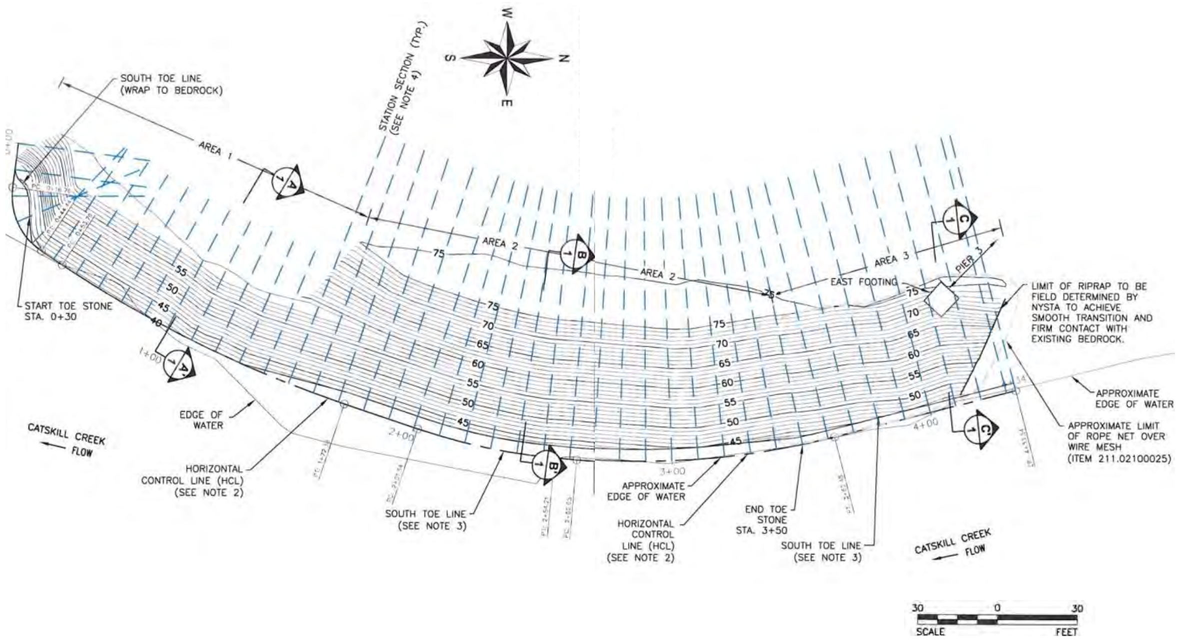

Figure 8 - Site remediation plan with Areas 1 - 3 shown from left to right (south to north).

The Thruway evaluated alternatives for stabilizing each of the three areas, as described in the following sections. Each alternative had to enhance stability using methods that would be constructible given the access limitations of the site and the limit of work, which included no permanent encroachment into the creek. All factors of safety in the following discussion were calculated using the material properties derived from the back analysis and with high creek conditions (creek at elevation 57 ft).

Alternatives for Stabilization of Area 1

Area 1 is characterized by a large slope failure that extends from the toe of slope at elevation 40 ft to top of the slope at approximately elevation 160 ft adjacent to the bottom of the soldier pile and lagging wall at the south abutment. Field reconnaissance revealed several scarps and tension cracks that resulted from the failure. The toe of the slope in Area 1 is also very close to the limit of work. Historic records indicate the slope failure likely was first observed soon after the bridge was constructed in 1952. (Borings D.H.-1 and D.H.-2 were drilled in 1960, presumably to investigate slope stability.) The soldier pile and lagging wall, which was built around 1992, was constructed to protect the south abutment from the effects of the moving slope. In its current state, the slope is marginally stable but sensitive to destabilizing events such as high groundwater or creek level, or loss of material at the toe due to erosion by the creek.

Alternatives for stabilizing Area 1 were limited to modifications to the lower portion of the slope because access to the middle and upper parts of the slope would be difficult with conventional earth moving equipment. Furthermore, because the limit of work essentially coincided with the current toe of slope, it is not possible to enhance stability in this area by constructing a stabilizing berm at the base. Therefore the alternatives evaluated for this area included the following:

- Alternative 1.1 – Steepening the bottom of the slope using riprap placed from the limit of work up the slope at a 1.5H:1V grade to elevation 60 ft.

- Alternative 1.2 – Riprap as configured in Alternative 1.1 with wire rope mesh placed over the riprap and anchored with cable anchors drilled and grouted into competent bedrock.

- Alternative 1.3 – Reinforced concrete wall constructed at the limit of work, backfilled with riprap and anchored as necessary to provide an adequate factor of safety.

Stabilization with Riprap Only (No Anchors or Mesh)

Alternative 1.1 would provide very little improvement in factor of safety – 1.1 compared to 0.99 for the unimproved condition. However the riprap slope protection enhances the erosion resistance of the material at the bottom of the slope. Furthermore the riprap can be graded to provide a smooth transition from the improvements in Area 2 to the natural creek bank contours downstream of the project area.

Stabilization with Riprap, Anchors and Wire Rope Mesh

Alternative 1.2 would provide an improved factor of safety due to the additional resistance provided by the cable anchors. Depending on the number of anchors provided, the factor of safety could be increased to as much as 1.3 for failures that included the riprap. However, for failures that daylight in the slope above the riprap, the factor of safety is not improved. This alternative essentially would reduce the likelihood of full slope failures, but would not improve the stability of potential mid-slope failures that could impact the abutment. Drilling for the cable anchors would be difficult because the thickness of unconsolidated material above bedrock was at least 30 ft and therefore the borings would have to be cased. Additionally, depth to competent bedrock could not be estimated with the data available, so there could be significant risk associated with anchor length and attendant cost. Anchors would consequently be expensive, and this alternative provides only partial improvement of the slope.

Stabilization with Reinforced Concrete Wall

Alternative 1.3 would provide similar stability improvement to Alternative 1.2 and would carry similar uncertainty regarding the length and difficulty of drilling for anchors. This alternative was not completely evaluated because the Thruway determined that the permitting required to construct it would likely delay the project.

Alternatives for Stabilization of Area 2

The slope in Area 2 is slightly flatter than 1H:1V and is characterized by a slope failure that likely extends from the bottom of the slope to a scarp mid-slope that varies in elevation from about 106 ft to about 120 ft. This scarp is an extension of one of the mid-slope scarps observed in Area 1. Field reconnaissance did not reveal any other scarps or tension cracks in Area 2. The toe of the existing slope in Area 2 is set back from the limit of work by as much as 40 ft. Prior to remedial construction, the slope was marginally stable but sensitive to destabilizing events such as high groundwater or creek level, or loss of material at the toe due to erosion by the creek.

Alternatives for stabilizing Area 2 were limited to modifications to the portion of the slope below elevation 75 ft, because access to higher parts of the slope would be difficult with conventional earth moving equipment. Because the separation between the limit of work and the existing toe of slope allows placement of a substantial stabilizing berm, the alternatives evaluated for this area included the following:

- Alternative 2.1 – Placement of a riprap stabilizing berm with its toe at the limit of work with an outside slope of 1.5H:1V and wire rope mesh with anchors to enhance stability.

- Four alternatives (Alternatives 2.2.1 through 2.2.4) – Riprap without wire rope mesh or anchors constructed to the following geometries:

    - Alternative 2.2.1 – Placement of a riprap stabilizing berm with its toe at the limit of work, an external slope of 1.5H:1V to elevation 59 ft, and a bench of varying width.

    - Alternative 2.2.2 – Placement of a riprap stabilizing berm with its toe at the limit of work, a varying external slope to elevation 59 ft and no bench.

    - Alternative 2.2.3 – Placement of a riprap stabilizing berm as in Alternative 2.2.1, but with a top elevation of 75 ft.

    - Alternative 2.2.4 – Placement of a riprap stabilizing berm as in Alternative 2.2.2, but with a top elevation of 75 ft.

Stabilization with Riprap, Anchors and Wire Rope Mesh

Alternative 2.1 provides the maximum improvement in factor of safety, due to the effect of the cable anchorage. However because of the potential difficulty and high cost of installing anchors, as described above for Alternative 1.2, Alternative 2.1 was not considered further.

Stabilization with Riprap Only (Four Alternatives)

Alternatives 2.2.1 through 2.2.4 provide improved factor of safety due to the stabilizing effect of the riprap berm. The analysis showed two significant failure geometries for these alternatives: a circular failure that included approximately the lower half of the slope and exited the slope at the base (base failure), and another failure located mid-slope which exited the slope near the top of the riprap (mid-slope failure). The analyses also showed minor surficial sloughing was possible in the portion of the slope above the riprap. This sloughing was considered to be tolerable, provided the two significant failure modes are adequately stable. The results of the analysis are summarized in Table 1 below:

Table 1 – Stabilization Alternatives for Area 2

Top of Riprap [ft-msl]	Configuration	Factor of Safety Mid-Slope Failure	Factor of Safety Base Failure
59	No Bench	1.018	1.394
59	Bench	1.008	1.760
75	No Bench	1.191	1.553
75	Bench	1.214	1.595

Alternatives for Stabilization of Area 3

The slope in Area 3 is slightly flatter than 1H:1V and is characterized by exposed bedrock below approximately elevation 60 ft and unconsolidated material above elevation 60 ft. The east footing for Pier 3 is located at the upstream end of Area 3. High creek flows scoured the unconsolidated material at and downstream of Pier 3 for a distance of about 65 ft. There was no evidence of slope instability in Area 3, but the scour had removed backfill from around the pier above the footing. Continued scour, if allowed to occur, would likely expose or undermine the east footing of Pier 3. Two alternatives were developed and evaluated for preventing additional scour in the area:

- Alternative 3.1 – Shotcrete over the unconsolidated material and the bedrock, using threaded steel bars drilled and grouted into competent bedrock for anchorage.

- Alternative 3.2 – Riprap placed to the limit of work, graded to a 1.5H:1V slope, and anchored with wire rope mesh using threaded steel bars drilled and grouted into competent bedrock.

Because shotcrete is a relatively thin application of concrete, Alternative 3.1 results in a final grading that approximates the existing grades. Shotcrete is also comparatively rigid and therefore unable to adjust to minor changes in slope geometry that may result from slope creep, freeze-thaw effects or hydraulic loads. Furthermore, the upstream edge of the shotcrete, if exposed to flowing water due to scour upstream of Area 3, may be susceptible to sudden damage. By comparison, Alternative 3.2 is quite flexible and likely to remain effective even if minor changes to slope geometry occur. Additionally Alternative 3.2 can be graded to provide a hydraulically smooth bank that would transition evenly to the Area 2 grading. Alternative 3.2 improves the factor of safety of the slope from 1.028 for the existing slope to 1.218.

SELECTED ALTERNATIVE DESIGNS

An alternative was selected for each area based on the following criteria:

- Enhanced stability
- Constructability considering the limited access conditions at the site
- Risk of encountering unanticipated conditions that may delay the construction
- Providing a hydraulically smooth bank condition that would reduce scour effects
- Cost

Area 1 – Riprap Only

Alternative 1.1 was selected for Area 1. While this alternative provides very little improvement to the slope stability, other alternatives were not possible either due to limitations on site access, inability to construct without encroaching on the limit of work, or the feasibility of obtaining access permits within a reasonable time frame. Due to hydraulic design needs, the toe of the slope was designed with Class IX oversize riprap (maximum d_{50} of 48 inches), anchored to the bedrock with grouted 1-inch diameter galvanized steel bars, embedded about 3 ft into bedrock, and spaced according to the block size (approximately every 4 ft.) The riprap placed in this alternative was shaped to conform with the final grading in Area 2 (upstream) and transition to the natural grading downstream of the project area, resulting in a hydraulically smooth bank. Figure 9 provides a cross section showing the key elements of Area 1, including the oversize toe riprap.

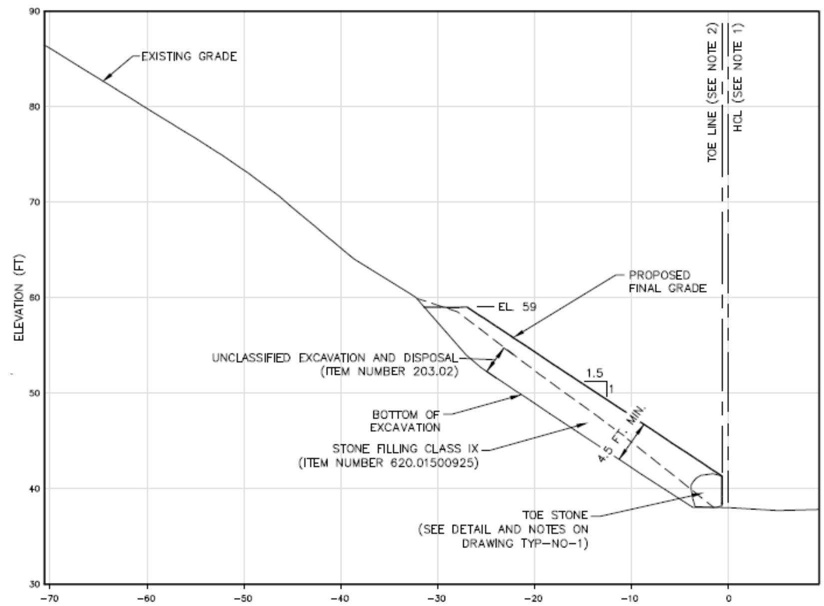

Figure 9 – Area 1 cross section (A-A') showing selected design elements.

Area 2 – Riprap to Elevation 75 with Bench

Alternative 2.2.3 was selected for Area 2. This alternative presents the best improvement in stability without introducing the risks associated with installation of cable anchors, is constructible with conventional earth moving equipment, and can be shaped to form a hydraulically smooth river bank. Figure 10 provides a cross section showing the key elements of Area 2, including the oversize toe riprap and bench at elevation 75 ft.

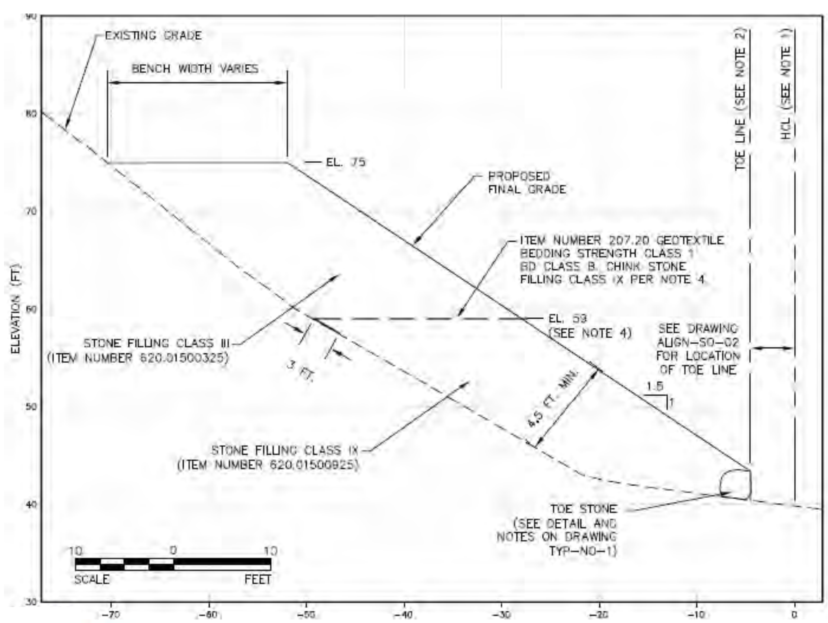

Figure 10 – Area 2 cross section (B-B') showing selected design elements.

Area 3 – Riprap With Tecco Mesh, Wire Rope (Spider) Mesh and Anchors

Alternative 3.2 was selected for Area 3. This alternative provides a smooth river bank and is not susceptible to sudden damage that might cause additional scour at Pier 3. The more robust Spider mesh was designed to supplement the Tecco mesh in order to provide additional stability to the riprap slopes under high water conditions, and to allow for a steeper slope in Area 3 such that the final repaired slope toe does not encroach on the river channel. Figure 11 provides a cross section showing the key elements of Area 3, including the bedrock anchors, riprap, Tecco mesh, and Spider wire rope, in relation to the east footing of Pier 3.

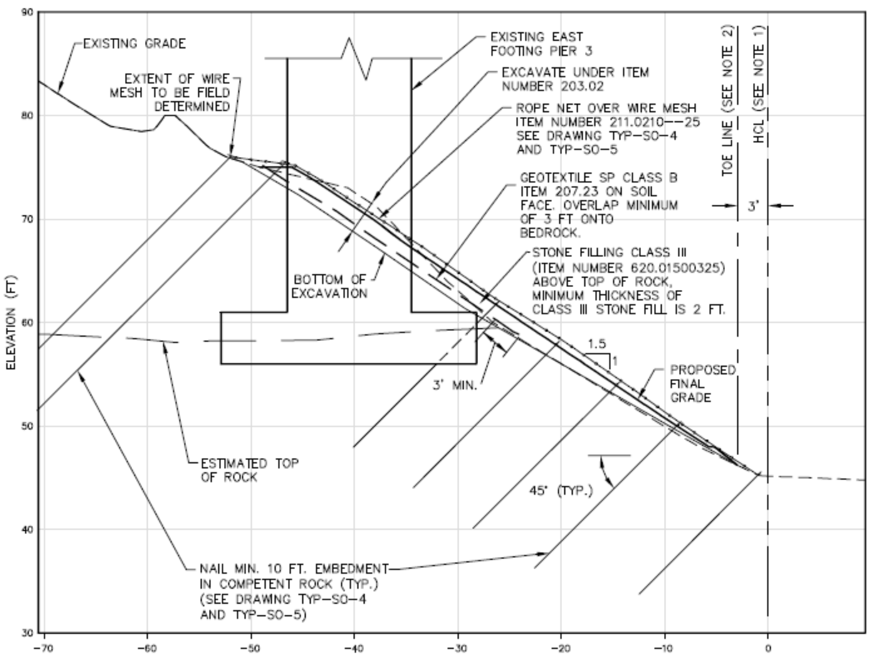

Figure 11 – Area 3 cross section (C-C') showing selected design elements.

CONSTRUCTION

Slope remediation construction was conducted between late November 2011 and early May 2012. Temporary river access was needed to build the riprap portions of the design, including doweling of oversize Class IX riprap at the toe. Drilling of the anchors was conducted with an excavator-mounted top hammer drill rig. Once the riprap slopes had been constructed, and the anchors had been installed and tested, the Tecco mesh and Spider wire rope systems were installed. Figures 12 through 17 provide photographs of the finished slope repairs.

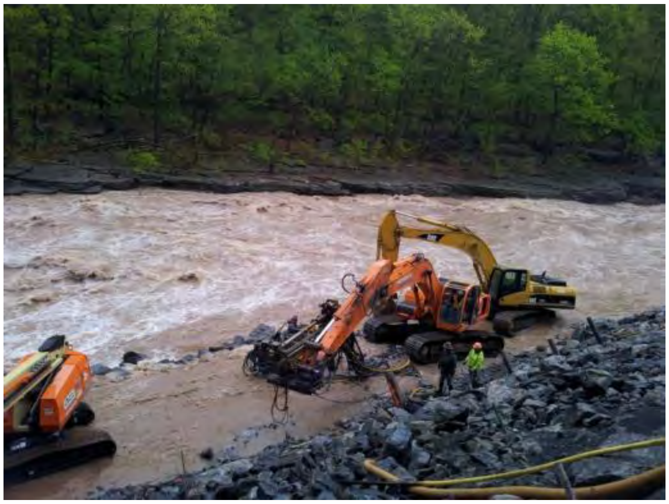

Figure 12 - Anchor drilling during spring runoff conditions April 2012.

Figure 13 - Installation of Tecco mesh and Spider wire rope over riprap in Area 3, May 7, 2012.

Figure 14 - Close-up view of rock anchor, Tecco plate, Tecco mesh and Spider wire rope, Area 3, May 7, 2012.

Figure 95 - Areas 2 and 3 complete, view to southwest, May 7, 2012. Note temporary access road in creek being removed.

Figure 16 - Area 1 complete, view to west, May 7, 2012. Note oversize Class IX riprap at toe.

Figure 107 - Areas 1 and 2 complete, view from bridge deck, May 7, 2012.

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PDA and Pile Restrike; A Better Understanding of Pile Resistances

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Disclaimer

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ABSTRACT

As KDOT continues to move forward using Load and Resistance Factor Design (LRFD) the utilization of high-strain dynamic pile testing is a fundamental step in generating our geotechnical recommendations. By implementing a PDA (Pile Driving Analyzer) during high-strain testing KDOT geologists and engineers have more confidence in the recommended bearing resistances. The goal for KDOT is to better understand pile resistances in various geologic settings to aid in reducing costs, reduce pile sizes and increase the loads needed to meet LRFD standards.

The current practice for PDA testing is to monitor piling to end of initial drive (EOID), and then perform short and long term restrikes. This current testing method has allowed KDOT geology to verify pile design resistances, and short and long term setup gains. Ultimately, KDOT anticipates establishing a new modifier for the ENR formula based upon data collected from PDA's and pile restrikes.

KDOT will utilize that PDA and restrike data in the design phase of future projects, thus taking advantage of the soil setup, reducing pile sizes, increase design recommendations to measured pile capacities, eliminate pile overruns, and expedite pile installation.

INTRODUCTION

In 2006 LRFD (Load and Resistance Factor Design) was implemented and Kansas Department of Transportation (KDOT) engineers and geologists have been utilizing high-strain dynamic pile testing to help estimate bearing resistances. These high-stain dynamic tests are used across the state and on a majority of KDOT bridges. The recommendations for these tests result directly from the geotechnical investigations, knowledge of the site condition, and cost effectiveness. By implementing a PDA (Pile Driving Analyzer) during a high-strain test KDOT engineers and geologists can have more confidence in the recommended bearing resistances. Ultimately, the goal for KDOT is to better understand pile resistances in various geologic settings to aid in reducing costs, reduce pile sizes and increase the loads needed to meet LRFD standards

GEOTECHNICAL INVESTIGATION

The state of Kansas can be broken down into eleven (11) different physiographic provinces. These regions are directly related to the physical geology. Each region has distinct characteristics which make foundation design extremely variable across the state. During the planning period and geotechnical investigation, knowledge of the geologic setting is a vital step in determining the type of foundation that may be utilized. Also the geologic setting will determine the equipment that will be needed to ensure that the best geologic data is obtained.

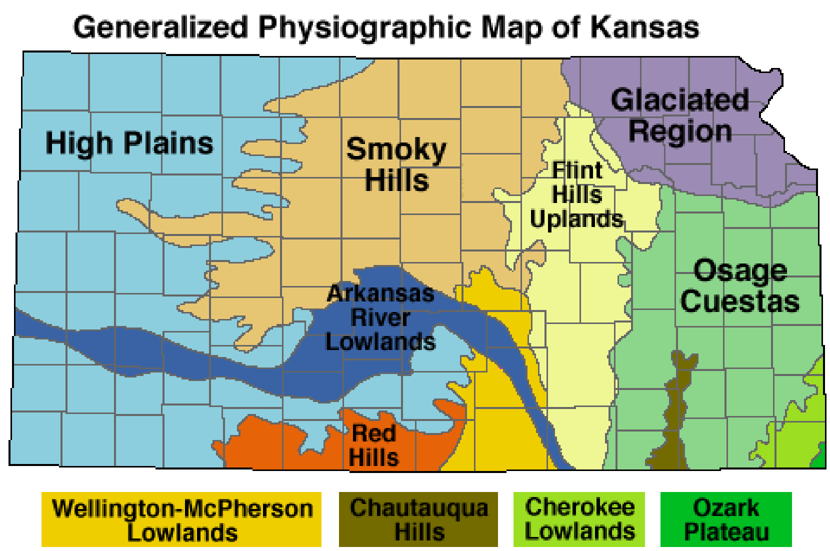

Figure 1

KDOT geology section currently has eleven (11) drill rigs that can be utilized on any geotechnical investigation and two (2) of these rigs are used to directly correlate a pile driving situation. These are a GeoProbe 7822DT, which records direct push data, tip resistance, pore pressure, soil profiles, and a calculated N_{60}, and a pneumatic hammer driven probe (Air Hammer), which is used to measure the time versus penetration. Both rigs have been used extensively in areas where high-strain dynamic pile testing is expected. Typically the majority of the projects where the GeoProbe 7822DT and Air Hammer are used are located within the western regions of the state. These regions commonly have bedrock depths exceeding 50 feet and high-strain dynamic pile testing is needed to ensure that the foundations obtain the designed capacity.

GEOLOGY REPORT AND RECOMMENDATIONS

KDOT projects typically have two (2) different types of geology reports issued. A surface geology report that describes the geologic units expected to be encountered on the project, backslope design and recommendations, and Volume Metric Factors (VMF). The other geology report typically issued is a foundation investigation report. The foundation report provides designers with geologic information, foundation recommendations, seismic site classification, Lateral-Loads, and hydrology issues. The foundation investigation reports when submitted are issued with an engineering geology sheet, which provides a plan and profile view of the proposed structure, geologic stratigraphy, boring locations, groundwater elevation, and test result data from samples collected during the field investigation.

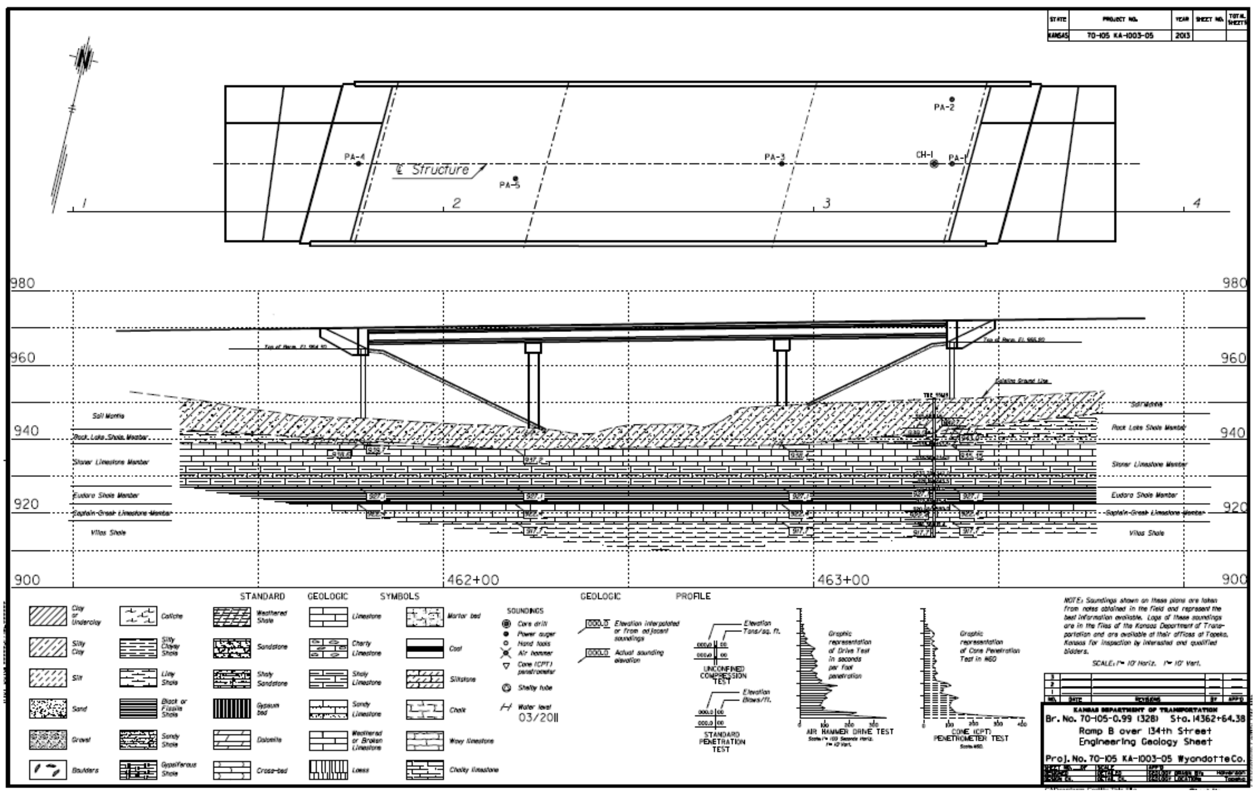

Figure 2

The foundation types that are recommended in the foundation investigation report range from spread footings to driven steel piles or drilled shaft foundations. The most common type of foundation across the state of Kansas is driven H-pile. Driven steel H-pile is used on nearly every bridge for abutment foundations and at pier locations where other foundation types are not economical. When pile foundations are recommended, KDOT geologists provide designers with recommendations for one (1) to three (3) different pile sizes. These recommendations include: anticipated pile tip elevation, expected pile capacity, phi factor, LRFD nominal and factored loads, pre-drilling if applicable, and the recommendation for a “Test Pile” or a “Test Pile Special”. The recommendations for “Test Pile” (non-PDA) or “Test Pile Special” (with PDA) are separate tests and may indicate some uncertainty where KDOT geologists think the driven pile will achieve the required resistance. But mainly “Test Piles” and “Test Pile Specials” are used for utilizing a higher phi factor.

PDA REQUEST

As KDOT projects are let and construction begins, contractors begin planning with the KDOT area engineers to conduct high-strain dynamic pile testing for where and when it is needed. Prior to the dynamic pile testing the contractor must provide information about the size and type of hammer used (e.g. diesel) to install the piling. Once the planning and information is gathered a request is submitted to the KDOT geology section to perform a high-strain dynamic pile test using a pile driving analyzer. KDOT geology section then mobilizes at a minimum two (2) geologists trained in high-strain dynamic testing. The geologists that are sent are familiar with the site geology, KDOT's testing method, required resistances, and the current equipment.

FIELD TESTS AND PDA EQUIPMENT

The field tests for driven pile on KDOT bridges can be as simple as counting the number of blows per foot and monitoring hammer ram stroke length or be as detailed as doing a PDA (Pile Driving Analyzer) tests. A field test recommended in the foundation report that does not include PDA instrumentation is referred to by KDOT personnel as a "Test Pile". This is simply a trained inspector observing the installation of a driven pile and recording the blows per/foot, stroke length of the hammer ram, and calculating the pile resistance using a Modified ENR (Engineer News Record) formula. After the "Test Pile" is driven and sufficient resistance is achieved, the driving criteria is set for the remainder of the pile for the structure. The other field test is high-strain dynamic pile test using a PDA. The PDA is an instrument that obtains data to measure resistance of the driven pile and other values obtained from the data collected by the pile-mounted strain gages and accelerometers. These tests are recommended in the foundation report with specific locations to help determine driving criteria across the proposed structure.

KDOT uses PDA equipment manufactured by Pile Dynamics, Inc. (PAX model). This model has allowed KDOT to use wireless accelerometers and strain gauges and has minimized the possibility of equipment damage that can occur with a wired system. Also, this PDA model has the capability to remotely connect to a PC at headquarters in real–time. This allows the geologists to view driving conditions from their office, which saves on travel time and expenses.

The installations of driven pile are monitored to the required resistances or a specified penetration depth, whichever is achieved first. Driving is stopped once the pile reaches the designated resistance or penetration, and an analysis of the PDA data is performed. KDOT geologists use PDI's pile wave analysis program CAPWAP to analyze and determine if the pile has met the required resistance specified by the designers. If the pile has not met the minimum resistance required by designers at the end-of-initial drive (EOID), the pile is then driven deeper to obtain resistances or allowed to sit for a designated amount of time and a restrike test is performed.

Figure 3

PILE RESTRIKE

A pile restrike test is a field test that is conducted on nearly all driven pile monitored with high-strain dynamic pile testing equipment. Restrike testing can be defined as impacting the top of pile with the driving hammer following initial installation for the purposes of confirming hammer ram stroke, pile set, pile integrity and/or geotechnical resistance (pile capacity). In some cases the restrike testing can be performed without PDA instrumentation if the goal is to confirm pile set and hammer ram stroke. On KDOT projects high-strain data is collected as part of the restrike testing to confirm pile capacity and to document time-dependent changes in capacity relative to that obtained at end-of-initial drive (EOID) testing.

By documenting capacity changes, KDOT could optimize pile length or pile sizes by relying upon time dependent capacity increases as the excess pore water pressures generated while driving the pile during initial driving are allowed to dissipate in fine grained soils such as silty sands or clays. For example, if a pile was required to have a nominal axial resistance of 300 kips, it may be required to install the pile to a depth of 80 feet below grade at a particular site to achieve the required capacity at EOID. However, a much shorter pile length may be employed if this same 300 kip capacity was allowed to develop over a period of several hours or days.

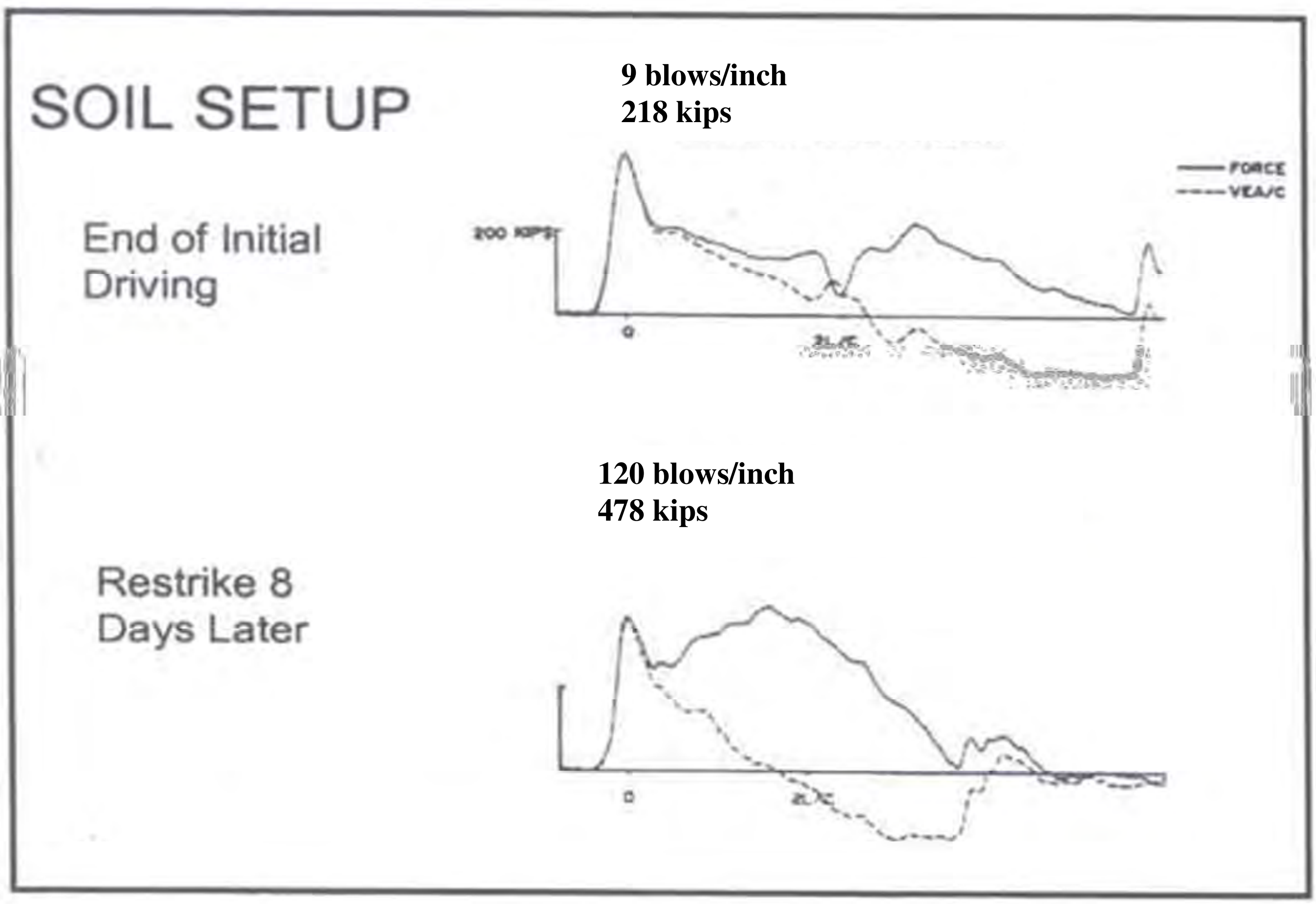

Figure 4

KDOT has implemented restrike testing on a number of projects where time-dependent capacity increases are expected, particularly for projects located in central and southwestern Kansas. KDOT geologists will perform restrike testing at one or more time intervals following EOID. The typical time periods range from 15 minutes to 3 days. Commonly restrike testing is performed at 15 min, 1 hour and at approximately 24 hours following EOID.

RESTRIKE PROCEDURE

When conducting pile restrike tests there are a number of different factors that may influence the test and data. Initially before restriking the pile, care must be taken to properly select a fuel setting for the diesel hammer to ensure that enough energy is imparted to the pile to mobilize the capacity, but not deliver too much energy that it would move the pile an excessive amount (over 3 inches). An unsafe, excessive stroke, or a resulting blow count that is unconservative for signal matching analysis (approximately 30 blows per foot or less) must also be avoided. Additionally, before impacting the pile, the diesel hammer must be warmed up by firing it 20 or 30 times on a pile located 30 feet or more from the pile subject to restrike testing. In cases where no other pile is available or is too close to the test pile, the contractor will typically impact the pile on a crane mat using no fuel or the lowest possible fuel setting.

After the hammer is warmed up and set on the instrumented test pile, reference marks are placed on the pile and the pile set is monitored over a 20 blow interval. Typically within the pile set it is recorded for every blow or every 5 blows. The pile set information is used in conjunction with the high-strain test data to confirm pile capacity and other relevant values. KDOT then issues the driving criteria to the contractor for installation of driven pile.

RECOMMENDATION POST PDA

Criteria for pile installation are given once all high-strain dynamic pile testing is completed. The recommendations for how the contractor is to install the production piling is given with detailed procedures that include: pile length, blows per/foot, hammer ram stroke, a target average pile set over the last 20 blows, minimum and maximum pile elevation and an ENR bearing target. If a production pile is installed under conditions that do not meet the recommended driving criteria for the planned tip elevation, KDOT may elect to confirm whether required capacity has been met during a later restrike test versus increasing the pile installation depth.

USING PDA DATA

The data collected through high-strain dynamic pile testing that has been conducted on KDOT projects is allowing geologists to better predict and verify pile capacities, and understand short and long term pile setup.

Currently, the data that is obtained during the EOID and restrikes tests are used in a logarithmic calculation to predict pile setup over time. This calculation uses the EIOD and interval restrike data to predict the short and long term pile capacities. By utilizing the EOID and restrike data to the predicted pile setup capacities, KDOT expects to take advantage of time dependent capacity increases to reduce overrun pile lengths, expedite pile installation, and in the future reduce the piles sizes that are being employed on KDOT projects.

PDA Capacity vs. Time Graph

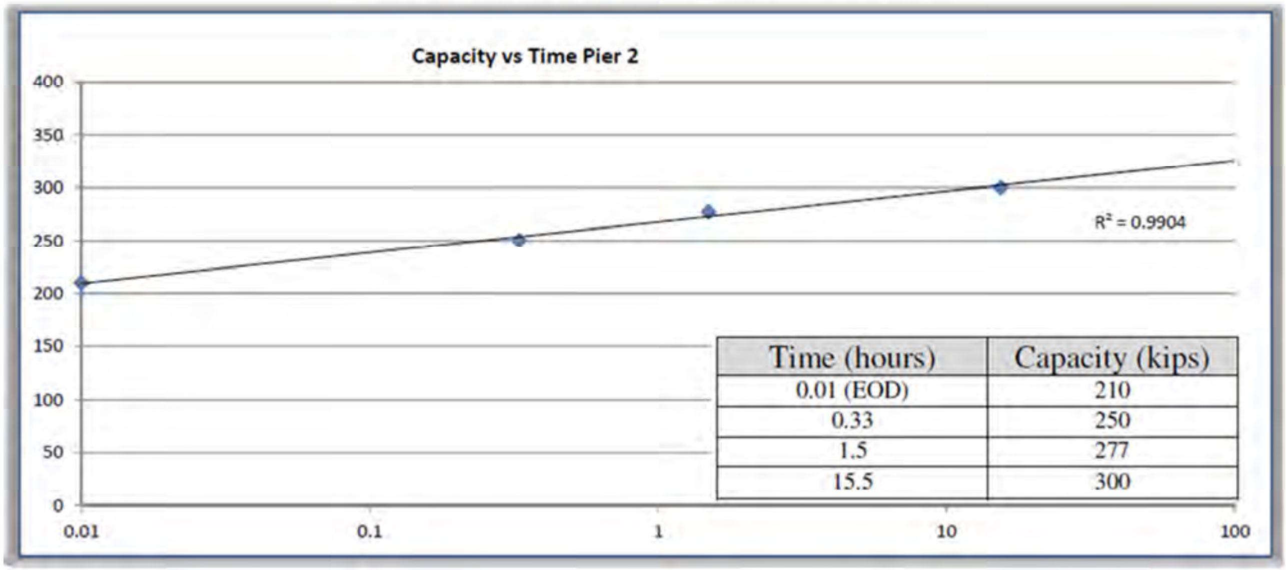

Figure 5

In conjunction with the field application of high-strain dynamic pile testing, KDOT has developed a database that allows geologists to query PDA data in over 20 different fields, on over 246 piles, and from 56 different projects. Currently, the database is being maintained and updated by the University of Kansas (KU). This partnership between KDOT and KU is to help analyze the current, historical, and future data and develop a new modifier to the ENR formula for field personnel that more closely relates to the PDA data and the actual conditions that are encountered across the state of Kansas.

CONCLUSION

The high-strain dynamic pile testing that is conducted and utilized on KDOT projects allows geologists to verify and have more confidence in pile capacities. By recommending a PDA test geologists can take advantage of higher phi factors, have more confidence in driven pile recommendations, and have verification of pile capacities. However, the ultimate goal for implementing high-strain dynamic pile testing for KDOT is to apply the information that has been obtained with the Pile Driving Analyzer and become more efficient and cost effective. This can be accomplished by utilizing the data collected during the end-of- initial-drive (EOID) and the intervaled restrikes to calculate the time dependent pile capacity increases seen in KDOT projects. Also by taking advantage of the capacity that develops over time, KDOT is optimistic that pile designs and installations will become more efficient.

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Cooperative Geotechnical Designs to Build on Liquefiable and Compressible Soil in Salem, Massachusetts

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ABSTRACT

The Massachusetts Bay Transit Authority (MBTA) is addressing accessibility throughout their facilities. The Commuter Rail station in Salem, Massachusetts is upgrading their facility to improve site accessibility and increase parking capacity. Proposed improvements include a parking garage replacing the existing parking lot, a pedestrian bridge replacing the existing stairway connecting track level with downtown Salem, and a full-length high-level platform. Historical records, a geophysical survey, and an archaeological survey indicate structural remains from an historic train depot are largely intact beneath the surface of the existing lot.

Subsurface explorations encountered fill overlying loose saturated sands above 40 feet of soft, compressible marine clay deposits extending to competent argillite rock at 60-80 feet below grade. Deep foundations bearing on rock were recommended for structural support of the garage, bridge, and platform. Potentially liquefiable sands, the potential for lateral spreading, and a poor seismic site classification exist at the site. Ground improvement techniques were recommended to improve the subsurface soil conditions and limit liquefaction and lateral spread potential. Several value-engineering options were explored, including options to replace traditional deep foundations with drilled displacement columns for garage support, using shallow retaining wall foundations for platform support, and using a slab-on-grade instead of a structural slab. The resulting cooperative designs required additional coordination between the design team to maximize efficiency of the project budget.

INTRODUCTION

The Massachusetts Bay Transportation Authority (MBTA) is addressing accessibility throughout their system and facilities. One of these facilities is the Commuter Rail Station in Salem, Massachusetts, which serves more than 2000 passengers daily and hosts regular MBTA bus service. The MBTA has planned a full-length high-level ("full-high") platform for access to commuter rail for this station, which will provide at-grade access into the commuter train. In addition to the platform upgrades, the MBTA is planning to build a parking garage to provide additional parking capacity at the station, as well as a pedestrian bridge that will enhance accessibility from downtown Salem to the station. The new site improvements will also include a passenger drop-off/pick-up area and bicycle parking, and will improve traffic flow patterns for buses and taxis. The garage will increase parking capacity from the current 340 parking spaces to about 700 spaces. An enclosed waiting area in the garage and platform canopies will offer shelter for passengers accessing the train along the new platform. Figure 1 depicts a rendering of the proposed garage and platform.

(Image credit: Fennick McCredie Architecture)

Figure 1 – Architectural rendering of the proposed parking garage and high-level platform (facing south)

SITE DESCRIPTION AND HISTORY

The site is a triangular parcel of land located immediately north of Bridge Street, on the northern edge of historic downtown Salem, Massachusetts (see Figures 2 and 3). The site is bound to the south by Bridge Street, to the southwest by a parking lot owned by the City of Salem, to the north by a seawall with the North River beyond, and to the east by a residential condominium property. The MBTA commuter rail platform and railroad tracks are along the eastern edge of the site. Another line of railroad tracks pass along the northern edge of the site.

(Image credit: ESRI USA Topo Maps, United States Geological Survey)

Figure 2 – Site Location

The site is generally asphalt paved and serves as a surface parking lot for commuters with 340 spaces. Existing site grades typically range from approximately El. 9 to El. 10.5 ft, National Geodetic Vertical Datum of 1929 (NGVD 1929). Street grade at Bridge Street is approximately El. 27 ft and the MBTA commuter rail line passes under Bridge Street through a tunnel. The existing grades of the commuter rail platform range from El. 10 ft at the north to El. 4 ft at the south with a retaining wall between the parking lot and the platform. Pedestrian access from Bridge Street is provided by a stairway at the south east corner of the site. The current site layout is shown in Figure 3.

The existing commuter rail platform is near track level, except at the northeast corner of the site, where a mini-length high-level ("mini-high") platform exists with a ramp for accessibility. There are two canopies along the existing MBTA commuter rail track that also serve as bus stops.

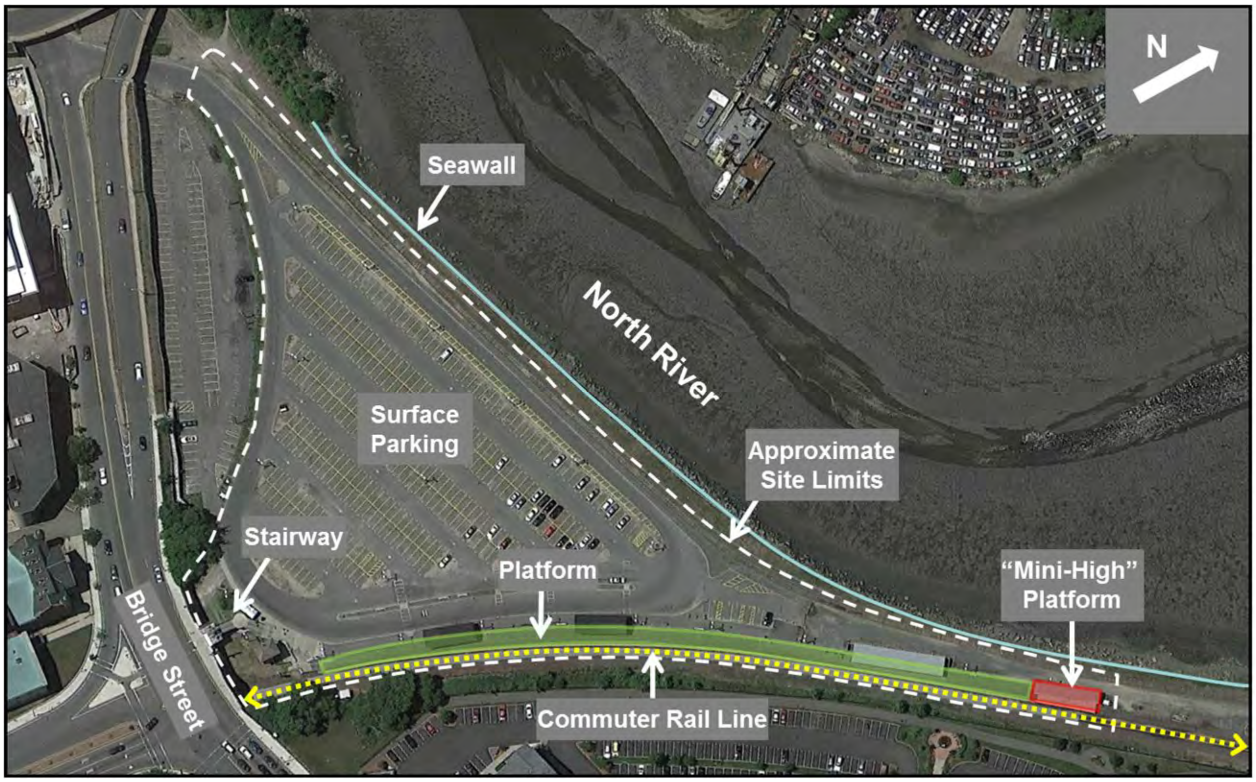


Figure 3 – Existing conditions at the site

Prior to use as a Commuter Rail station/parking lot, the site served as the Salem Train Depot with maintenance facilities for locomotives. Site history, photos, and fire insurance (Sanborn) maps were acquired dating back to 1890, when turntable and roundhouse structures stood amidst a number of tracks leading to and from the maintenance facilities. The foundations of these facilities remain buried in place.

SUBSURFACE CONDITIONS

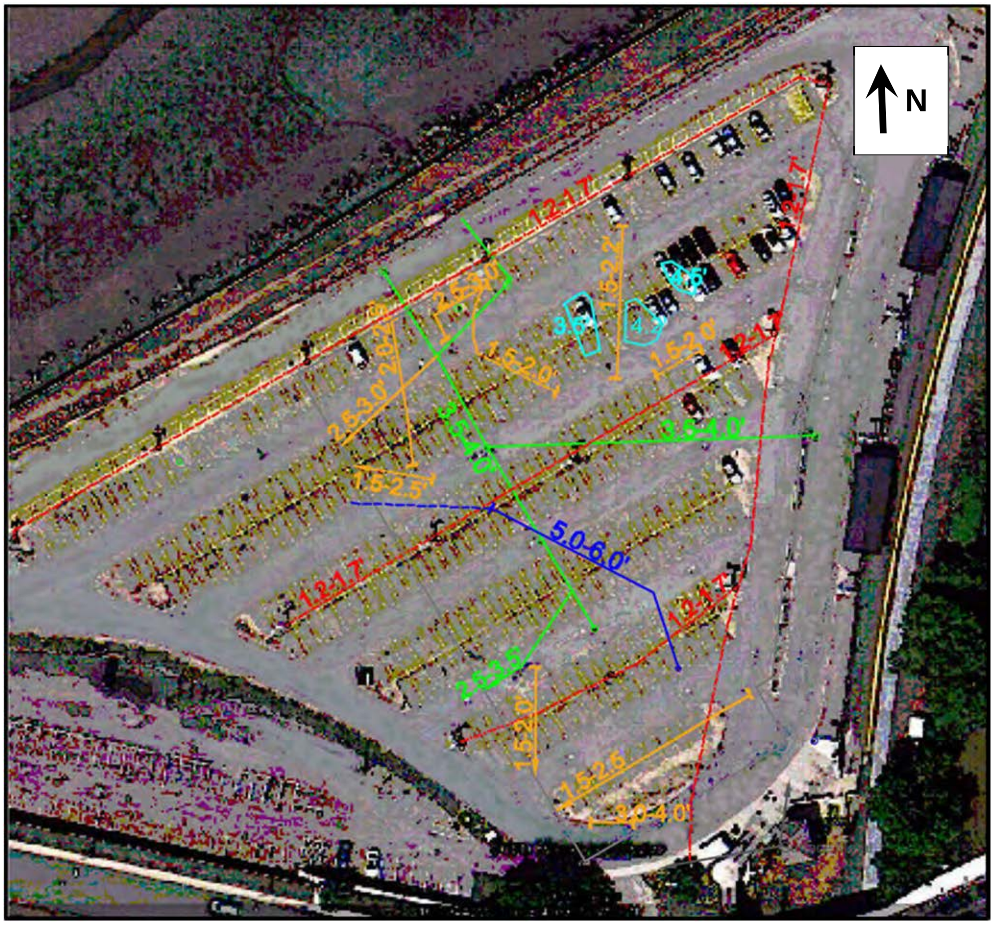
Site Geology

The site is located in the New England Seaboard Lowland Section Physiographic Province of the northeastern United States and is reportedly underlain by members of the Milford-Deham Zone intrusive rocks including the Proterozoic Z-aged Diorite and Gabbro (Zdigb) and the Ordovician gabbro at Salem Neck (Ongb) based on mapped bedrock conditions in the Bedrock Geologic Map of Massachusetts *(1)*. These rocks consist of a complex of diorite and gabbro, with intrusives including granite and granodiorite. However, the bedrock encountered during the subsurface explorations at the site was more consistent with the lower Silurian-to upper Ordovician-aged Beverly Syenite of the quartz-poor facies of the Cape Ann Complex mapped within the area. This formation reportedly consists of a cream colored, medium to coarse grained syenite, rich in alkali feldspar. Dikes of these intrusive rocks reportedly intrude gabbro at Salem Neck, as well as elsewhere in the Salem area.

Based on mapped soil conditions in the Surficial Geologic Map of Northeastern Massachusetts *(2)*, the site is underlain by artificial fill. The materials reportedly consist of earth materials and/or manmade materials that have been artificially placed in areas such as urban developments and filled wetlands. The artificial fill is underlain by glaciomarine fine deposits that are also mapped within the North River estuary. These materials reportedly include silty clay, fine sand, and some fine gravel deposited along the coast and in river estuaries. The upper portion of these materials reportedly consists of fine to very fine sand, grading downward into interbedded very fine sand, silt, and silty clay. Lower silty clay and clay is massive and thinly laminated. Total thickness of these deposits is reportedly up to approximately 75 feet. These mapped conditions were generally consistent with the conditions encountered during the subsurface evaluation.

Geophysical Survey

During the site history review process it was suspected that foundations of the roundhouse, turntable, and other appurtenant structures may have been abandoned in place and if so, they would have archeological significance. A geophysical exploration was performed using precision utility locating methods (PUL), time domain electromagnetics (EM), and ground penetrating radar (GPR) techniques to explore their presence. The GPR and PUL results and the EM results for the site are shown in Figures 4 and 5, respectively.

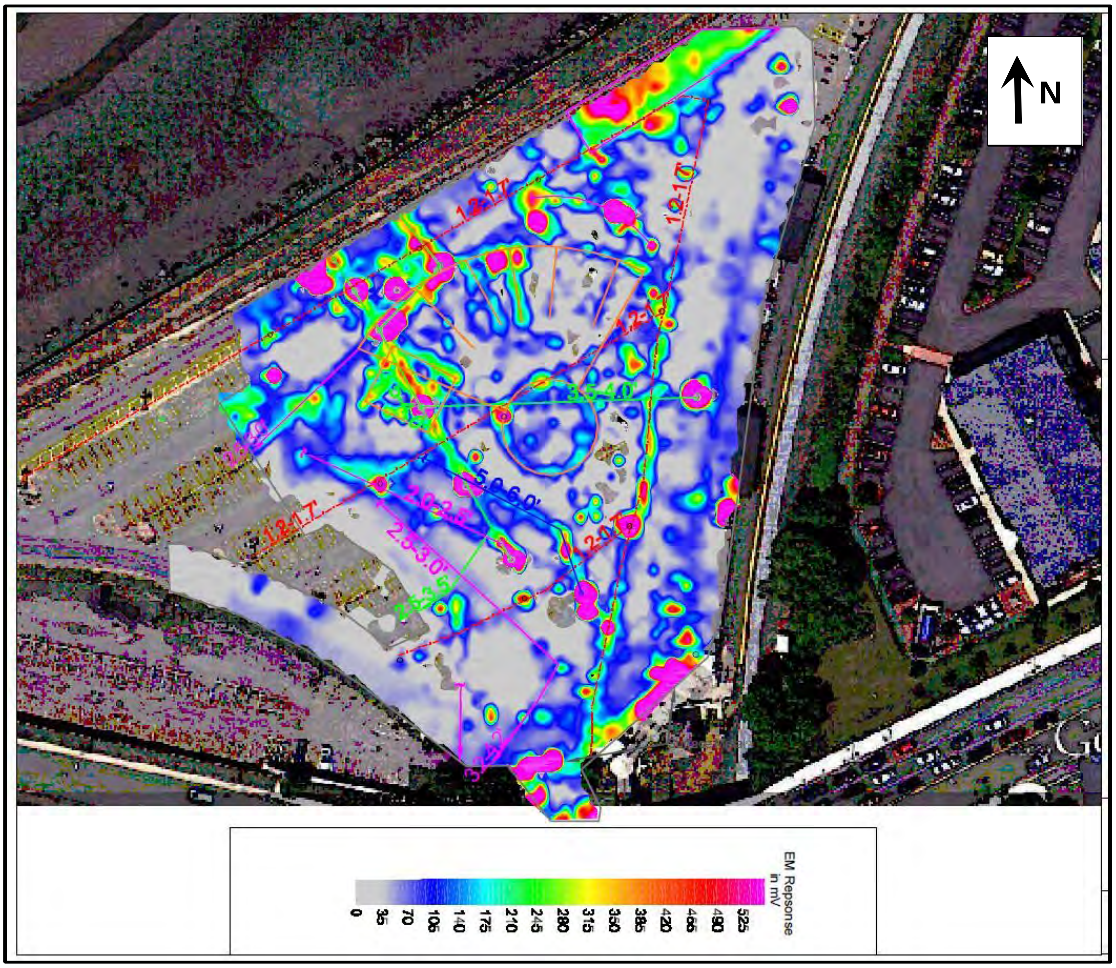


(Image credit: Hager GeoScience, Inc.)

Notes: Solid lines indicate existing utilities (drainage, water, electric, other); Blue polygons indicate anomalies, possibly indicating other buried metals

Figure 4 – GPR and PUL Survey Results

The EM results (Figure 5) suggested the presence of subsurface structures resembling the historical turntable structure and also showed strong correlations with the locations of the roundhouse and turntable structures indicated on the Sanborn maps.



(Image credit: Hager GeoScience, Inc.)

Figure 5 – EM Survey Results

Results of the EM and GPR results together provided good indications that the features identified in the geophysical surveys correspond well with the locations of the historical structures. The results of the geophysical survey reinforced that buried structural remnants are likely present on the site and would require historical cataloging to meet site permitting and historical commission requirements in addition to extra consideration for construction methods and sequencing.

Archaeological Exploration

Based on the geophysical survey results, a full archaeological excavation and survey was performed to catalog and record the historical findings and the subsurface structural and cultural remnants. Figure 6 is a photograph of the site during the archeological excavations, showing the turntable and roundhouse foundations.

(Image credit: The Public Archaeology Laboratory, Inc.)

Note: Photo shows turntable foundation in the upper left, and roundhouse engine stall maintenance bays arcing around the turntable.

Figure 6 - Photo of archaeological exploration (facing southwest)

The archaeological survey of the site exposed the locations and extents of many of the existing subsurface structures, many intact, including granite, concrete, and brick foundations, rails, rail ties, and other structural and cultural materials. The archeological survey results were incorporated into the project design and construction recommendations.

Geotechnical Explorations

Geotechnical explorations at the site were performed in three phases with a total of 21 borings performed in November 2011, March/April 2012, and September 2012 using a combination of mud rotary and rotary wash drilling methods. The test borings extended to depths ranging up to 100 feet below ground surface. The first phase of borings was performed for a preliminary site characterization. The second phase of borings was performed for the garage and platform structures, generally within or near the proposed structure footprints. The third phase of borings was performed specifically to assess the extent of potentially liquefiable soils on site. Approximate locations of the test borings are shown on Figure 7.

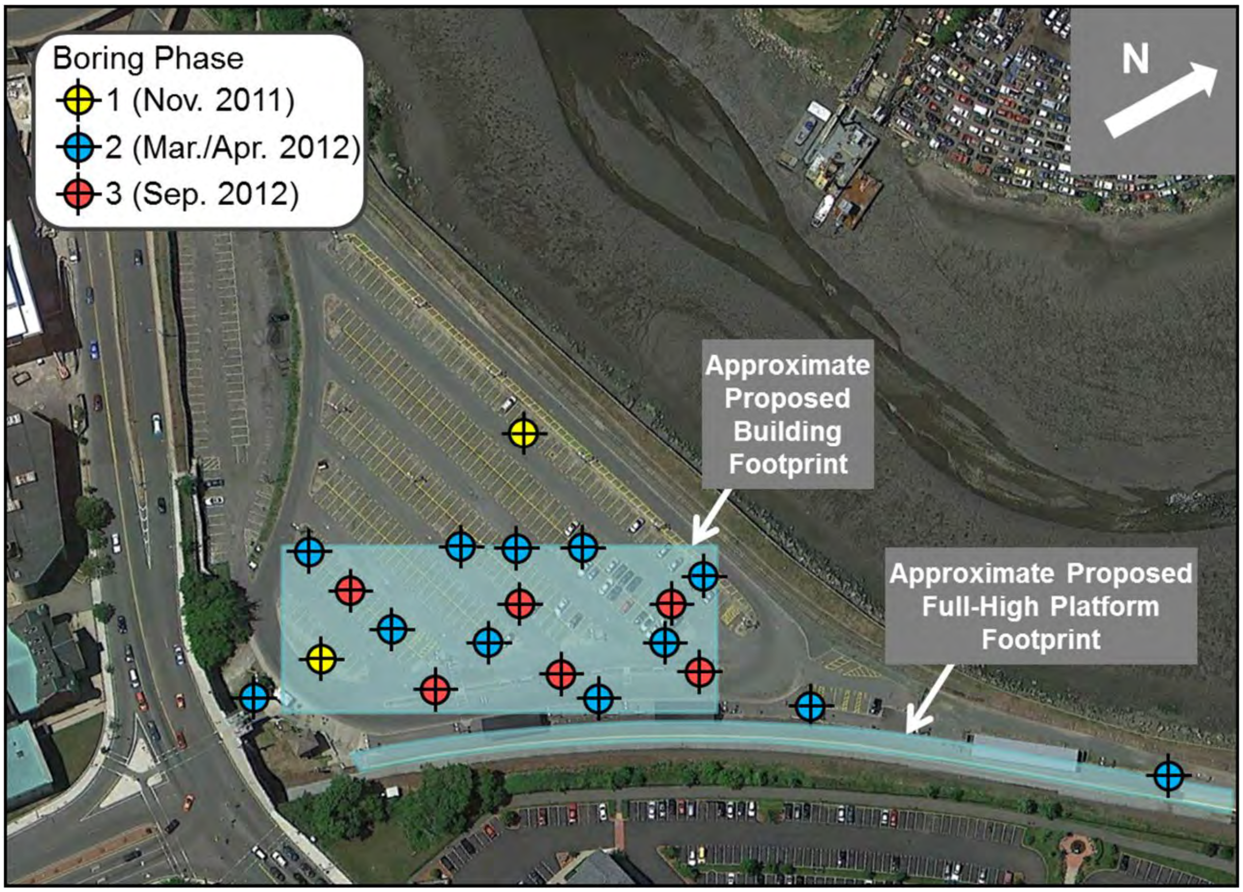


Figure 7 – Boring Locations

The geotechnical explorations generally indicated a layer of fill extending to between 5 and 15 ft below ground surface overlying a layer of loose sand and silt deposits between 0 and 20 feet thick. A discontinuous layer of organics was encountered across the site beneath the sand layer, overlying a thin layer of sand or overlying soft marine clay deposits. The soft marine clay deposits range from about 30 to 60 feet thick and extend to a relatively thin layer of glacial till (2 to 10 feet thick). Rock or weathered rock was encountered between 60 and 80 feet below ground surface and generally increased in depth from south to north across the site. A generalized soil profile for the site is depicted in Figure 8.

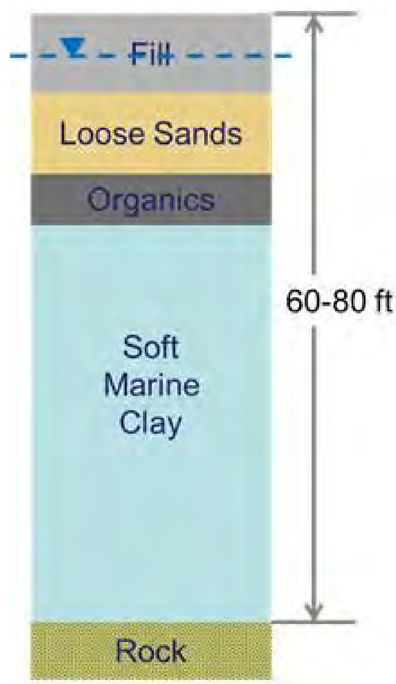


Figure 8 – Generalized soil profile

Groundwater was recorded at approximately 5 ft below ground surface. Based on the shallow groundwater level and the presence of loose, saturated sands and silts, the site is potentially liquefiable when subjected to earthquake ground movements.

GEOTECHNICAL DESIGN

Background

The proposed garage building is a five story concrete building with no below grade space. Column loads are up to 2200 kips and the building period in one direction is 0.56 seconds. The ground floor of the building is 3 to 6 ft above existing grade. The proposed platform and canopy loads are up to 26 kips including the raise in grade. The proposed high-level platform is 4 ft above existing platform levels, and has a low settlement tolerance due to accessibility concerns.

Design recommendations were provided in accordance with the provisions of the Massachusetts State Building Code, 8th Edition which amends the International Building Code 2009. Based on the Building Code, the site has a seismic site classification of F due to the presence of the potentially liquefiable sand layer and the structure period. The controlling earthquake magnitude was estimated to be 5.8 using an S_{DS} and PGA (per the Building Code) of 0.477g and 0.19g, respectively, and the 2008 USGS interactive deaggregation tool.

In addition, with the proximity of the site to the North River (the northern border of the site is protected by a 12 foot high seawall) and the elevation of the loose saturated sands, the potentially liquefiable sand layer also presents a risk of laterally spreading into the river under earthquake loading. Lateral spreading is a post-liquefaction phenomenon consisting of blocks of soil “laterally spreading” due to either a gently sloping ground or an open face such as an open creek or river channel. During lateral spreading, blocks of non-liquefied soil "float" on top of liquefied soils below. Lateral spreading has been observed in previous large earthquakes, even for gently sloping sites, at distances of over 500 feet from a free face. Lateral spread movements are typically greatest near a free face (such as creek or river channels) and diminish with distance from the free face. Potential for lateral spreading is high especially at the northern portion of the site where the proposed garage and platform are nearest to the North River. Laterally spreading soils can induce significant lateral deformations and increased lateral loads on deep foundations. In many cases, the deformations and increased loads exceed the allowable limits.

Design Recommendations

Deep foundation systems extending to bedrock were recommended for support of the proposed garage, as well as the platform, canopy, and pedestrian bridge structures due to the soft/loose ground conditions encountered in the test borings, planned increase in grade, and the project’s settlement tolerance. Drilled shafts and a structurally supported ground floor slab were recommended for the garage building, using the capacity of the rock to support the structural loads because of the potential for large consolidation settlements in the relatively thick soft marine clay layer and seismic settlements in the upper sandy soils. Driven H-piles were recommended for the platform, canopy and bridge structures. The archeological findings were not required to be left intact and the structural remnants encountered within the proposed garage footprint were recommended for removal.

Due to the potential of lateral spreading at the site and structural requirements for the garage building, the use of ground improvement techniques was recommended to mitigate the lateral spreading hazard and improve the seismic site classification for the garage building from Site

Class F to Site Class E in accordance with the Massachusetts State Building Code. Vibro-replacement stone column methods were selected as the preferred ground improvement technique, which reinforce the treatment zone and increase the soil strength through densification and replacement. Completed stone columns create a rigid column with densified surrounding soil and act as drains to assist in relieving pore water pressure buildup during earthquake shaking. The stone columns will be installed to various depths within the building footprint and at the northern end of the platform.

With the need for ground improvement at the site and the project's budgetary limits, the project team was challenged to make adjustments to the design. The project team worked together to provide alternative systems which consisted of the following:

- Recommendations for an alternative foundation system for the building consisting of drilled displacement columns (DDC) were provided. In DDC systems, soils displace radially around a displacement tool (a purpose-built auger head), densifying the soils around the displacement tool point. Grout is injected under pressure as the displacement tool is withdrawn. With the densification and injected grout, the soil strength and stiffness increases. The DDCs are not structurally connected to a building and therefore are a type of ground improvement system such that the building can be supported on shallow footings founded on top of the ground improvement DDC elements. Recommendations for both the alternative DDC option and the original deep foundation design were provided for the garage building in the contract documents.

- A flexible asphalt surface with more flexible flatwork and expansion joints was recommended to allow for ground floor settlement within the building footprint. Design recommendations were provided for this adaptive design and planned maintenance (future leveling from long-term settlement).

- For support of the platform loads, an alternative foundation design using a shallow retaining wall option instead of deep foundations was explored, with the understanding that using a shallow foundation increases the risk of intolerable settlements; a deep foundation system minimizes differential movement between the platform surface and train. Two shallow platform foundation options were explored including 1) a continuous retaining wall consisting of a cast-in-place continuous footing, stem wall, top slab cantilever, and an at-grade bituminous pavement platform, and 2) cast-in-place, isolated spread footings, and concrete pedestals supporting a pre-cast concrete platform. Design recommendations and estimated settlements for each option were provided to the client for their use in evaluating the final design options.

Each of these alternative design solutions required the full collaboration of the geotechnical, structural and architectural design teams and the Contractor. The project continues to require cooperation between the design team members along with the MBTA and the Contractor to provide an economical and structurally sound design option.

CONCLUSION

The Salem Commuter Rail station is in need of more site improvements to relieve congestion with additional parking and improve pedestrian access to serve its residents and visitors. Some of the proposed site improvements include a parking garage, pedestrian bridge, and new train platform. Deep foundations were recommended using the capacity of the rock to support the structural loads because of the potential for large consolidation settlements in the thick soft marine clay layer encountered at the site. Drilled shafts and a structural slab were recommended for the garage building.

With the potential for liquefaction and lateral spreading at the site, and the project's budgetary limits, the project team was challenged to make adjustments to the design. Ground improvement measures were recommended to mitigate a lateral spreading potential and to improve the seismic site classification for the garage building from Site Class F to Site Class E. The project team worked together to provide an alternative foundation system consisting of drilled displacement columns instead of the deep foundation options. Similarly, the platform was designed for shallow foundations as an alternate to deep foundations. The garage ground floor was changed from a structural slab to a flexible asphalt surface with an adaptive design to allow for ground floor settlement. The garage structure and pedestrian bridge will be founded on either drilled shafts socketed into rock, or on the alternative drilled displacement columns, and the train platform and canopy structures will be a cast in place retaining wall with a continuous shallow foundation.

The project continues to require cooperation and consistent efforts by the design team along with the MBTA and the Contractor to provide the most economical and structurally efficient design option. Construction begins in summer 2013, and the new facility is slated to open in October of 2014.

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Cellular Geosynthetics in Highway Applications

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ABSTRACT

Expanded polystyrene (EPS) is a closed-cell, polymeric ('plastic') foam. It was invented circa 1950 and is now a commodity material that is manufactured worldwide for numerous, diverse commercial applications. In its generic block-molded product form (EPS-block), it is the geofoam material and product of choice as lightweight fill for earthwork construction such as highway embankments on soft ground. It has been used for this geosynthetic-functional application for over 40 years since the first documented project in Norway in 1972. This mature, well-established geotechnology is now widely known and used worldwide, with exponential growth occurring throughout the U.S. and Canada during the past 20 years.

However, there are many other potential functional applications and uses of not only EPS-block geofoam but a broader range of cellular-geosynthetic (geofoam and geocomb) materials and products in highway-related applications that are less well known and used to date. This paper highlights these lesser-known capabilities of cellular geosynthetics that have already been used and proven in practice and may be of interest to geo-professionals involved in transportation-related projects. Also presented in this paper are highlights of new developments related to the well-known and established uses of cellular geosynthetics such as the use of EPS-block geofoam for soft-ground applications.

Particular topics of relevance and interest addressed in this paper include presentations and discussions of:

- results from the latest National Cooperative Highway Research Program (NCHRP)-funded research into broader uses of EPS-block geofoam in slope stabilization, not limited to soft-ground conditions. This research included development of an updated version of the first-of-its-kind material and construction standard developed a decade earlier as part of the original NCHRP-funded research into embankments on soft ground
- reduction of lateral earth pressures behind both new and existing earth-retaining structures of all kinds, e.g. free-standing retaining walls, conventional jointed-bridge abutments, and integral and semi-integral bridge abutments
- compressible inclusions to reduce both vertical and horizontal stresses on structures from expansive soil and rock
- control of seasonal ground freezing beneath pavements and behind earth-retaining structures
- protection of rock and snow sheds from slide and other falling debris
- important issues concerning failures in project applications; manufacturing and construction quality; and material standards and generic construction specifications that have emerged as hot-button issues throughout the U.S. in particular in recent years.

INTRODUCTION

The defining feature of materials or objects with a cellular structure, whether open- or closed-cell, is that they contain relatively small volumes of solid material per unit volume of the overall material or object. The distinguishing physical properties that result from this are:

- Low to very low density/unit weight compared to solid materials or objects.
- Relatively substantial strength and stiffness due to a structurally efficient arrangement of the solid-material fraction.
- Relatively large void volume in which to contain fluids (gases and/or liquids) and/or other solid materials. The contents of the voids can both independently and synergistically contribute to the 'mechanical' (stress-strain-time-temperature), thermal, chemical, and/or biological properties of the overall material or object.

Thus it is no surprise that nature has evolved to utilize cellular structures in a wide variety of ways with insect-constructed honeycombs being an example that is both well-known and easily viewed.

CELLULAR GEOSYNTHETICS

Given the inherent efficiency and diversity-of-use of the cellular structure, it is no surprise that humans have replicated this structure both by happenstance and intention in manufactured materials and products developed for use in a wide variety of commercial applications. Of interest and relevance to this paper are the use of cellular materials and products as *geosynthetics* where the solid fraction may constitute as little as 2% of the total volume, i.e. a porosity, n, of 98%. However, as in nature this solid fraction is arranged so that the overall material or product can have remarkable stiffness and strength if desired despite its very low density. In addition, the significant void space in cellular geosynthetics can sometimes be used productively to store fluids or soil particles.

The evolution of *cellular geosynthetics* revolutionized geosynthetics technology because of their inherent three-dimensional (3-D) structure compared to the essentially two-dimensional (2-D) structure of traditional 'planar' geosynthetics such as *geotextiles*, *geomembranes*, and *geogrids*. This 3-D structure, referred to as 'thick geosynthetics' in early publications (*1*), allows cellular geosynthetics to not only provide several geosynthetic functions unique to them but, in some cases, interact synergistically with planar geosynthetics to provide functions neither 3-D nor 2-D geosynthetics alone could provide.

SCOPE AND ORGANIZATION OF PAPER

Cellular geosynthetics, which includes *geocells*, *geocombs*, and *geofoams*, is a broad topic and the subject of books (*2*) and other publications so there must be a focus to limit this paper to the available length. This paper is limited to geocombs and geofoams which not only have the broadest potential use in highway applications but have seen their use increase dramatically worldwide beginning in the 1980s and continuing to the present. As a result of this technology expansion in recent years, most geo-professionals are now at least somewhat familiar with the more common geofoam materials such as *block-molded expanded polystyrene* (EPS-block) and *cellular* (a.k.a. *foamed*) *Portland-cement concrete* (CPCC) and their now-routine use as lightweight fill, especially in road construction on soft ground.

However, geofoams and geocombs offer many more geosynthetic functions and potential highway applications than these. In addition, there have been some significant recent developments with regard to the well-known use of EPS-block geofoam as lightweight fill. Therefore, this paper will focus on the broader highway-related applications of geofoams and geocombs as well as discuss recent developments for the already well-known applications.

With regard to the organization of this paper, geofoams are discussed first followed by geocombs. For each of these geosynthetic categories, basic definitions and terminology are reviewed, followed by a brief review of available materials and products, and then an update on current activities related to their well-known use as lightweight fills in road construction. This is followed by a discussion of other functional applications of proven or potential use in highway applications.

Those seeking more detail than can be presented in this paper will find a detailed treatment of geofoams in (*2*), with summaries in (*3*,*4*). The bibliography in (*5*) is useful as a starting point for more-advanced study and research, and is also a reference source for topics not explicitly cited in this paper.

GEOFOAMS

Definition and Terminology

Although most geo-professionals have at least heard the term 'geofoam', misconceptions about its definition are widespread and, unfortunately, continue to the present. Therefore, before proceeding further it is necessary to set the record straight in this regard.

Since the early 1990s, 'geofoam' has been defined as the generic term for any synthetic geomaterial created in an expansion process using a gas called a *blowing agent* and resulting in a texture of numerous closed cells. Therefore, 'geofoam' is not just one material or product as some believe. It is actually a very diverse family of many different kinds of materials and products. A summary of geofoam materials identified to date is presented in the following section.

It is relevant to note that the term 'geofoam' has been and is still used at various times and in various geographic regions for consumer products that have nothing to do with geosynthetics or construction. In addition, in the past 'geofoam' was a U.S. registered trade mark for a now-defunct proprietary commercial product that was marketed and used almost exclusively in Alaska in the 1970s, and from time to time 'geofoam' has continued to be used as part of a registered trade mark for various products, most of them having nothing to do with geosynthetics. Nevertheless, for over 20 years now, at least as far as geosynthetics are concerned it has been and continues to be used generically as defined in the preceding paragraph.

Materials

Several proven geofoam materials exist. There are additional materials that have been tried over the years but were found to be technically unacceptable. The latter are not listed here but are discussed for their historical relevance in (*2*).

Geofoam materials can be divided into three major groups:

- polymeric ('plastic')
- cementitious, typically using Portland cement (*cellular* or *foamed concrete*)
- vitreous (*cellular glass*).

The polymeric category is further subdivided based on polymer chemistry. The various polymeric geofoams, with the names and acronyms used in U.S. practice, are:

- *rigid cellular polystyrene* (RCPS)
- *polyethylene* (PE)
- *polyethylene-polystyrene* (PE-PS) blend
- *polyurethane* (PUR).

RCPS is the only polymeric geofoam that is subdivided further based on the explicit expansion process used to achieve the cellular structure:

- *expanded polystyrene* (EPS, formerly known as *molded expanded polystyrene*, MEPS)
- *extruded polystyrene* (XPS, formerly known as *extruded expanded polystyrene*, XEPS).

It is of significant relevance and importance to comment on some colloquial terms that are common in the U.S. First and foremost is the near-universal tendency to refer to all polymeric foams as *styrofoam*. This is and has always been incorrect as *STYROFOAM*™ is the trade name of a line of XPS/XEPS products manufactured by The Dow Chemical Company. A very simple rule to use is that unless a polymeric foam is colored blue it is not *STYROFOAM*.

The second term to note is *beadboard*, used more in the past than at present. This is a colloquial term for EPS-block, specifically after being cut into relatively thin panels for use as thermal insulation. Some consider this term to be somewhat deprecating and thus inappropriate to use in professional practice because of past connotations implying low material quality.

Despite the relatively large number and variety of geofoam materials, as a result of in-ground experience that dates back to at least circa 1960 EPS/MEPS has emerged worldwide as the material of choice in most applications.

Products

Most geofoams, including the dominant EPS, can only be manufactured in a dedicated, fixed plant to predetermined product geometries dictated by the mold that is used. However, molds can be created in an essentially limitless variety of shapes. In addition, both in-plant and field cutting of basic, generic product shapes to accommodate a particular construction situation can be done using a variety of tools. Other geofoams such as PUR or cellular concrete that are foamed in place simply fill the shape of the volume that is to be filled.

With particular regard to the dominant EPS, there are two ways to mold the final EPS product:

- *EPS-block: block molding* of relatively large prismatic blocks
- *EPS-shape: custom shape molding* or simply *shape molding* of an application-specific product (the ubiquitous white foam coffee cup is perhaps the best-known example of an EPS-shape product).

Historically and continuing to the present, EPS-block has dominated the geofoam market, especially in the U.S. and Canada. The block-molding process is discussed in (*2*) with a detailed, up-to-date treatment of U.S. practices presented in (*6*).

Functions and Applications

Lightweight Fill

Although not the oldest geofoam function, lightweight fill using blocks of EPS is perhaps the most intuitive. It is certainly the most widely known and used, with highway and other transportation-related applications by far the most common.

The use of EPS-block geofoam for lightweight fills has now evolved globally to the status of a mature geotechnology. The earliest documented project use was to reconstruct a road on soft ground in Norway in 1972 (*2*). In the last several years, it has reached the status of a generic, routinely-used geotechnology throughout the U.S. and Canada. Consequently, recent research and development efforts have been concentrated in two broad areas:

- Improving the technical understanding and ease of use for practicing geo-professionals.
- Finding new and innovative products and applications.

Significant activities in the U.S. along these lines and of relevance to this symposium have taken place in several distinct ways. First and foremost, National Cooperative Highway Research Program (NCHRP) research has now been performed under the overall umbrella of NCHRP Project 24-11. This work was conducted in two separate studies (with two phases for each study) beginning in the late 1990s and ending in 2011.

The first study, NCHRP 24-11(01), focused on embankments on soft ground and included development of the first and still-only comprehensive, zero-based material-and-construction standard for EPS-block geofoam that was intended for use as the basis of project-specific contract specifications. The final results of this study were made available to the public as both a comprehensive report (*7*) and summary document (*8*). The first major highway project to use the outcomes of this research (actually while it was still being conducted) was the well-known Boston 'Big Dig' (I-90/I-93) that used the new standard in particular as the basis for a project-specific specification for numerous fills that eventually included several analysis and design innovations (*9*,*10*). It is relevant to note that each of the EPS-block geofoam fills on this project was a cost-effective, cost-saving alternative design for what were to be conventional elevated structures supported on deep foundations.

The second NCHRP study, designated 24-11(02), focused on slope stabilization under all ground conditions, reinforcing and emphasizing the fact that EPS-block geofoam when used as lightweight fill is not just a 'soft-ground' technology. Reports covering the outcomes of this research were just released to the public in early 2013. These include a heavily-abridged extended abstract/executive summary (*11*) to encourage its being read by a wider audience of geo-professionals who might not have time to read the complete 600-plus page final report (*12*).

This second study also included a revised version of the material-and-construction standard that was pioneered, as noted above, with 24-11(01). This revised standard has already proven itself in use on the Idaho Transportation Department's (ITD's) first project use of EPS-block geofoam, for the widening and rebuilding a section of U.S. Route 30 (the old 'Oregon Trail') in the vicinity of Topaz, Bannock County (*13*). This project incorporated a number of design innovations and received awards from both ITD (*14*) as well as the American Road & Transportation Builders Association (ARTBA) (*15*).

Contemporaneous with NCHRP Project 24-11, the U.S. Federal Highway Administration (FHWA), National Highway Institute (NHI), and, most recently, the new *GeoTech Tools* website (*16*) that was created as part of the second Strategic Highway Research Program (SHRP2) have embraced and encouraged the broader use of EPS-block geofoam as lightweight fill in road construction (*17*,*18*). Specific initiatives of the FHWA have included:

- highlighting the use of EPS-block geofoam as a specific technology in its efforts to identify and promote technologies conducive to *accelerated construction* (*18*,*19*,*20*)
- collaborating with the Virginia DOT to showcase the use of EPS-block geofoam on the I-95 Woodrow Wilson Bridge (WWB) Project across the Potomac River (*20*,*21*,*22*).

The WWB Project is especially noteworthy as it was another early, major-project use of the outcomes of NCHRP Project 24-11(01).

The exponential increase in the use of EPS-block geofoam for road construction in the U.S. since the 1990s has, unfortunately, been accompanied by an uptick in performance issues that are grouped under the broad heading of 'failures'. This is noteworthy as the 50-plus-year use of block-molded EPS as a geofoam material and product has historically been virtually failure-free (*23*,*24*). Possible causes for this disturbing trend were explored in (*25*) and indicated that the culprit is not an underlying systemic, fatal flaw in the technical aspects of the geotechnology itself. Rather, all known 'failures' can be traced to its implementation in the U.S. through what can best be characterized as a fractured, at times dysfunctional, technology-transfer process due to the generic, commodity nature of block-molded EPS. The very fact that there is generally fierce, cost-based competition for EPS-block geofoam projects (for both supplying the raw material as well as the finished-product supply) means that no one entity or even group of entities in the U.S. is willing to take overall charge and control of technology transfer to geo-professionals. This has been particularly acute in two areas: basic understanding of key technical information concerning the manufacturing of EPS (*6*) and conflicting standards (*17*,*26*).

However, prospective users of block-molded EPS as a geofoam material should not be put off or deterred by this discussion of 'failures'. Rather, they should simply be aware of the fact that EPS-block geofoam is an inherently sound geotechnology that, as with any technology that uses a generic, commodity product, needs to be used with proper knowledge and care. This was the underlying reason for the aforementioned NCHRP Project 24-11: to create detailed, objective, design-related information that could be available to all geo-professionals at no cost.

Despite some of the 'growing pains' associated with the explosive growth in using block-molded EPS in geofoam applications in the U.S., there have been many positive advances in the technology, both in the U.S. and globally. These advances have been in three broad areas:

1. Improved understanding of material behavior and analytical methodologies, both of which have been incorporated into the design and standards documents generated by NCHRP Project 24-11 (*7*,*8*,*11*,*12*).
2. Development of broader applications beyond the well-known and widely used road, airfield, and railway earthworks such as supporting shallow foundations for relatively lightly-loaded buildings and small bridges directly on EPS blocks, and backfills and fills behind earth-retaining structures to significantly reduce both gravity and seismic loads acting on such structures (*2*,*27*,*28*,*29*).
3. Developing new products and associated technologies.

With regard to the third item, in the U.S. the primary efforts have focused on permanent facing treatments for vertical-side fills which have become quite common. This was because research conducted for NCHRP Project 24-11(01) revealed that the cost of facing systems (historically precast, reinforced-PCC panels) was a significant part of the overall construction cost (facing panels are relatively heavy and often require their own deep-foundation support system) yet provide only a decorative/architectural function as such fills are generally inherently stable without an earth-retaining structure. As a result, it is now routine to use alternative facing treatments such as the well-known *Exterior Insulation Finishing System* (EIFS, a.k.a. 'synthetic stucco') that can be made to look like a wide variety of materials. For example, on the Boston Big Dig project EIFS finished to look like PCC was used as a facing for a majority of the EPS-block geofoam fills on this project (Figure 1).

An even more cost-effective facing alternative where its appearance is judged to be acceptable is shotcrete. While the use of shotcrete with EPS-block geofoam fills goes back to the earliest applications in Norway (*2*), the recent award-winning Topaz (US-30) project in Idaho is believed to have been the first in the world where the shotcrete was colored to better blend in and harmonize visually with the surrounding environment. Figure 2 shows the overall finished Topaz project and Figure 3 is a close-up of the barrier-and-drainage system along the edges of the top of the approach-embankment fill. Figure 3 clearly shows the color of the shotcrete (where the facing wraps around the top of the fill) compared to the normal PCC used for both the cast-in-place (CIP) PCC gutter and precast-PCC roadway safety barrier (a 'Jersey' type barrier with slots cast into the base to allow roadway runoff to flow underneath and into the gutter).

Figure 1 - EIFS Facing System Used on Boston 'Big Dig' (I-90/I-93)

Figure 2 - U.S. Route 30 Near Topaz, Bannock County, Idaho (Overall View)

Figure 3 - U.S. Route 30 Near Topaz, Bannock County, Idaho (Top of Embankment)

It is relevant to note that with both the EIFS and shotcrete facing systems all facing materials are suspended from and supported by the EPS blocks. No separate foundation system is required to support the facing as is always required with precast-PCC facing panels or other types of facing that have been used such as segmental-retaining-wall (SRW) blocks.

Outside the U.S., product development has focused on proprietary products to address specific design concerns that sometimes arise with generic EPS blocks. Examples include the development of *anti-buoyancy blocks* by several manufacturers in Japan for applications where high groundwater levels and concomitant buoyancy issues are a controlling design issue. Most of these anti-buoyancy blocks are created using the shape-molded EPS process so are technically EPS-shape geofoam products (Figure 4).

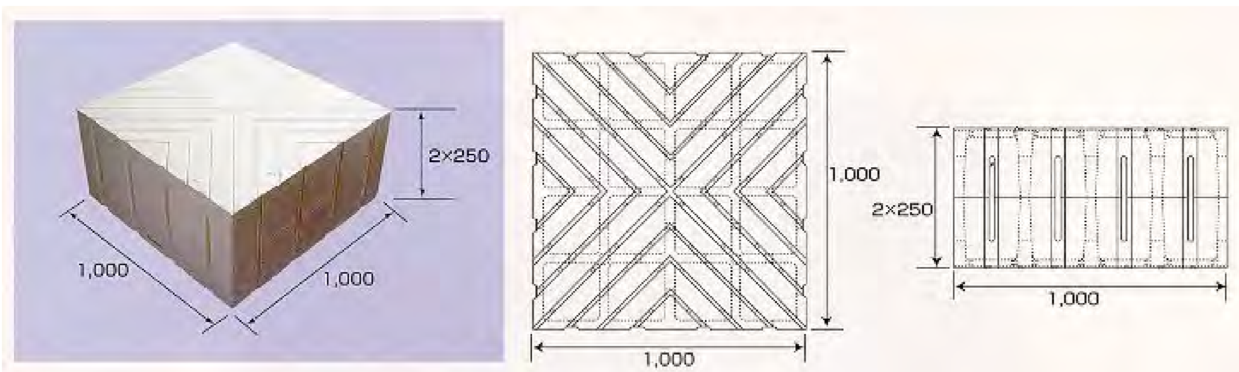

Figure 4 - EPS-Shape Geofoam 'Anti-Buoyancy' Blocks (dimensions in millimetres)

Another new product, developed and produced in Canada but also available in the U.S., is Plasti-Fab's *DuroFloat*™ product. This is an otherwise-generic EPS block but with a permanent coating that is resistant to both liquid petroleum hydrocarbons such as gasoline and UV radiation among other things (*30*). While designed for exposed marine applications, there is no technical reason why these blocks could not be used for terrestrial fills.

Thermal Insulation

This is the first known geofoam function, dating back to at least circa 1960, and one developed initially with road applications in mind (*2*,*5*). A wide variety of geofoam materials, both polymeric and vitreous, have been used successfully for this functional application although EPS and XPS have long predominated in practice. However, the great enigma is that despite over five decades of proven, successful use, thermal insulation of road pavements is still an underutilized geofoam function in many countries, including and especially the U.S. This is all the more surprising given the fact that the use of polymeric foams as thermal insulation in transportation applications was actively researched and patented in the U.S. in the 1960s.

Current activities in the U.S. and Canada related to this function are largely confined to educating geo-professionals about its existence; rediscovering applications long forgotten in practice; and playing catch-up with usage in other geographic areas, notably northern Europe. Some of the potentially significant transportation-related applications are:

- beneath pavements and railway track to prevent or at least limit subgrade freezing and concomitant frost heave (and the pavement problems that eventual thawing creates such as potholes)
- above water-bearing utility lines, especially those beneath paved areas, to allow shallower embedment while preventing freezing of the contents (*frost shielding*)
- beneath the inverts of open culverts to prevent *frost jacking*
- behind earth-retaining structures such as rigid retaining walls, bridge abutments, and soil-nailed walls to prevent freezing of the drainage systems and/or retained soil (*insulated drainage*)
- above the roof slab of shallow buried structures such as cut-and-cover tunnels and parking garages to limit seasonal thermal changes and concomitant thermal expansion and contraction of the roof which can cause structural problems
- with shallow foundations to reduce embedment (*frost-protected shallow foundations*)
- beneath the lining of mined tunnels to prevent freezing of their groundwater drainage systems (*insulated drainage*).

Note that some of these applications are appropriate to all climates. Thus thermal insulation using geofoams is not just a 'cold-climate' geotechnology as many believe.

Drainage (Fluid Transmission)

This is a geofoam function that has been relatively little used to date although it was identified at least as far back as the 1970s (*2*,*5*). The primary reason for its modest use is that if drainage is all that is desired in a given application then there are other types of geosynthetics such as *sheet-drains* and *geonets* that can provide this at a lower cost compared to a geofoam-based product.

However, geofoam-based drainage products (only polymeric foams have proven to be useful in this functional application) have a distinct advantage over these other types of geosynthetic drainage products in that they can be multifunctional depending on the specific material and product used. This multifunctionality, which is inherent in most geofoam materials, has, in general, not been fully appreciated and utilized to date. Thus if designers made use of the fact that a geofoam-based drainage product can simultaneously provide other geosynthetic functions such as the above-described thermal insulation and *compressible inclusion* (defined subsequently), then the use of geofoam-based drainage products could and arguably should increase significantly. On transportation-related projects, such multifunctional applications would be particularly useful with virtually every type of earth-retaining structure.

Noise and Vibration Damping

The vibrations considered here are limited to the small-amplitude motions associated with motor vehicles and trains that cause serviceability issues, typically in the form of human perception and concomitant complaints. Seismic vibrations fall under the lightweight-fill or compressible-inclusion functional categories depending on the particular application.

Noise and vibration damping is another little-researched and -used geofoam function that dates back to at least the 1980s (*2*,*5*). It is certainly a niche application which accounts for some of its modest use. Another reason is that there is no simple, universal analytical methodology that can be used. Rather, each application needs to be evaluated on a project-specific basis that can be analytically demanding. Nevertheless, polymeric geofoams have proven to be useful in applications such as attenuating ground-borne vibrations from motor vehicles and trains as well as wheel-noise from trains.

Compressible Inclusion

This is one of the newer geofoam functions, since circa 1980s (*2*,*5*), but has the potential to be the most widely used of all because of the number of potential applications, especially those involving earth-retaining structures where there is a real potential to revolutionize design and construction. Thus the potential impact on transportation is significant (*31*).

Considerable basic analytical research into compressible-inclusion applications occurred during the 1980s and 1990s (*2*,*5*). During the same timeframe there was also considerable research and development devoted to finding technically efficient, cost-effective materials and products based on both normal block-molded EPS and *resilient* (a.k.a. *elasticized*) block-molded EPS (*2*). An introduction to the subject of compressible inclusions can be found in (*27*,*32*) with more-detailed treatment of the analytical aspects in (*33*,*34*).

The potential applications for geofoam compressible inclusions include:

- Allowing fundamental shear-strength mobilization within soil adjacent to rigid and/or non-yielding earth-retaining structures to reduce lateral earth pressures acting on these structures. This includes the *Reduced Earth Pressure (REP)* concept that reduces pressures to nominally the active earth-pressure state and the *Zero Earth Pressure (ZEP)* concept in which geosynthetic tensile reinforcement (geogrids, geotextiles, metallic elements) acts synergistically with the compressible inclusion to reduce lateral earth pressures to essentially zero. These benefits can be obtained under both gravity and seismic loading.
- Accommodating the volume change of soil or rock that may be inherently expansive or subject to freezing, e.g. adjacent to an earth-retaining structure. The extensive occurrence of expansive (swelling) soils worldwide makes this also a potentially very useful application.
- Accommodating structure movement such as that which occurs with integral-abutment (IAB) and semi-integral-abutment (SIAB) bridges (*34*,*35*,*36*).

An example of a project application for a SIAB in the Commonwealth of Virginia is shown in Figure 5. Note that in this case the inherent multifunctionality of geofoams was used to good advantage as a single geocomposite product, the *GeoTech GeoInclusion®*, was used to act as a compressible inclusion and provide insulated drainage as well. A detailed discussion of the performance of the compressible inclusion on this project can be found in (*37*).

At the present time, relatively simple analytical methods exist for REP-concept applications under gravity loads. Most research in recent years has been focused on REP-concept applications under seismic load, what has been termed *seismic buffers* (*28*,*29*,*38*). Research to develop simplified analytical methods for ZEP-concept applications under both gravity and seismic loading has been initiated (*33*,*34*) and is currently ongoing.

Figure 5 - Geocomposite Consisting of Geofoam Compressible Inclusion and Sheet Drain

Structural/Miscellaneous

This final category is the one most recently identified and is an eclectic collection of applications, mostly using various types of polymeric foams and all of them with potentially significant transportation-related applications (*5*):

- impact cushioning for rock and snow sheds in mountainous regions
- crash barriers for motor vehicles and aircraft
- void filling and foundation remediation using polymeric (PUR)-foam grouts
- lightweight facing panels for mechanically stabilized earth walls (MSEW)
- insulated wall forms for CIP-PCC construction
- void formers for CIP-PCC construction.

Note that in many of these applications other geofoam functions could be utilized if desired. For example, wall forms could be designed to act as a drainage layer and compressible inclusion in addition to providing post-construction thermal insulation.

GEOCOMBS

Definition

A *geocomb* is defined as an open-cell polymeric material with a honeycomb-like cross-section that is created in an extrusion process performed in a fixed plant. It is essentially an assemblage of contiguous open-end tubes, the color of which depends on the particular polymer used (Figure 6). Each tube is of the order of 1 inch (25 mm) across and the material is approximately 96% voids overall (a porosity of 96%).

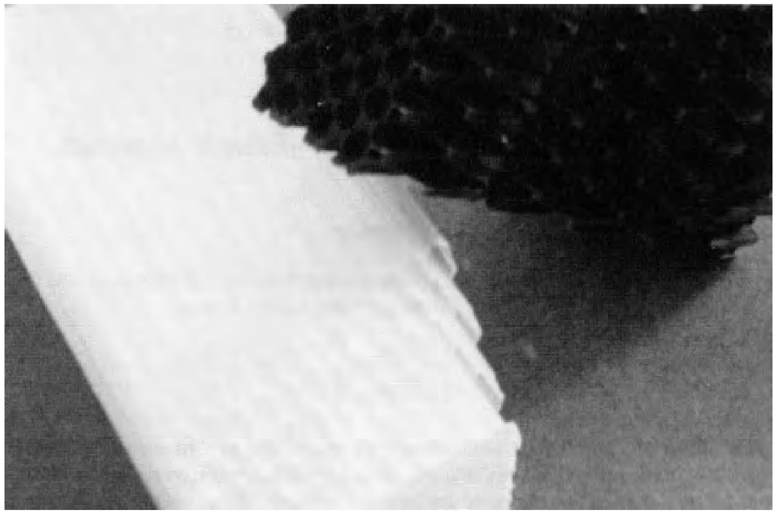

Figure 6 - Sections of Geocomb Blocks

Geocombs are one of the newest geosynthetic product categories to be identified. Although the term was coined only in 1999, they have been used in France and its territorial affiliates since the 1980s (*39*) where they are known as *structures alveolaires ultra légères* (SAUL; in English *ultra light cellular structures*, ULCS). However, they are still not readily available outside of France although this changing.

Materials

Two different polymers are known to have been used for geocombs to date: a translucent polypropylene (PP) that appears white in photographs and black polyvinylchloride (PVC). Examples of both are shown in Figure 6. The PP product line appears to be predominant in terms of past and current usage (*39*).

Products

The extruded-honeycomb material is typically factory-cut into panel- or block-shaped pieces that are the basic final product. The geocomb blocks have dimensions that are close to those of smaller EPS blocks that are currently used for geofoam lightweight-fill applications, approximately 2 x 4 x 8 feet (600 x 1200 x 4800 mm). In many cases, the geocomb product has a non-woven (typically heat-bonded) geotextile factory-bonded to one or both open ends of the tubes to prevent soil particles from entering the tubes once the blocks are in place. The panels or blocks are placed with the tubes oriented vertically as the overall product is significantly stiffer when loaded parallel to the tube axes as opposed to perpendicular to them.

Functions and Applications

Lightweight Fill

This appears to be the predominant geocomb function used to date and the one most widely documented in the literature (*3*,*39*). Geocomb blocks are broadly comparable to EPS-block geofoam in terms of load-carrying capability when used as lightweight fill for roads (Figure 7). Although a geocomb block cost more than an EPS block on a unit-volume basis and has an overall density approximately twice that of block-molded EPS, geocomb has the distinct advantage of having virtually no buoyancy upon submergence as its open-cell structure allows groundwater to fill the void spaces as well as readily drain if and when the groundwater subsides. This can be a crucial advantage that makes geocomb the lightweight fill of choice in applications where permanent or potential submergence is an important, controlling design consideration.

Figure 7 - Geocomb Blocks Used as Lightweight Fill on a Bridge Project in France

Drainage (Fluid Storage and Transmission)

A geosynthetic function of geocomb that appears to be of growing interest is to make use of its inherent open-cell structure and porosity in applications where fluid handling, primarily of water, is the primary function. Not only does geocomb readily transmit groundwater but it can also be used to store water for some indefinite period of time. The primary application for this appears to be on transportation-related projects where temporary subterranean detention of surface water followed by natural release to the groundwater system is a benefit. For example, what amounts to a subterranean reservoir with a very efficient 96% voids per unit volume can be constructed without limit beneath parking lots and similar paved areas.

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Corridor Management: Capturing Geotechnical Impacts on Highway System Performance

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ABSTRACT

Risk-based transportation asset management plans are required under new performance-driven legislation. Bridges and pavements are required within these plans, and the inclusion of other assets is encouraged. One could argue that the primary assets of a transportation agency are the transportation corridors that have been established to provide means for moving people and goods safely and efficiently. A corridor's performance in this regard is only as good as its weakest link. Therefore, the way an agency can manage an asset, such as a corridor, to a standard for system performance, is to consider its components concurrently, not by individual asset classes. A corridor has embankments, slopes, walls, bridges, and pavements, and considering these geotechnical features separately does not make sense from a system performance perspective. Settlement, slope instability, rockfall, erosion and corrosion are events which can be surprising, or recognized in advance and managed. The corridor concept can bring geotechnical assets into consideration and result in better management for system performance. It also provides a means for rational prioritization that allows for a phased approach to the daunting task of collecting inventory and condition assessment for features that have not previously been managed. Geo-professionals are developing tools and practices for inventorying, assessing performance, predicting life-cycle costs and degradation, and evaluating risk associated with geotechnical features. These tools and practices will contribute to effective corridor management.

INTRODUCTION

Transportation asset management (TAM) and highway system performance within the US are evolving with the underlying intent of making informed, rational resource allocation decisions in order to sustain and improve the performance of the national highway system. In July, 2012, the new highway bill entitled, ‘‘Moving Ahead for Progress in the 21st Century Act,’’ or MAP-21, was enacted into law. This new legislation reestablished national goals for the federal highway program and established requirements for performance management and associated performance measures *(1)*. Under MAP-21, State Departments of Transportation are required to “develop a risk-based asset management plan for the National Highway System to improve or preserve the condition of the assets and the performance of the system,” and will need to set targets and track progress toward achieving those targets based on the established national goals and performance measures. States are required to include performance management of bridges and pavements within their asset management plans, and are encouraged “to include all infrastructure assets within the right-of-way corridor in such plan” *(1)*.

State highway agencies are in the early stages of developing their Transportation Asset Management Plans (TAMP), and will be for several years. Asset management systems for bridges and pavements have evolved separately under different motivations and are, at this point, more advanced with respect to condition indicators, performance prediction modeling and life-cycle cost analysis. As a result, there seems to be a tendency to manage these and other asset classes separately. In addition, bridges and pavements represent significant safety and cost implications, and are both highly visible to the public, explaining their more rapid advancement. However, geotechnical features also significantly impact highway system performance, and they and their risks should also be recognized and managed as well. Since the ultimate objective is to manage highway system performance, integrating an engineering-systems approach to manage corridors – arguably the true asset of a transportation system – within these TAMPs would provide a rational means to evaluate the condition and performance of all assets and features, and associated risks within corridors concurrently. Described herein are examples illustrating the impacts of geotechnical features to system performance, an explanation of how this “corridor concept” could be applied within the decision making process, and challenges facing the geo-industry as geotechnical asset management progresses within the TAMP framework.

GEOTECHNICAL FEATURE PERFORMANCE

A geotechnical feature is defined here as a part of a highway right-of-way comprised largely of soil or rock, or another improvement that has direct bearing on soil or rock performance or influence over the effects of their performance. Examples are cut slopes, embankments, retaining walls and the soil or bedrock foundation upon which all structures and roadway are built. Improvements are things such as surface or subsurface drainage ditches, pipes and trenches, rock bolts or ground anchors, and rockfall mitigation systems. When a geotechnical feature is performing well it goes unnoticed but when it is not performing well it causes escalation in maintenance costs or catastrophic failure. Either way, it causes a drain on limited resources, a potential safety hazard, and a reduction in performance. A few recent slope and embankment examples illustrate this point well *(2)*.

Embankment Failure on I-75 in Campbell County, TN – March, 2012

In March, 2012, a slope failure within a 150-ft high side-hill embankment section propagated into the southbound travel lanes of I-75 in Campbell County, TN, forcing southbound traffic to be rerouted for five days until one southbound travel lane could be reestablished along the northbound side. The investigation of the failure revealed a deteriorated corrugated metal pipe (CMP) culvert and saturated weathered-shale clay embankment material and underlying natural soils to be primary factors contributing to the failure *(3)*. An emergency repair contract was executed in mid-April, 2012 with an estimated repair cost between $9.4M and $12.6M and an estimated completion date of September 28th (5.5 months).

Figure 1 – Aerial photograph of the I-75 embankment slope failure site in TN as construction repairs were on-going in May, 2012. (Photograph courtesy of Tennessee Department of Transportation).

A detour was created within the right of way and one southbound lane remained closed during most of the repair. In addition, two alternative routes approximately 20 and 30 miles in detour length were recommended to travelers. Even so, with an annual average daily traffic (ADT) volume of approximately 28,000 vehicles, long delays and traffic back-ups in excess of 20 miles along the interstate were generally expected during holidays and peak travel times *(4)*. An aerial photograph of the embankment failure site taken by Tennessee Department of Transportation (TDOT) in May, 2012, shortly after a localized failure occurred near the upper southern end of the site during construction, is shown in Figure 1. Traffic was reduced to one-lane northbound and closed to southbound traffic for 14 days until the localized failure was stabilized.

Rockslide on I-40 in Haywood County, NC – October, 2009

In October, 2009 a large rockslide occurred near mile post 3 along the I-40 Pigeon River Gorge corridor in North Carolina essentially closing a 53-mile section of the interstate between North Carolina and Tennessee for 6 months while debris could be removed and the rock-slope was stabilized. An aerial view of the rockslide soon after failure occurred is shown in Figure 2.

Figure 2 – Rockslide within the Pigeon River Gorge on I-40 in North Carolina, October, 2009 (Photograph courtesy of North Carolina Department of Transportation).

A second failure occurred in January 2010 near mile post 7 while the interstate was still closed. The interstate reopened in April, 2010 to limited traffic for an additional 6 months until repairs for the two failures and mitigation measures for five other high-risk sites were completed The total cost for all slope repairs, mitigation measures and operations was $19.2M *(5; Joel Setzer, personal communication, July 9, 2012).*

During the I-40 closure, an ADT of approximately 24,000 vehicles per day was rerouted approximately 130 miles along I-81 and I-26. Frequent traffic back-ups in excess of 7 miles were commonly observed in Asheville, NC due to increased congestion. Coincidently, US Highway 64 near the NC/TN border was also closed from November 2010 to April 2011 due to rock-slope failures. The total repair cost for the US-64 slides was approximately $3M. Local economic impacts due to the closures of two regional national highway corridors are difficult to quantify. However, a report examining the economic impacts of these two coincidental rockslides was prepared for the Appalachian Regional Commission. The report suggests that rural businesses within the surrounding area experienced reductions in revenue ranging between 30 to 90 percent compared to previous years. The regional economic and transportation costs were estimated to be $197M due to increased congestion, additional travel times, vehicle operating costs and pavement maintenance. Approximately 90 percent of those costs were attributed to the I-40 closure *(6)*.

The I-40 Pigeon River Gorge corridor has a history of rockfall and rockslide activity since its original construction in the late 1960s. Significant failures causing closures and partial closures for five or more months occurred in 1985 and in 1997. Most recently, three failures and two partial closures of two weeks or less occurred in January and March, 2012. Closures of this frequency and duration almost certainly mean the performance of this corridor and perhaps network of corridors is controlled by geotechnical feature performance.

Rockfall on I-70 in Glenwood Canyon, CO – March, 2010

In March, 2010, a rockfall event occurred on I-70 within Glenwood Canyon, Colorado. The rockfall covered all travel-lanes in both directions, damaging the bridge-deck that elevates the roadway through this section. One boulder completely ripped through the deck, damaging a support beam and retaining wall below the deck. The aftermath of the rockfall is shown in Figure 3. The corridor was closed for four days and re-opened to limited traffic for two-months while debris was removed and repairs were made. The repair costs for this event totaled $1.6M. A significant impact of this closure involved the necessary 200-mile detour to connect local communities and to reroute an ADT volume of approximately 27000 vehicles per day.

This corridor has seen two other recent events with similar closures. Another rockfall event occurred in November, 2004, and in June, 2003, a culvert failure occurred that completely destroyed an embankment section of roadway. The repair cost for the culvert failure was $4.2M. Again, the frequency suggests that geotechnical features likely control the performance of the corridor.

Figure 3 – (A) Rockfall debris on I-70 elevated deck. (B) Portion of deck torn-through by boulder (Photographs courtesy of Colorado Department of Transportation).

In summary, these recent examples from three different interstate highways show the significant impact to the performance of a highway system caused by closing a highway corridor. The large impact from these examples is not in safety, as one might first think, as there were no serious injuries or fatalities from these events. The impact is in direct repair costs, which are higher for emergency situations than for programmed work, and indirect costs associated with closure and reduced mobility for the community. These are impacts to system performance from geotechnical features that can be managed.

PERFORMANCE MANAGEMENT IN THE FUTURE

Performance Management is a systematic approach to making investment and strategic decisions using information about the condition and performance of the system and developing an approach to achieve goals. The MAP-21 goals are as follows *(1)*:

- Safety – reduce fatalities and injuries;
- Infrastructure Condition – maintain the highway infrastructure asset system in a state of good repair;
- Congestion Reduction – reduce congestion on NHS;
- System Reliability – improve efficiency;

- Freight Movement and Economic Vitality – improve the freight network, strengthen ability of rural communities to access national and international trademarks, support regional economic development;
- Environmental Sustainability – enhance performance of transportation system while protecting and enhancing the natural environment; and
- Reduced Project Delivery Delays – reduce project costs, promote jobs and economy, and expedite the movement of people and goods by accelerating project completion through elimination of delays in project development/delivery process.

All of these can in some way be impacted by geotechnical features and for some of these goals the connection is significant. The applications of asset management systems and principles are intended to assist highway agencies with achieving these performance goals. If established indicators, measures, and state-developed asset management plans do not collectively consider the impacts of all manageable features having significant influence on the effective performance of the highway system and its corridors, there is a risk that established standards and targets might well be met while the impacts due to other significant features and their associated costs may be left ineffectively managed resulting in inadequate performance to the system and its components. From the geotechnical perspective, this risk can be mitigated by educating those establishing and evaluating the measures and putting geotechnical work in a framework and language that can be understood and incorporated. Geotechnical engineers have a significant responsibility for doing this and a real opportunity to get others to take a fresh look at the impact geotechnical features have on transportation system performance.

APPLYING ASSET MANAGEMENT PRINCIPLES

Management for our national highway system is evolving. Early attention has been given to Transportation Asset Management (TAM) strategies focused largely on certain elemental components of highway systems; bridges and pavements are primary examples, and for good reasons. Bridge collapse as a result of scour, corrosion or other degradation is a significant safety risk. The National Bridge Inventory and bridge management systems mitigate this risk. Pavements are pervasive and in total they represent a large percent of highway dollars. Pavements also have a life cycle and a condition degradation curve that is relatively easy to see and understand. Both bridge and pavement management systems are well developed and mature as compared to systems for geotechnical features.

To demonstrate alternate ways a state may manage their highway assets, hypothetical data from an inventory of six highways are shown in Figure 4. A state would have a much larger list of highways and different numbers (whatever the unit of measure) for each asset class. The important thing is that the table captures all of the assets of each class for the highways in the system. In practice, the data have been available to fill out the columns on pavement and bridges, both in terms of inventory (Figure 4a) and some type of condition rating (Figure 4b) for many years. In contrast, many states are just now getting inventory information and considering ways to rate condition for the asset classes in the other columns. As states look to managing geotechnical assets (which are in the other columns) they are faced with the following substantial and sequential implementation steps:

Step 1. Collect inventory information on ”new” asset classes,
Step 2. Assess the existing condition and rate “new” assets, and
Step 3. Model or predict change in condition through time of “new” assets and relate to a Level of Service (LOS).

The first two implementation steps (inventory and rating) are about as much as any state has accomplished with geotechnical assets. These can be huge tasks because of the vast inventory and the fact that many are starting from no records whatsoever. In fact, this has been such a large undertaking that some states (e.g. Colorado and Washington) recently cut back on plans to inventory and assess retaining walls because of the cost of implementation and uncertainty in the payoff from the investment. No state has completed inventory and rating for all of the asset classes shown in Figure 4. The size of the investment in time and resources to do so has been an obstacle to more rapid progress in geotechnical asset management. Emphasizing the concept of managing a transportation corridor as the asset has the potential to overcome this obstacle.

		Asset Class						
		Pavement	Bridge	Walls	Culverts	Slopes	Embankments	Drainage
Highways	1	100	30	80	100	50	60	10
	2	200	50	40	150	30	40	20
	3	300	70	20	50	10	20	40
	4	200	40	20	100	60	50	20
	5	400	60	30	100	40	30	10
	6	600	80	70	150	20	20	20
SUM (units)		1800	330	260	650	210	220	120

(a) Inventory, with total for each Asset Class shown.

		Asset Class						
		Pavement	Bridge	Walls	Culverts	Slopes	Embankments	Drainage
Highways	1	6	7	6	6	8	6	6
	2	7	9	7	9	6	9	5
	3	5	7	8	6	7	9	8
	4	9	6	8	8	9	7	7
	5	8	7	7	8	8	8	8
	6	6	5	9	7	8	8	9
AVG.		6.8	6.8	7.5	7.3	7.7	7.8	7.2

(b) Condition (performance) rating or level of service (LOS), with average shown for each Asset Class.

Figure 4 – (a) A hypothetical inventory organized by asset class and, (b) hypothetical values of assessed condition or performance, also organized by asset class.

A corridor is a defined section of transportation pathway (Right-of-Way) that traverses and crosses natural and manmade obstacles and provides for economic vitality by allowing for the safe and efficient movement of people and goods. The asset types are like pieces of equipment along the way that are needed to accomplish this mission. There may be good reasons to define a corridor as a limited part of a highway or a sequence of highways linked together. Wyoming has addressed this and has a good example of a statewide corridor system *(7)*. At a national level, a logical collection of corridors is the Interstate Highway System (IHS) and there are some recent advances and publications on the management of this system.

In 2009, NCHRP published *Report 632: An Asset-Management Framework for the Interstate Highway System* that proposed a framework for an integrated, performance-based and system-wide approach, recognizing the critical significance of the interstate highway system to global, national, regional, and local movements of people and goods. The report identified the need for comprehensive management strategies, and focused on 1) how to incorporate assessment of the risks of system failure into the asset management framework; 2) guidance for handling all interstate highway system assets, particularly those other than bridges and pavements; and 3) recommended sets of measures and approach to performance management for highway assets *(8)*. Importantly, the report identifies retaining walls, tunnels and drainage structures as asset types and it does call for consideration of natural hazards as a source of risk, but the report is silent with respect to slopes and embankments, for example. Natural hazards are limited here too. For example, there is no mention of swelling or collapsing soils, sink holes or other geohazards that wouldn't likely impact an existing bridge but could have a large impact on a corridor. The framework does, however, incorporate a risk assessment approach within the asset management plan development process, whereby risks to system failure affecting safety, property damage and system/mission disruption from identified threats, including natural hazards and deficit conditions of assets, can be managed.

A continuation of the NCHRP Report 632 effort resulted in *Report 677: Development of Levels of Service for the Interstate Highway System*. This report presents a standard template and it recommends asset classes and elements to communicate critical funding needs to decision-makers, direct resources to problem areas, and demonstrate accountability to taxpayers *(9)*. However, noticeably absent from this report are indicators and measures of any real significance relating the condition of geotechnical features to system performance for decision-makers.

Most recently, the FHWA Office of Asset Management has been developing guidance with much focus toward development of Transportation Asset Management Plans. Among the developments are a series of reports on Risk-Based Transportation Asset Management. *Report 4: Managing Risks to Critical Assets* and *Report 5: Managing External Threats through Risk Based Management* are particularly interrelated to management of corridors and geo-hazards, yet do not specifically merge the two areas *(10, 11)*.

The conclusions here are that there is movement in TAM to include recognition of geotechnical features but there is still a ways to go. The movement towards performance of corridors and systems promises to help integrate geotechnical features into TAM, especially as TAM is applied to performance. Looking back at Figure 4b, the hypothetical entries represent condition for different asset types, often called asset classes – as in the figure, and the tendency is to look at the asset classes individually, as in stovepipes separated from one another. Because many geotechnical assets are not yet inventoried or assessed, and the term itself is not defined or established (for example, it is not mentioned in the recent NCHRP reports), now is the opportunity to consider what benefit there is of creating or defining geotechnical features as asset classes that might become additional stovepipes. While assessing a population of individual asset classes certainly could have value for tracking preservation efforts for those classes, the approach is limited when attempting to provide any significant collective indication of system performance.

The corridor concept, which is shown conceptually as rows in Figure 5, provides a rational approach to using TAM for optimizing system performance. The stovepipes (columns), as in Figure 4, are deemphasized and identified as features of a corridor. The corridor (row) is the meaningful asset. This allows states to phase in the three implementation steps, focusing on high priority corridors first. In this approach, corridors are identified based on meaningful characteristics and they are prioritized based on their significance to the performance of the system.

		Feature Type							
		Pavement	Bridge	Walls	Culverts	Slopes	Embankments	Drainage	MIN.
Corridors (by performance priority)	1	6	5	4	8	6	2	7	4.0
	2	5	8	3	6	3	7	2	2.0
	3	7	8	6	7	5	5	7	6.0
	4	7	3	3	5	7	3	6	3.0
	…	…	…	…	…	…	…	…	…
	99	4	6	7	4	2	6	4	2.0
Inventory evaluated as a list of corridors prioritized for system performance									

Figure 5 – Level of Service (LOS) for a hypothetical inventory of corridors listed in priority order and evaluated for the Feature Type with minimum LOS.

One could imagine that corridors on the IHS would be near the top of this list and systems of corridors could be grouped in priority categories. For each priority category, targets and tolerances would be established for the LOS of each asset class (feature type). The LOS rating scale for all asset classes can be made similar such that it would be possible to compare the LOS between different assets. Given that the performance of a corridor is only as effective as its weakest link, risks to safety, property damage and system/mission disruption could be assessed/reassessed for assets with LOS's falling below established tolerances, and for other identified threat sources, such as geohazards. Similarly, for preservation purposes, decisions can be based on LOS and corridor priority. So, rather than emphasis being placed on the LOS of an asset class, asset classes are evaluated "horizontally" and screened within corridors, and according to corridor priority for low LOS, as shown hypothetically in Figure 5. The prioritized corridor approach also allows for meaningful information to come from an incomplete survey of geotechnical assets, thus allowing states to proceed gradually into the inventory and assessment of geotechnical assets.

As can be seen, the Asset Class term in Figure 4 has been replaced with Feature Type in Figure 5 to introduce terminology that emphasizes the corridor as the asset. The shading indicates that the "horizontal" approach includes many types of geotechnical features, even in this simple hypothetical example. Since the data in Figure 5 for LOS are static, they are a snapshot in time, and the influence of time needs to be considered as well. A third recent NCHRP report, *Report 713: Estimating the Life Expectancy of Highway Assets*, provides a guidebook approach on how this can be done *(12)*. In concept, every asset has a deterioration curve that represents how condition (or performance) changes through time and how it can be impacted by actions during its life. A classic example is for pavement and is shown in Figure 6, where PCI is the Pavement Condition Index.

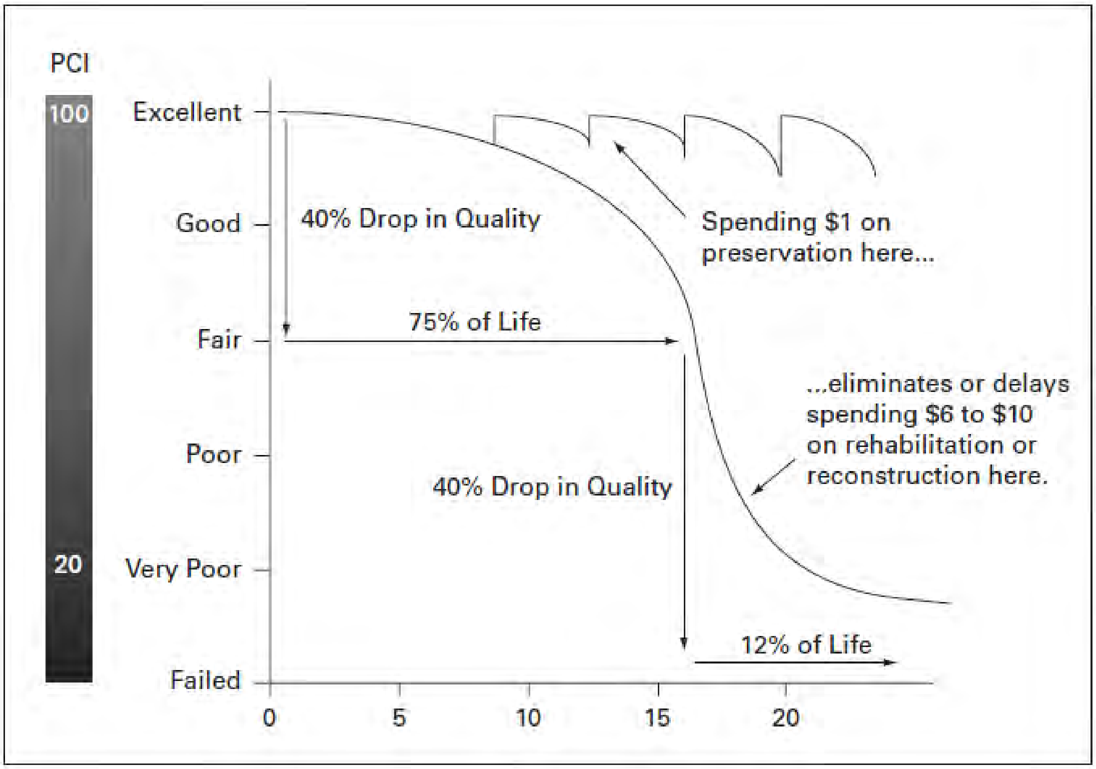

Figure 6 – Example deterioration curve for pavement showing the value of investment in preservation at a certain age *(13)*.

GEOTECHNICAL CHALLENGES

Geotechnical challenges relate to the steps of asset management discussed earlier. Tools and protocols are what comprise asset management systems and some states and other agencies have good systems in place; for example, Alaska *(14)* and Washington *(15)* have slope management systems and the National Park Service *(16)* and Nebraska *(17)* and the city of Cincinnati have retaining wall management systems in place. To date, most agency work has been on inventorying and condition assessment because these are the first steps. Continued work in refining these systems and improving the efficiency of their implementation is an important area of research and development for the geo-professional as this serves to address the first and second challenges mentioned above.

The second challenge (and opportunity for research and development) has to do with establishing performance expectations for geotechnical features and identifying and implementing methods of measuring and testing performance with respect to these standards. For example, the expectation of the frequency of rockfall from a rock cut, or the long term settlement of a bridge approach, or movement of an anchored wall, or corrosion of steel reinforcements in MSE are all things that the profession hasn't established or hasn't developed means for measuring or recording in consistent ways. There is some important activity in these areas, for example the ongoing NCHRP Project 24-35 titled *Guidelines for Certification and Management of Flexible Rockfall Protection Systems* addresses performance life-cycle expectations for these mitigation measures. The Long-term Bridge Performance program administered by the Federal Highway Administration includes performance expectations and monitoring of approach embankments. The Alaska DOT&PF has been instrumental in advancing the discussion on performance measures and LOS for slopes *(18, 19)*.

The third challenge mentioned previously – the one that has had the least attention – is the need for predicting how performance changes through time and identification of the most advantageous times for investment for long-term optimization of the level of service. Stanley and Pierson *(20, 21)* and Vessely *(22)* have made some predictions on how these curves might generally look for some geotechnical features, but there is a unique challenge for many geotechnical features in that their performance curves may be more like step functions where natural but rare events have a dominant impact on performance. This is primarily where risk enters in the process for geotechnical features. NCHRP Report 713 has excellent coverage of many types of curves that may be applicable to geotechnical features *(12)*. NCHRP Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforced Systems looks at the reliability of corrosion rate models and metal loss for metal reinforced systems, and provides the ground work for addressing long-term performance expectations for mechanically stabilized geotechnical features and mitigation systems, including walls, anchors and bolts *(23)*.

Bear in mind that tools and protocols for assessment of all feature types are needed but it is not necessary to identify the entire inventory or assess the entire inventory, or do it for all corridors with equal frequency. The prioritization for optimizing system performance relieves us of that burden and should be helpful to states looking at insurmountable challenges to inventory and assess all of their geotechnical and other ancillary assets.

CONCLUSIONS

The slope and embankment failure examples described in the Geotechnical Feature Performance section of this paper demonstrate how significant the impact of geotechnical features can be on the performance of a transportation system. Starting in 2013, state transportation agencies will be required to use asset management approaches to demonstrate that the investment of federal dollars into their systems leads to improved system performance. Most asset management is currently focused on asset classes and reporting on their individual performance and level of service. A corridor approach is described whereby the significant impact of geotechnical features on system performance could be captured and the adoption of asset management principles for geotechnical features could be introduced gradually, in a meaningful and more affordable way.

There are challenges to managing geotechnical features as part of a corridor asset and solutions to these challenges are needed by the transportation industry. These challenges are good opportunities for research and development now. There has already been a trend for agencies to use TAM approaches to manage their infrastructure, but it has just now been written into law. A melding of existing geotechnical solutions to (a) inventory and condition rating, (b) risk assessment, and (c) performance monitoring with the practice of TAM and performance management is the future. Someday it will be possible, for example, to identify the deterioration of the embankment on I-75 in Tennessee and take timely steps to improve drainage, and thereby the LOS, without such a large negative impact to performance. Or, for example, it may be possible to demonstrate the value, from a performance perspective, of multi-million dollar solutions to mitigate the hazard from the rock slopes above I-40 in North Carolina or I-70 in Colorado.

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Four Times the Effort: Big Blue River Bridge Project

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ABSTRACT

Projects for the Kansas Department of Transportation, Geotech Section are usually pretty straight forward. The geology section has a great working relationship with our design squads. Design changes are usually minor, such as small alignment corrections, right-of-way needs, or minor adjustments to a bridge span. However, one particular project was not that simple, the Big Blue River Bridge replacement and realignment of US-77 highway. The geologic setting is the Flint Hills Region of Kansas with approximately 200 feet of topographical relief and an extensive gypsum mining operation.

This project went through 4 alignment changes. Some of these changes moved the roadway as much as ¾ of a mile, others only a couple hundred feet. The Geotech Section was given 3 months to complete the investigation. After completion of the field work the alignment changed to eliminate an 80 foot rock cut slope. Other alignment shifts were put into place, always after the field work had been started. The final alignment shift was begun by a local landowner. He had a better plan than our design squad.

What started out as a simple project now had consumed 1 year of field time, involved numerous design revisions, had major utility impacts and resulted in alterations to two Kansas highway alignments.

INTRODUCTION

The Big Blue River Bridge Replacement Project began like most of our Kansas DOT projects. The project was to replace the existing structure and improve the horizontal alignment for US-77 and the intersection with the K-9 highway. This Fiscal Year 2011 project is located just to the east of the town of Blue Rapids in north central Kansas. It started with the Preliminary Engineering Study, but the project ended quite differently than originally planned.

PLANNING

The existing alignment for US-77 and K-9 highways was constructed in 1949, with the completion of the river bridge in 1950. The existing roadway is deficient in design speed, cross slope and superelevation. Reduction of Speed and Curve signage is required. The existing Big Blue River structure is a four-span, 727 foot long, 26 foot wide, steel truss (Photo 1). The bottom of the deck was severely map cracked with cracked welds. The bridge is a fracture critical structure and is functionally obsolete based on the National Bridge Inspection Standards. The structure does not allow for deck repairs without a closure of the roadway. At the start of construction in 2011, the deck on the existing bridge was patched with 13 metal plates. Closing the roadway was determined to be too costly of an option, the preferred option was to realign the roadway and reconstruct the bridge.

The Preliminary Engineering Study (Figure 1) was completed in January of 2001. The study addressed the horizontal alignment issues, and 14 different alignments were investigated. One of proposed alignments was a complete corridor realignment that bypassed the city of Blue Rapids to the North (Figure 2). All of the alignments would have two major concerns at this stage: Geotechnical and Archeological. Ten of the alignments shifted the highway to the east and four moved it to the west. The Geotechnical concerns were backslope designs (Figure 3), rock excavation and underground gypsum mines (Figure 4). Archeologically there were issues with Kansa Tribe encampments and historical markers.

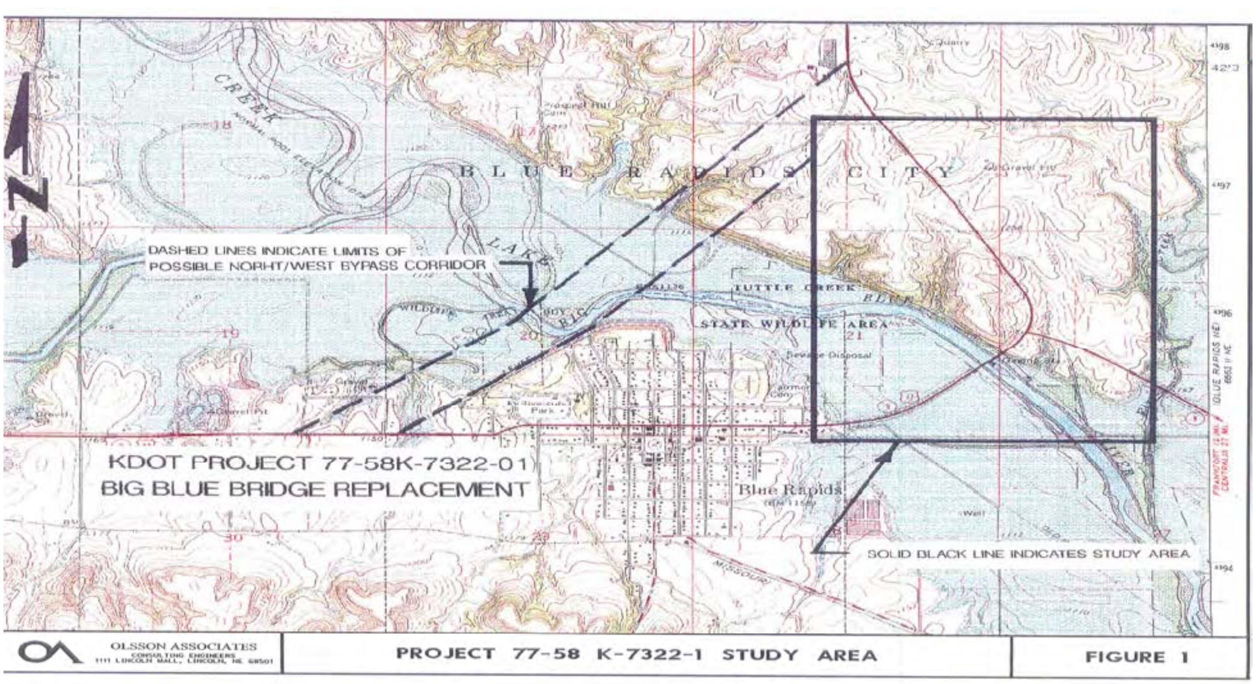

Figure 1 – Map outlining Preliminary Engineering Study

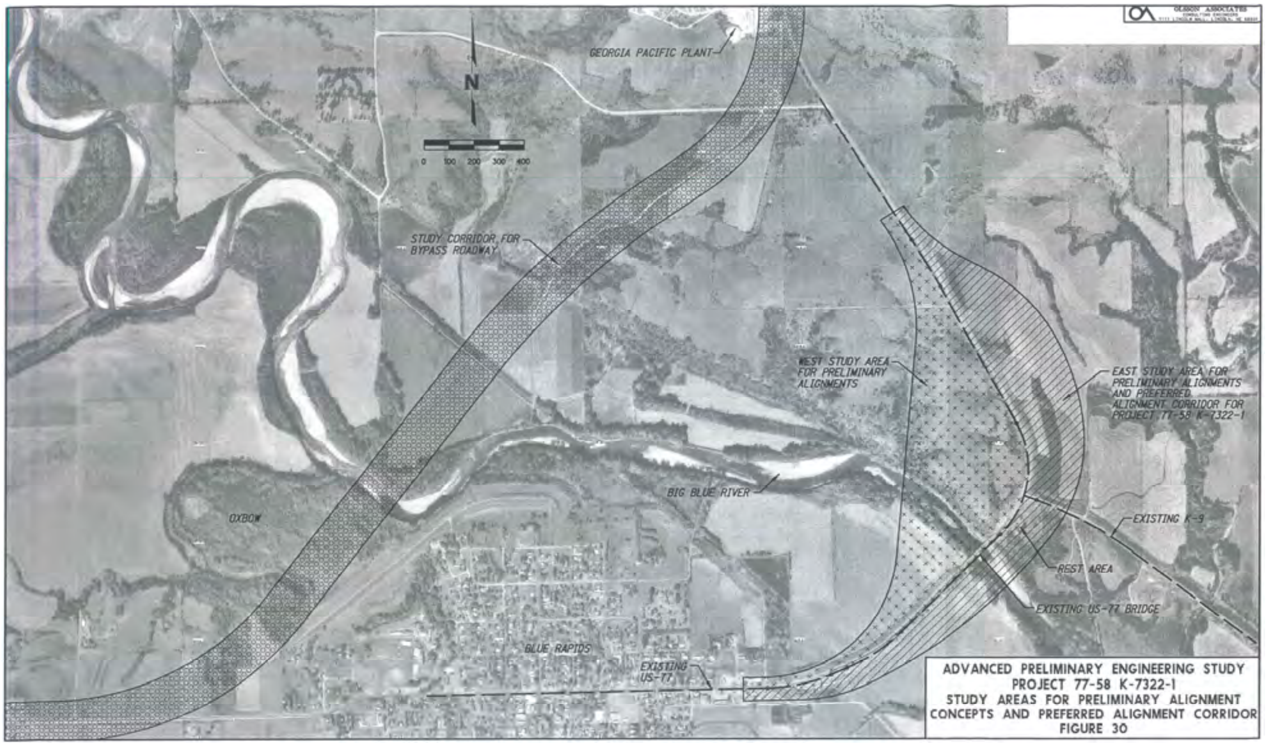

Figure 2 – Corridor North of Blue Rapids

Photo #1 Existing River Bridge and the New Structure to the South

GEOTECHNICAL ISSUES

The Geologic Setting for this project is within the Flint Hills Region of Kansas. The Flint Hills are Permian aged sediments deposited in a warm shallow land locked sea with fluctuating water depths. The hills are capped by resistant beds of chert bearing limestone followed by alternating layers of soft shale, limestone and the occasional evaporate bed. The limestones are typically very well suited for building stone as they are easily cut, shaped and polished. Gypsum is the most common evaporate mineral found in this region. Mining for gypsum began in 1857. During the most active period there were 8 mines in operation but that number was soon pared back to 4. Most of the mining was done by the shallow open pit method with the gypsum being cooked over open flames in 50 gallon vats. Four mines were underground room and pillar operations. One of these underground mines was near the confluence of the Little and Big Blue Rivers. This mine was operational until the 1920's. It soon became a grand location for the local bootleggers to store and sell their wares. This mine was blown in when it was deemed hazardous to the local children. The largest underground operation is still active today and is owned by the Georgia-Pacific Corporation.

The underground mines posed a potential problem for several of the alignments. The study located only 2 small areas that had undergone subsidence events (Figure 5). However, the study only looked at the ground owned and mined by Georgia-Pacific. As the Geology Section reviewed the first proposed alignment in the field over 20 small collapse features were recognized.

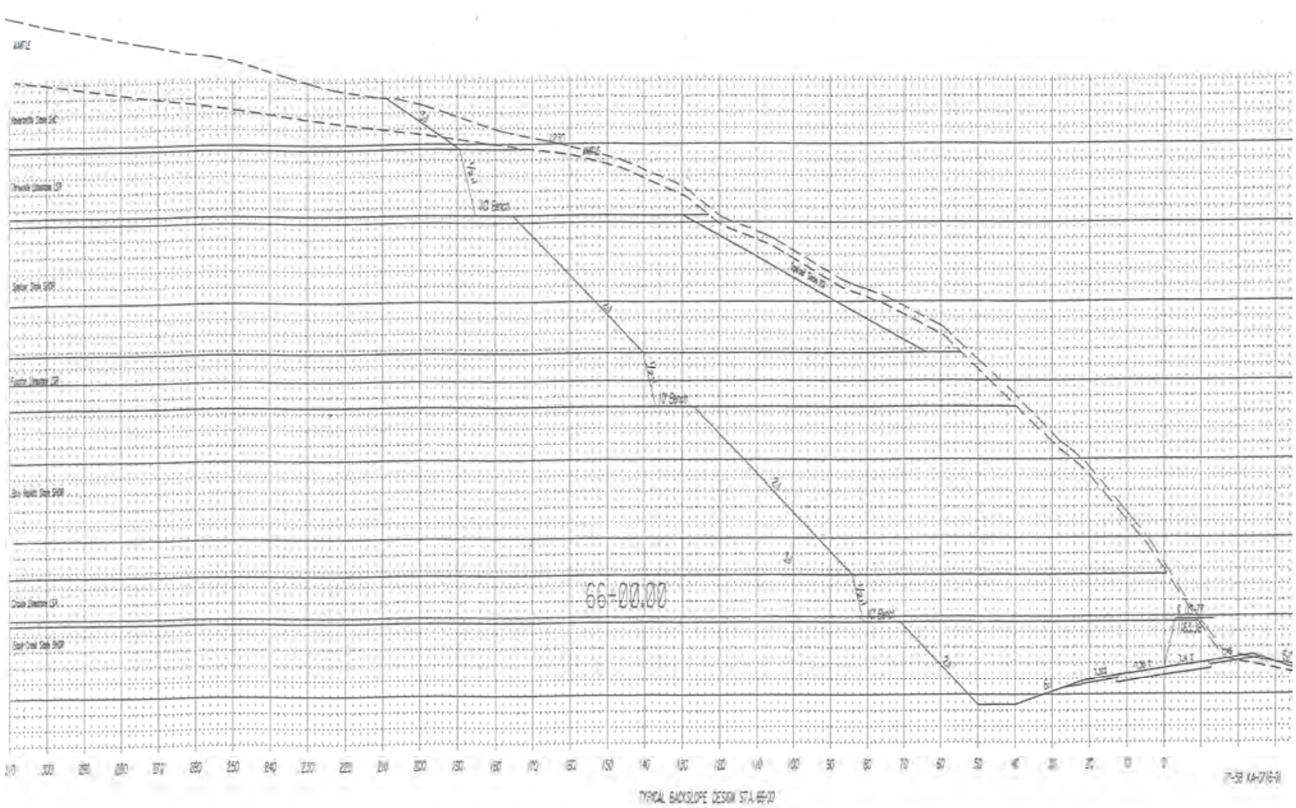

Figure 3 – Rock Excavation Backslope Typical

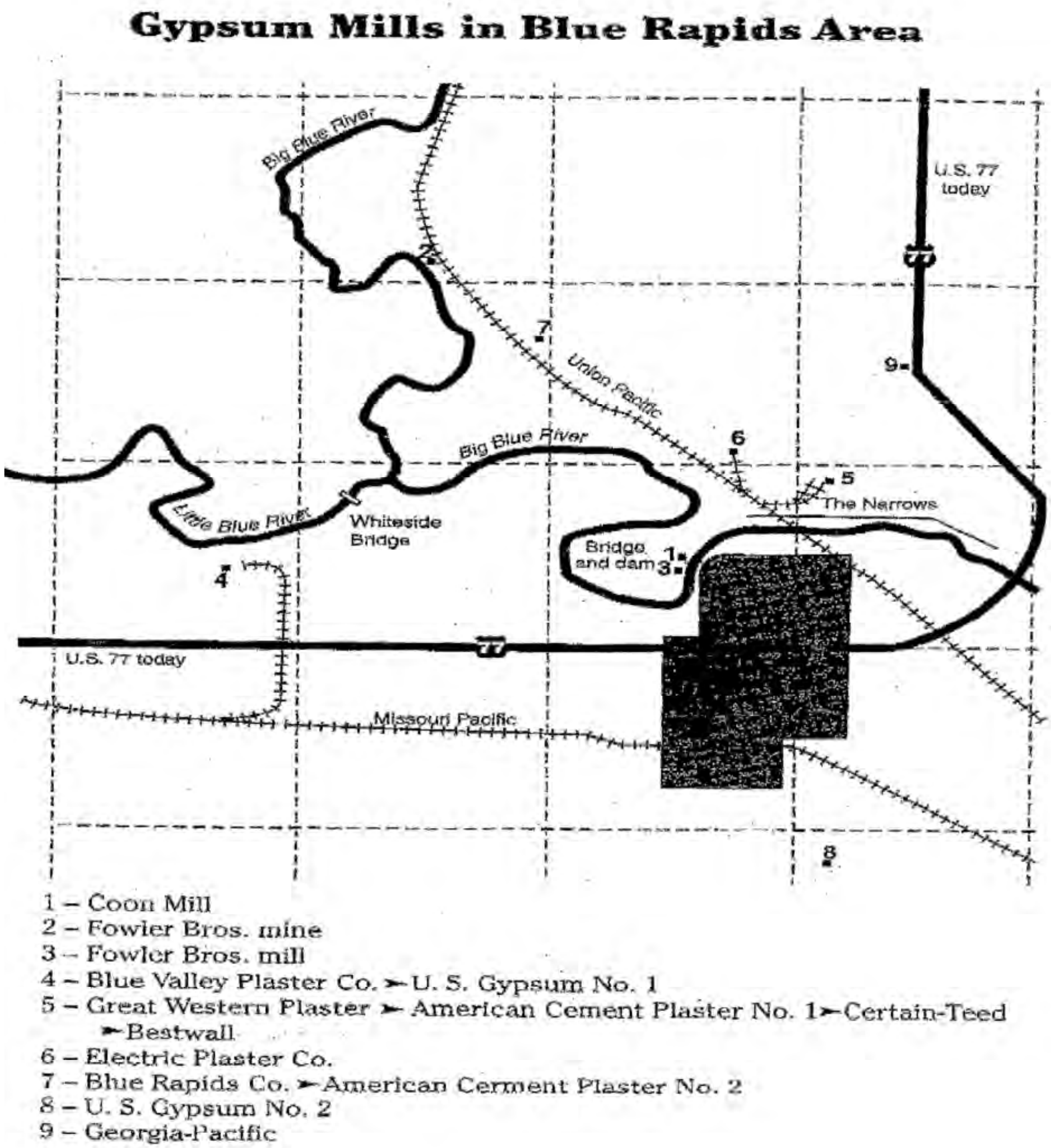

Figure 4 – Historical Map of Gypsum Mills and Mines

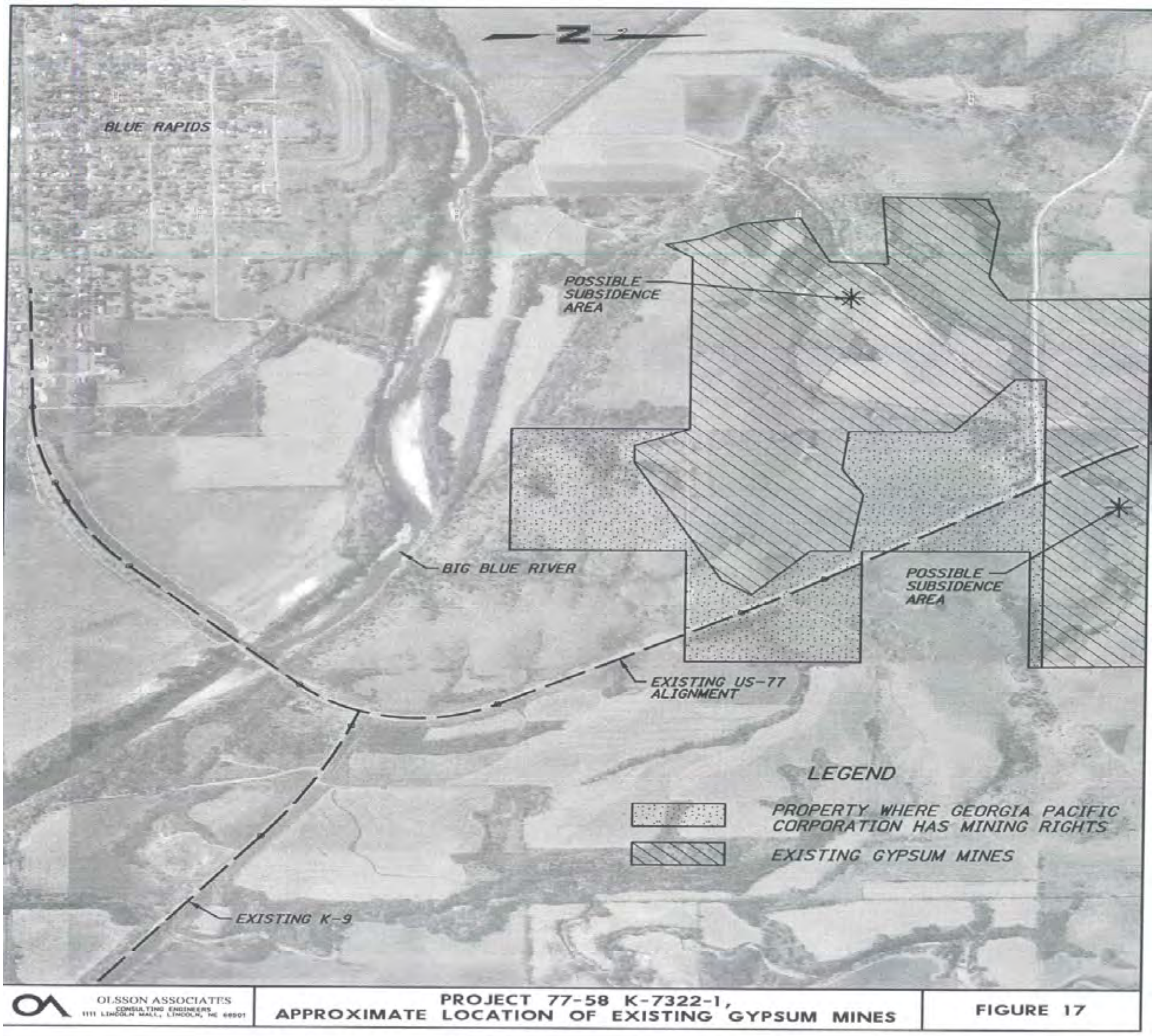

Figure 5 - Map of Possible Subsidence Areas in Georgia Pacific Gypsum Property

SELECTED ALIGNMENT #1

The first alignment that we were tasked with was to the west of the existing roadway. The western portion crossed the flood plain on engineered fill; the proposed bridge was designed on an inclined curve. This was done to transition out of the flood plain to the top of the hill on the east portion of the alignment and to carry the designed speed. The Geology Section began their investigation. The additional subsidence areas were located, and drilling to design the 80 foot rock cuts was started. The geologic setting of soft shales and limestones made the backslope design very difficult without purchasing much more right-of-way. The cost for this alignment began to escalate: more right-of-way, rock excavation, and mine remediation. One additional issue was that the existing bridge condition had deteriorated, so the contractor would not be able to utilize the 60,000 plus yards of excavated material for the fill on the opposite side of the river. Archeologically, this route was near the first surface gypsum mine, and it also had a potential of being an Indian encampment. This alignment was abandoned!

SELECTED ALIGNMENT #2

This alignment was located approximately ½ mile to the east of the first route. This route was selected due to increased allowable speeds, minimal rock excavation and no impact from the mines. This new alignment was staked by the Geology Section, and we mobilized to go back to the field. We got a call from the designers that the roadway geometrics between US-77 and K-9 were not desirable. This was due to the superelevated section of US-77 needed to maintain the design speed and its intersection with K-9. The design had limited the sight distance for west bound K-9 and produced a short grade that semi-trailers would have not been able to traverse.

SELECTED ALIGNMENT #3

Alignment #3 was chosen, and this route was approximately 70 feet to the west of the existing roadway. The grade was raised to eliminate some of the required rock excavation. Rock excavation was limited to the west side of the roadway, and the cut was approximately 50 feet deep. The material excavated could be used to construct the new embankments on the same side of the river. We completed our investigation and designed the new backslopes to fit within our available right-of-way. Estimated quantities for the rock excavation were less than 15,000 cubic yards with a total project cost of 7.8 million dollars. We went to Field Check, which initiates the start of the bridge foundation investigation. The Geology Section began working on the bridge foundation recommendations. Five borings had been completed when a Public Interaction Meeting was held to discuss the project with the locals. Several people did not like this alignment, one individual in particular. After the meeting, phone calls were made, and we were told the alignment was to be moved. The letting date had to stay within the same fiscal year for budgetary purposes, so only an additional 3 months were given to redo the design. We were back to square one for the fourth time.

FINAL ALIGNMENT

The final alignment was chosen after several discussions with the landowners and KDOT personnel. Alignment # 4 was a modification of one the earlier routes from the Preliminary Engineering Study. This new route was located east of the existing highway. Rock excavation was reduced by 15,000 cubic yards, which was a savings of approximately 180,000 dollars. However, the new alignment would require the relocation of a transmission power line at a non-participating project cost of 800,000 dollars. This moved our estimated project cost to 9 million dollars. From a geotechnical standpoint the change required us to re-drill a major portion of the project including the Bridge Foundation Investigation. The new bridge would be a 760 foot long 4 span structure. One abutment would be founded on steel H-piles, and all the other elements would be set on drilled shafts.

To meet the accelerated deadline we placed 3 drill crews on the project, and the project was completed 2 weeks ahead of schedule.

CONCLUSION

As a geotechnical professional one should expect minor changes and tweaks to an alignment. This project was an exception. Due to these changes there were many power auger soundings and core holes that were not utilized in the design of the project. Two of the core holes were stratigraphic determination holes that were in excess of 100 ft. deep. Even with the many changes, the project was let on time, and the price was approximately 1 million dollars under the engineer's estimate. As an Engineering Geologist one should never believe the designers when they say "Trust me, this is the last change".

DIGITAL TERRAIN MODELING TECHNIQUES FOR A BETTER SUBSURFACE SOIL LAYERS REPRESENTATION

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DIGITAL TERRAIN MODELING TECHNIQUES FOR A BETTER SUBSURFACE SOIL LAYERS REPRESENTATION

Alexander Mabrich, PE, MSc

Abstract

Surveyors provide a wealth of data to optimize the civil aspect of capital projects, such as electronic ground data in the form of elevations, ground features, terrain configuration, etc that taken for further processing give the engineers the ability to see a graphical representation of the working site in their computer monitors.

Unfortunately in some cases, this information is taken with one discipline in mind: civil engineering, whereas much needed and specific information is required for geotechnical professionals. Data received from surveyors sometimes lack an accurate representation of the ground or terrain features like slope breaking lines, or water flow lines. As the Digital Terrain Modeling or DTM is received in the office, we cannot just assume that this is the accurate representation of our project and when sometimes a site visit is not possible we should employ a DTM post-processing software to evaluate the quality of the information received.

The purpose of this paper is to explore available software techniques that could be used to better analyze the data given and interacting with a geotechnical database be able to model a better representation of ground and subsurface conditions in our projects. This paper discusses the different methodologies used to take ground information and thereafter create a proper DTM model of the surface conditions. A Geotechnical database needs also to be properly configured in order to interact with the ground information and depending of the amount data collected we can create an accurate representation of the soil layers in an electronic format, rather than creating soil profiles, interpolating between them and manual connecting the soil layers in a graphical borelog profile report.

1 SUBSURFACE DATA

Given the ground configuration, geotechnical engineers are required to provide information about the subsurface conditions of the worksite. Lamentably, not many tools are available for that purpose as they need to rely in just two major sources of information: Geological Maps and Soil Borings.

Geological maps are done showing a large area but due to the small size of the project become too generic to represent the specific conditions of our project.

Soil borings, being the next source of information, are taken along the project, but at predetermined intervals just based on spacing rather than actual ground conditions. For example, a typical roadway project may require soil borings every 150 m feet just along the centerline of construction, or a soil boring at a foundation location. Using a combination of these two sources of information, the geotechnical engineer is summoned to determine an "actual" subsurface map of the project.

Later, the soil information is drafted in the final plans of the project, along the alignment, profile and sometimes cross sections, then a soil boring or geotechnical sheet accompanies the final delivery upon which different specialties (drainage, structures, hydrology, etc.) use the information provided and sometimes without knowledge of the level of extrapolation or assumptions taken to show these data into the final plans.

2 GEOTECHNICAL DATABASE

Most of the soil information received from the field is stored in paper copies or spreadsheets. This valuable information is collected on a project basis and after they materialize in the geotechnical report or plan sheets, it is stored in file cabinets or hard drives. Data recollection is difficult afterwards since it all depends on the particular engineer in charge, and his method of storing and indexing the gathered data. Major organizations, like State DOTs or Departments of Transportation, are always in a constant struggle for the best way to organize this information and possibly make it available to the general public.

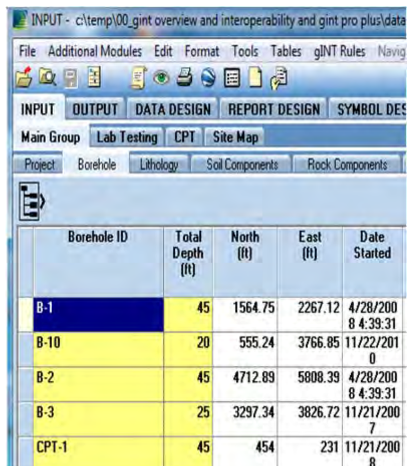

Figure 1. Database needs to record coordinate information that can be used in other engineering disciplines.

This vast amount of information needs to be managed on a project basis and also incorporating soil samples information collected over the years around the project vicinity. A specially designed geotechnical database needs to be created that would serve most of the disciplines involved into a civil project.
These could be the following aspects to take into consideration:

Data Organization and Accuracy: Each organization needs to define the information that would be included in the database. As many disciplines and specialties will use this data, the difficult part is to achieve a consensus of at what level of accuracy the information needs to be taken from the field and recorded into the database. As civil projects go, they may only required information based along a reference line by station and offset, other by Cartesian coordinates (XYZ) or if used by GIS professionals, a latitude and longitude data needs to be recorded. This is just the starting point, but what about the other types of data required: hydraulic, well, spt tests, environmental, etc. It is up to the organization to balance the amount of information that needs to be stored and what it is actually going to be used without detriment of creating a database too large or too small.

Data Consistency and Validation: Validation and consistency routines need to be implemented into the database as many operators will be entering information. Being that some of the soil descriptions are based on subjective observations, the geotechnical database should have tools to standardize the data entry process so little is leave to a particular personal preference.

Data Interoperability: As many disciplines will interact with the database, special routines or translation/correspondence files should be developed if the information stored needs to be sent to more specialized software for environmental, geological or mining purposes. The interoperability must guarantee that consistency and quality of the information is not diminished during the transferring process. Anyhow, the geotechnical database needs to remain the single source of truth or central data repository even if more information is required from other disciplines.

Reporting: the database, remaining the central repository of all geotechnical data, should provide flexible reporting options that adapt its output to the professional requiring the data. Data reporting for an environmental engineer has different formatting and information than a report for a structural, roadway or site engineer.

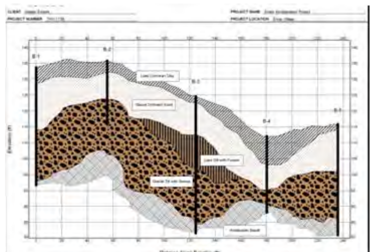

Figure 2. Flexible reporting must be part of the database design.

3 DIGITAL TERRAIN MODELING AND SUBSOIL STRATIGRAPHY

A DTM or Digital Terrain Model is an electronic or digital 3D representation of the ground surface created from elevation data. Any civil engineering project needs to start with some kind of ground information. Depending on the stage of the project, planning, design, or construction, the ground information is recorded using different tools at a various levels of accuracy and precision.

During the design process, a more refined ground information is needed. Low level aerial photography, and/or electronic surveying techniques are employed not only to take elevations of the ground but also to record the topographic information. Elevations of buildings, roadways, and water bodies are also taken in and showed with the ground data. All these information will become part of the final topographic map of the project.

As the DTM model is finished, engineers will take ownership of the model and use it accordingly to their needs using appropriate software tools to analyze the model. Hydraulic engineers may review the watershed or basin areas, roadway engineers will plan the future alignments and profiles, right-of-way professional will check the impact of the project in adjacent properties, and at one point geotechnical engineers will use it to review the ground topography.

Commercial software provides the necessary tools to analyze the given ground or DTM information. Elevations and slope ranges, watershed delineation, water paths, and pattern flows are familiar terms to professionals dedicated to review and manipulate DTM data.

Being that DTM is a useful tool for engineers, sometimes little or no attention is given to the possible subsurface configuration. Surveyors and civil engineers taking the ground information pay more attention to the physical features of the worksite rather that the different type of soils or the actual topography (e.g. breaking slopes in a "flat area"). This information is frequently not surveyed or recorded in the field notes as every type of soil is just considered "dirt".

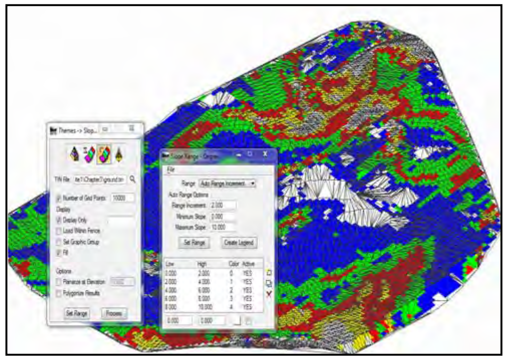

Figure 3. DTM showing ranges in slopes on the terrain

An integrated solution that uses the subsurface information collected, validated and stored in a geotechnical database and with the abiility to provide enough information to create a DTM model of the subsoil layers is proposed as new working technique for a better subsoil surface layer representation.

4 EXAMPLE OF AN INTEGRATED SOLUTION: VIRGINIA DOT CASE STUDY

Background Information: Virginia DOT (VDOT) was using gINT software as a way to standardize the reporting of geotechnical borelog reports. A geotechnical database was created and the proper reporting templates were developed addressing the needs of multiple type of users across the State. The standard database format and related libraries are maintained by the VDOT Central Office and published in their website for State personnel and consultants to use and submit their work.

VDOT also uses GEOPAK Civil Engineering Suite and MicroStation as the software of choice to design roadways and prepare final plans, in which they require the plotting of Geotechnical Boring sheets and also showing the borelogs in plan, profile and cross section along the roadway.

The Geotechnical and Civil Departments were disconnected since, after processing the information and generating the reports in the Geotechnical software, the roadway engineers needed to redraw the same reports in a CADD environment using customized macros. Apart for the extensive amount of time employed in this task, there was no option to generate a subsurface DTM of the multiple layers of soil present in the project.
The creation of subsurface layers was done by drawing lines along the existing ground lines following the soil boring stratigraphy on each of the roadway cross section cut.

Solution: Bentley Systems proposed an automated solution to bridge the gap between the geotechnical and roadway engineers. A new series of specially designed Geotechnical tools need to be programmed into VDOT's Civil Engineering solution. This new tool will have the ability to read the geotechnical database in use and load the information needed into a new civil oriented geotechnical database.

VDOT Civil Engineering software solution will then provide the CADD engine to draw the borehole locations and stratigraphy in plan, profile and cross section view combining the coordinate information stored in the geotechnical database with the station and offset calculated from the roadway alignment.

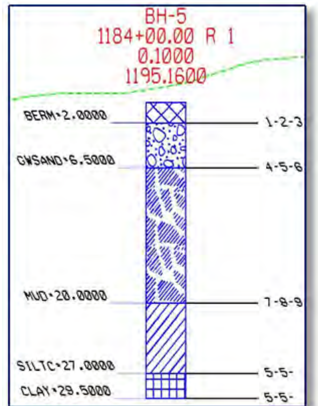

Figure 5. Borehole data could be represented in plan, profile and cross section views.

Civil Engineers will also create and manage the ground DTM of the project, and using the soil boring information and depth will automatically generate subsurface DTMs for each layer of soil present. The civil and geotechnical engineer will decide the validity of the new subsurface DTM's if a minimum number of observations are not present. It is an engineer's decision to adjust the acceptable level of extrapolation to use in order to create these DTM layers.

Results: VDOT benefited greatly as the newly developed standard geotechnical database allowed different disciplines to access and query information across multiple projects. Incorporating GIS capabilities and web technology, all geotechnical information in the State of Virginia is on-line and ready to be accessed for viewing, downloading and printing.
Moreover, using the information collected, either from old projects or new boreholes, the engineers were able to create DTM's of the subsurface layers allowing for better modeling of the existing conditions. As a result, their accuracy in calculating earth movement improved and reduced the overall cost of their projects.

5. CONCLUSION

The need of integrating the different disciplines involved in a civil project is in great demand now as the need of a seamless transfer of information will reduce risk and minimize errors. Therefore, Geotechnical engineers are becoming a key factor in the process as any civil project starts with ground information. Geotechnical software tools and techniques need to be used to interact with other disciplines and other software programs. The ability to manage and filter borehole data and displaying it in plan, profile and cross section views allows the engineer to create a subsoil DTM. Therefore, representing the ground and subsoil layers with greater accuracy alllows the engineers to design the most appropriate structure to the soil conditions and better calculate earthwork quantities.

The VDOT case study has proved that geotechnical data can be shared with roadway engineers and that DTMs can be used by both parties with the caveat that its creation using the shown techniques is a task of equal responsibility for geotechnical and civil engineers.

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Online Geotechnical Database Considerations and Data Sharing

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ABSTRACT

Geotechnical data for a project can come from two primary sources: a general, geologic review of the area as well as site-specific investigations.

There are many ways to provide data to optimize the civil aspect of capital projects. Today's digital data (e.g., elevations, ground features, terrain configuration, LiDAR profiles, satellite pictures) with further processing give engineers the ability to see a graphical representation of the working site in their computer monitors. Many of the sources to get detailed information about the project vicinity are widely available, but not project specific.

Gaining insight into subsurface conditions is done on a project-by-project basis via geophysical methods, cone penetrometer testing (CPT), dilatometer testing (DMT), and standard borehole explorations. Linking information from separate projects in the same area is rarely done. This is, in part, because by traditional work methods, data exchange is not possible as proper software tools are not available.

Organizations utilizing a robust geotechnical database are able to use general project data as well as information from projects in the vicinity to quickly and easily gather valuable information with minimal work time.

This presentation will review two state-supported online geotechnical databases, and review technical components, development methods and system considerations that Minnesota DOT and Virginia DOT have encountered during their on-line database implementation, as well as current capabilities of their systems.

Lessons learned and benefits expected will be reviewed. Future possibilities as technology advances and becomes more accessible to organizations will also be discussed.

INTRODUCTION

"Online Geotechnical Database Considerations and Data Sharing" means the exchange of geotechnical data, and not just geotechnical reports, between users, while using online resources.

We will start with a background and history of geotechnical databases and the sharing of data, and then focus in on two examples: Virginia DOT and Minnesota DOT. Each state DOT has an on-line geotechnical database management system.

We will review:

- Some of the evolution of the systems
- Why the online systems.were developed
- Items considered when developing the network, including technology
- Lessons learned throughout this process
- New data exchange ideas for future use

TRADITIONAL DATA COLLECTION AND STORAGE

Subsurface data is traditionally exchanged on paper or electronically (raster data). It is presented in a final format such that, if another user wants to use the data again on another project or as an investigation into the current, they will have to manually read data points and re-enter the data into another media, analysis program, or even a database for further analysis *(1)*.Traditional formats include paper, CADD drawings, and PDF *(1)*. Information includes geotechnical logs, fence diagrams, and lab data. Even data from geologic quadrant maps is often taken from a paper map. If this paper or electronic paper format is stored in a warehouse (physical or digital), a user must read pertinent data and manually re-enter it for further analysis or for inclusion in new reports and analysis programs.

This process is fairly standard in the geotechnical community. Although this is data reuse, it is not efficient. Much personnel time is spent going through historic records, searching for the relevant data. Once the user has the data, it is a time intensive process to include the data in analyses, drawings, and reports. Additionally, manually entering information always allows for error, and therefore another layer of review is necessary.

Contrast this to GIS systems . GIS specialists regularly use data from many database resources. The data is used over and over. GIS specialists build several queries to evaluate scenarios and optimize decisions. GIS personnel will create maps and reports based on the data pulled from a database within minutes instead of days. Comment: this paragraph read too much like a Power Point sales pitch rather than a formal paper, and was edited for style.

STATE OF THE PRACTICE

Although slowly changing towards a data reuse/sharing goal, the geotechnical industry is still dominated by a "one and done" outlook of data. "One and done" refers to the practice of getting the geotechnical log in paper, PDF or CAD format forthe user. The paper and digital copies are placed in storage and are not used again unless there is another project on the same site at a later date. This is in part because of cost: the cost of the geotechnical investigation is typically a very small percentage of the total project costs *(8)*. A geotechnical database has not been on the forefront of discussion until recently as development of a custom geotechnical database management system is often thought of as cost prohibitive. In 2005, it was noted in a study by Geosyntec for the Ohio DOT that few DOT's have a data management system for geotechnical data *(10)*.

However, as technology advances, setup and maintenance costs are reduced. Often the technology infrastructure is already present in an organization. Agencies such as Departments or Ministries of Transportation are moving towards requiring that data be submitted in electronic format so that data can be transferred into an organizational database.

In a 2008 phone survey performed by Bentley's gINT to DOT's, half of the respondents did require electronic data submittal in formats such as Excel, gINT, MicroSoft Access, etc.Today more and more government agencies are looking into a geotechnical database management system to store their data and automate reporting and analysis processes. However, there are many things to consider and learn along the way while implementing such a system.

COMPONENTS OF A GEOTECHNICAL DATABASE MANAGEMENT SYSTEM

A single geotechnical database management system is composed of many parts. Even an off-the-shelf geotechnical DBMS requires additional items to function. And many of these parts are not geotechnical. Rather, the additional parts have to do with data storage, GIS interactions, and applications to enable user interaction.

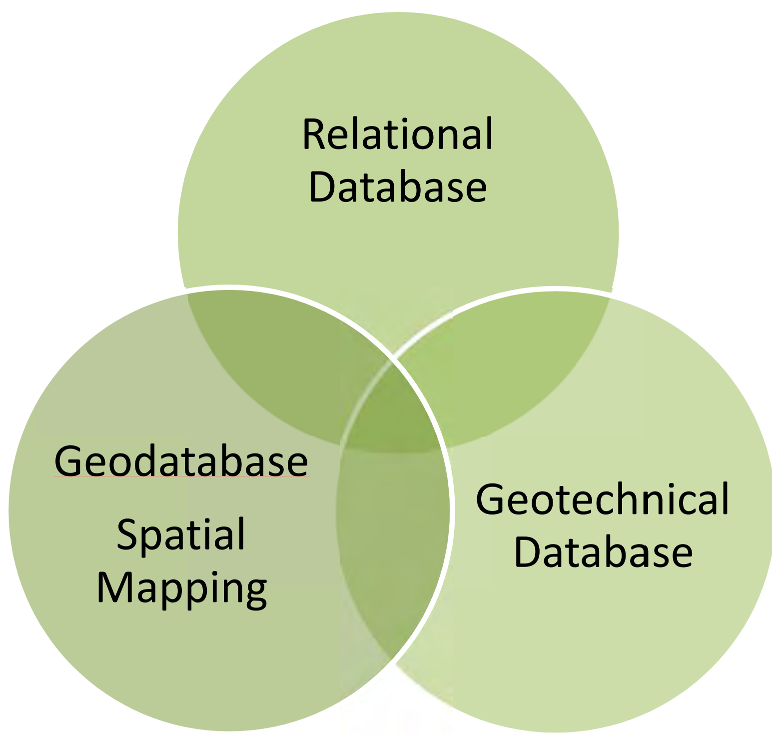

Figure 1 - Components of a Geotechnical Database Management System

Whether a custom system or off the shelf, similar considerations must be given. Figure 1 details the primary components, but there are many other details: an internet web server, a site host, custom scripting to work with database systems, and more. Fortunately, as technology advances, many organizations have standards for these items in place already. It is more of a matter of the geotechnical department and IT working together and learning each other's requirements and technical language.

Most database systems require a relational database management, such as Oracle or Microsoft SQL server, which are two frequently used database management systems in medium to large organizations. The geodatabase enables spatial mapping and querying by the user. The geotechnical database is complex and requires considerable effort to develop.

GEOTECHNICAL DATABASE CONSIDERATIONS

Before implementing a geotechnical database management system, several decisions regarding all geotechnical information, and how to include it in the GDBMS, must be made . The first step an organization must make is deciding what processes they wish to automate by creating an online GDBMS. Most of the processes involve using geotechnical data (borehole logs, fence diagrams, lab reports,) and site condition analysis . The online part of the GDBMS is one way to allow users to quickly and easily access the data which will be used to build a custom geotechnical database.

An organization must look at all geotechnical data and decide what elements they wish to include in the database. Data may include: in situ tests(standard penetration test , cone penetrometer test, dilatometer test), laboratory test data, groundwater monitoring information, slope inclinometer readings, geophysical test results, pile driving data, etc.

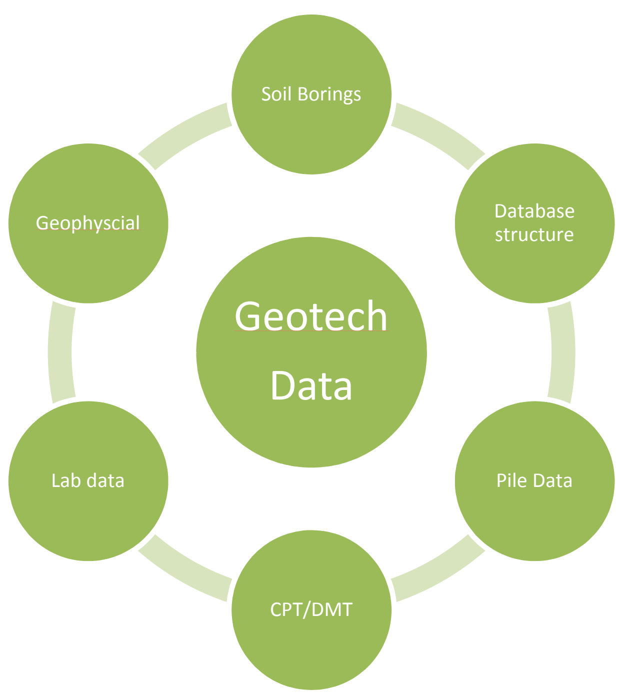

Figure 2 - Geotechnical Database Considerations

Minnesota DOT and Virginia DOT currently limit their geotechnical database to site investigations of SPT, CPT, and DMT. However, both of their systems are capable of including more information..

Often organizations wish to streamline the generation of specific geotechnical reports (borehole logs, fence diagrams, lab reports). However, organizations should look not only at the numbers on the reports, but the data used to calculate the results. A properly built database should store the raw data and then be programmed to perform calculations.

Additionally, once data has been entered into a database, another consideration is what data will be necessary for analysis in other programs. For example, Virginia DOT added a scripting function which enables the generation of input data sets for five design analysis programs they use, saving time and effort in setting these files up *(9)*.

Finally, a database has to be able to grow or adjust for new technologies and considerations. For example, Minnesota DOT has used the same database since 2003. In 2006, CPT testing had increased enough in Minnesota that they added the capabilities to import and analyze CPT soundings into their database. The table included calculations and analysis schemes *(5)*.

TIME SAVINGS

Both Virginia DOT and Minnesota DOT acknowledge that being able to quickly access data in useful format is one of the most valuable benefits of their GDBMS, allowing them to save personnel time and effort on projects.

During a review process before developing the GDBMS,Virginia DOT reviewed what some of their time intensive activities were, and found ways to include them in the database system to become more efficient.

For example, users can:

- Access borehole data in a gINT format and have the data available for application and analysis.
- Download data in CSV format, which can be used in other applications *(9)*.
- Select a maximum of 10 boreholes and generate a Drawing Exchange Format (CAD) file of a fence diagram*(9)*.
- Create PDFs of boreholes for inclusion in reports *(9)*.
- Generate input files for 5 analysis programs typically used for slope stability, pile analysis, et cetera. This means the data necessary for an analysis is picked out and put into the correct format for specific applications *(9)*.

Virginia DOT estimates that reuse of data from their online GDBMS can result in 16 person-hour savings per project to gather and process the data *(5)*. At an average rate of $100 per hour that is $1,600 per project. Virginia DOT has been approving approximately 100 bridges per year for construction which results in an approximate annual savings of $160,000 per year *(5).*

Minnesota DOT estimates cost savings per project can range from $250 to $12,000 with an average of $1,000 *(6).* Derrick Dasenbrock, Foundations LRFD Engineer of Minnesota DOT, stated “We easily save a great many minutes (which add to hours) by being able to pull electronic files.”

BENEFITS OF AN ONLINE GEOTECHNICAL DATABASE MANAGEMENT SYSTEM

There are many recognized, overall benefits to an on-line GDBMS, all of which add up to time savings. Specific benefits realized by Minnesota DOT and Virginia DOT include:

- Identification of trouble points – all data is stored in one location and reviewing data becomes easier. One can query and search for known qualities that may indicate problem points during construction for the lifetime of the structure.
- Reuse of existing data – on a single project or on separate projects in the same area. On a single project you can export data to be used in different analysis programs without rekeying data into the other program. For separate projects in the same area, you have the added dimension of time. Anywhere there is a road, a geotechnical investigation has been completed in the past. Reusing the data allows you to have an idea of subsurface conditions before going in for upgrade, maintenance or restoration work. This reduces the number of boreholes (confirmation and additional information), laboratory work, and results in a faster turnaround time overall. It also lets you know general conditions before going in for a new project which can help you better plan a geotechnical investigation.
- Correction of errors in existing data – rules can be implemented to ensure that data entry standards are maintained. Standards can be implemented to prevent errors being entered in (and remaining in) the database. Common errors include keying coordinates incorrectly or duplicate borehole numbers on a project.
- Instant access to legible data – a user can view information as soon as it is in the database format. One does not have to wait for items to get typed up *(5).*
- Quick access of data for analysis and reports - all data in that database are readily available for analysis and report generation. You no longer need to search for data in several spread sheets throughout the office *(5)* or in a project file system on a server (or colleague’s computers).
- Faster turnaround time for reports – with the click of a few buttons reports are easily generated. You do not have to wait for a paper copy to be typed. Rather the database generates reports on demand. Also, if the database is well designed the calculations are done by the computer and not the individual which means review time is reduced, as is time for a report to be completed *(5).*
- Simple to back up the data *(5).*

Once in a robust database, interpretation and re-use of geotechnical data becomes much simpler. Figure 3 shows a Digital Terrain Model (DTM) being generated using data a CAD application with data directly from its geotechnical database. A user can pull data in for multiple material layers and perform advanced analyses such as volume calculations and cross-section views within the CAD application.

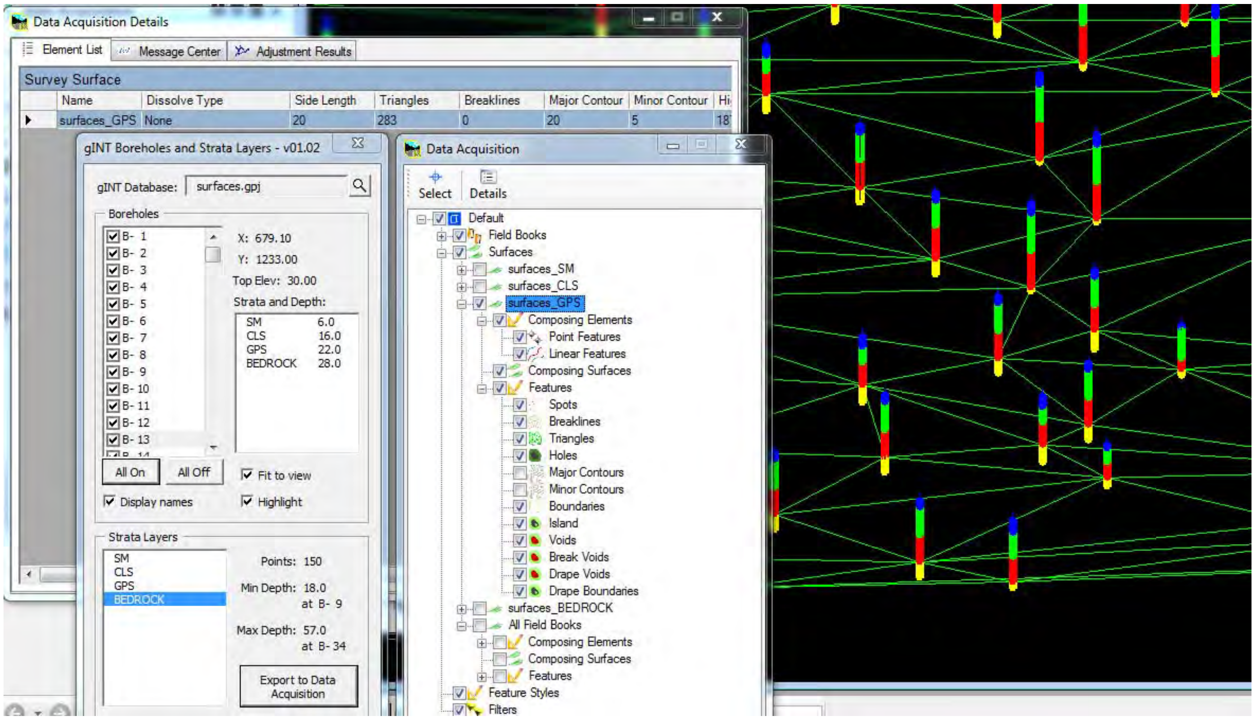

Figure 3 - use of geotechnical data from a database live in a CAD application

CONCLUSIONS

- Thorough consideration to a geotechnical database must be done by parties involved.
- Consideration should be given as to which processes an organization wishes to streamline.
- Consider all geotechnical data.
- Consider present analysis needs as well those for future use.

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Validation of Interferometric Synthetic Aperture Radar as a Tool for Identification of Geohazards and At-Risk Transportation Infrastructure

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Disclaimer

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ABSTRACT

As part of the USDOT-funded research program RITA-RS-11-H-UVA, "Sinkhole Detection and Bridge/Landslide Monitoring for Transportation Infrastructure by Automated Analysis of Interferometric Synthetic Aperture Radar [InSAR] Images," the authors broadly validated the use of InSAR data as a tool for early detection of geological hazards and failing infrastructure, including sinkhole development, potentially dangerous rock slopes, distressed bridges, rock buttresses, and other geotechnical assets. By bringing the InSAR dataset into a GIS dataframe and correlating the data to published maps of sinkhole locations and karst terranes, the authors were able to correlate average displacement velocities of InSAR data points (scatterers) with respect to their proximity to mapped sinkholes. Additionally, the authors correlated the InSAR signal characteristics with kinematic analysis of rock slopes using point-cloud data generated using digital photogrammetry and LiDAR. Lastly, the displacement time-series of the InSAR scatterers were used to screen for compromised geotechnical assets and infrastructure, and the findings were strongly confirmed by field inspection of distressed bridges and a failing rock buttress. The validation of InSAR data for these purposes thus allows generation of GIS-based geohazard and at-risk infrastructure/asset maps and provides the opportunity to augment or eventually replace a periodic inspection-based infrastructure management system with continuous performance-based system.

INTRODUCTION

By combining several overlapping images of the ground using millimeter-scale wave radiation, Synthetic Aperture Radar (SAR) takes advantage of the motion of a satellite along its flightpath to create a long synthetic antenna, thus resulting in an image of much higher resolution than the one that would be created using a single image from the real aperture. The radar image contains both amplitude and phase information of the backscattered radiation from each pixel[1]. When two images of the same location taken at different times are available, the phase information can be used to evaluate the local topography (InSAR – Interferometric SAR) and, if combined with already existing elevation information, it can be used to evaluate the changes in elevation of each pixel (DInSAR – Differential InSAR) (*1*). A major limitation of these techniques is the phase distortion introduced by the changes in atmospheric water vapor content between acquisitions, resulting in erroneous evaluation of ground displacement. If several images of the same location are available, this error can be greatly reduced by identifying those pixel displaying stable scattering properties over the entire dataset. These pixels, called Permanent Scatterers (PS), can be used to remove the atmospheric interference thereby achieving a much higher resolution in detection of elevation changes. This technique is known as PSInSAR (*2*). PS are often due to man-made structures thus showing higher density in populated areas. To allow detection of changes in rural regions, the PSInSAR technique was extended to identify larger geographic areas exhibiting coherent spatiotemporal behavior. When these Effective Areas (EA) are referenced to Distributed Scatterers (DS) the resulting technique is called SqueeSAR (*3*); it is often coupled with PSInSAR to evaluate topographic changes over time (*4*). The authors use the term InSAR as a general term for all interferometric SAR applications related to topographic change and infrastructure evaluation. Under ideal conditions, changes in 0.1-in scale can be detected, and displacement and surface kinematics can be evaluated.

While SAR data has been available since the 1950s (*5*) and airborne InSAR was first used in the early 1970s (*6*), it was not until the 1990s that InSAR was used to investigate topographic change over time (*7*). Most of those applications were for large-scale, slow-moving topographic changes, such as slowly-moving landslides (*8*) or changes in rock-glacier mass (*9*). Applications to smaller-scale phenomena, such as formations of sinkholes, activity on rock slopes, or distortions to bridges or rock buttresses, have generally been targets of investigation for InSAR only more recently; furthermore, most investigations have been in relatively flat-lying topography and tectonically simple geology.

The authors evaluated the use of InSAR for such evaluations by bringing the InSAR dataset into a GIS dataframe and correlating the data to sets of control data. For karst geohazards, these correlative datasets included published maps of sinkhole locations and karst terranes, as well as field validation. For rock slopes, the authors correlated the InSAR signal characteristics with kinematic analysis using point-cloud data generated using digital photogrammetry and terrestrial LiDAR. Lastly, the displacement time series of the InSAR data were used to identify potentially compromised geotechnical assets and infrastructure, and the findings were evaluated by field inspection of distressed bridges and photogrammetric time-series analysis of a failing rock buttress. The validation of InSAR data for these purposes thus

[1] A pixel is the smallest ground resolution element. For our data one pixel is 3x3m (10x10ft).

allows generation of GIS-based geohazard and geotechnical/asset database and provides the opportunity to augment or eventually replace a periodic inspection-based infrastructure management system with continuous, performance-based system.

UD DOT PROJECT "RITA-RS-11-H-UVA"

The authors are cooperative investigators in RITA-RS-11-H-UVA, a USDOT-funded project titled "Detection & Bridge/Landslide Monitoring for transportation Infrastructure by Automated Analysis of Interferometric SAR Images." The Research and Innovative Technology Administration (RITA) coordinates the U.S. Department of Transportation's (DOT) research programs. The purpose of RITA is to advance innovative and interdisciplinary technologies leading to improvements in the US transportation system. In order to evaluate whether InSAR data should be further subjected to algorithms intended to detect and quantify surface change and to evaluate infrastructure condition, it was determined that the data should first be broadly validated with regard to control or ground-truth datasets. The authors selected an Area of Interest (AOI) corresponding to one full Cosmo-SkyMed image tile of 617.8 square miles (40 by 40 km, or 1,600 square km) for data acquisition. The environment of the AOI is fairly mixed. Dense vegetation covers nearly half of the satellite tile, while active agriculture, fallow fields, infrastructure and towns (including Staunton, Stuarts Draft, Vesuvius, and Middlebrook) comprise the remainder of the area.

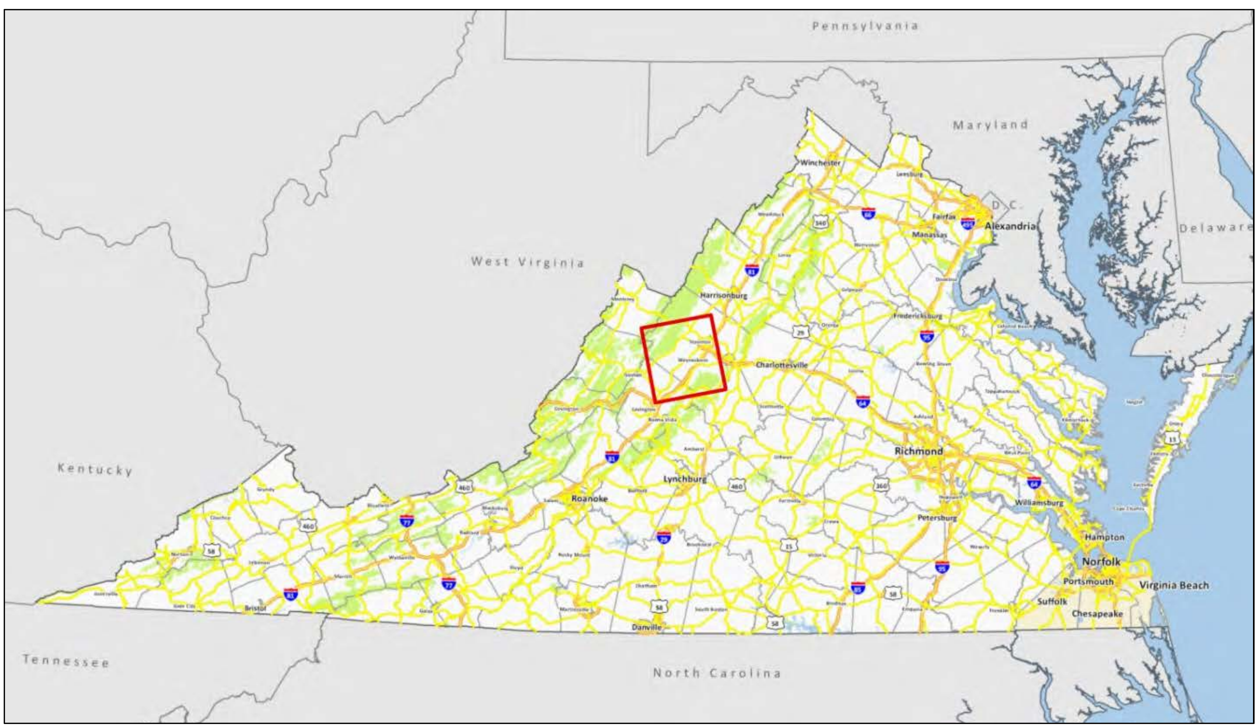

Figure 1 - Area of Interest

The AOI is a tectonically complex area spanning the Valley and Ridge and Blue Ridge physiographic provinces (*10*). Geological ages ranging from Holocene sediments to Precambrian granulite gneiss (*11*), with frequent unconformities, are represented within the AOI. The predominant tectonic framework consists of eastward-dipping thrust faults and decollements related to repeated orogenic cycles (*12*). The AOI contains carbonate, non-carbonate clastic, and metamorphic terrains, resulting in both rock slope stability and karst geohazards. The karst areas range in age from Cambrian to Devonian and formed during the Taconic and Acadian Orogenies and their associated divergent and inter-orogenic periods. Karst lithologies consist mainly of limestone and dolostone, while non-carbonate clastic lithologies consist of occasionally interbedded shales, siltstones, conglomerates and sandstones, and the metamorphic lithologies consist of charnockite, granulite gneiss, quartzite, and greenschist and blueschist-grade metabasalt. Figures 2 and 3 represent areas of karst and rock-slope geohazards, respectively.

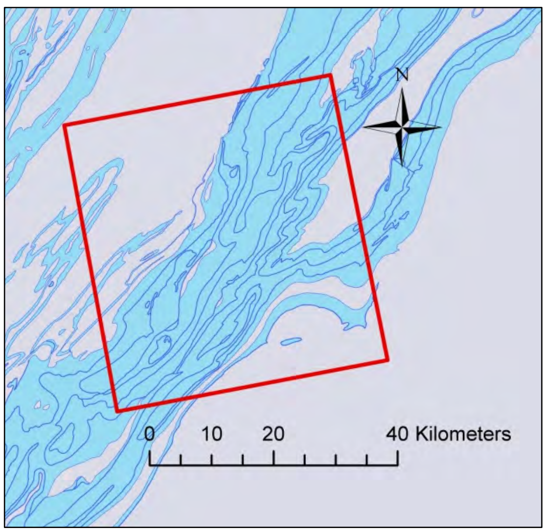

Figure 2 - Areas of Karst Geohazards (Blue) and AOI

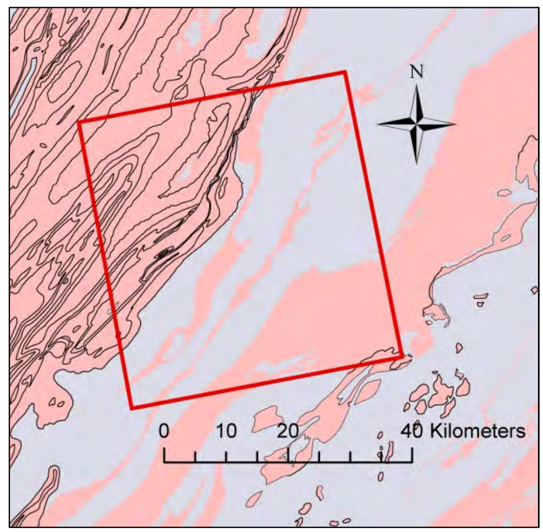

Figure 3 – Areas of Rockfall Geohazards (Rose) and AOI

Several control datasets exist for existing sinkholes; Figure 4 is an aggregate dataset of known sinkhole locations compiled from Virginia Department of Transportation records of repaired sinkholes and limited-release data from the Virginia Department of Mines, Minerals, and Resources. Figure 5 represents locations of bridges and box culverts within the AOI.

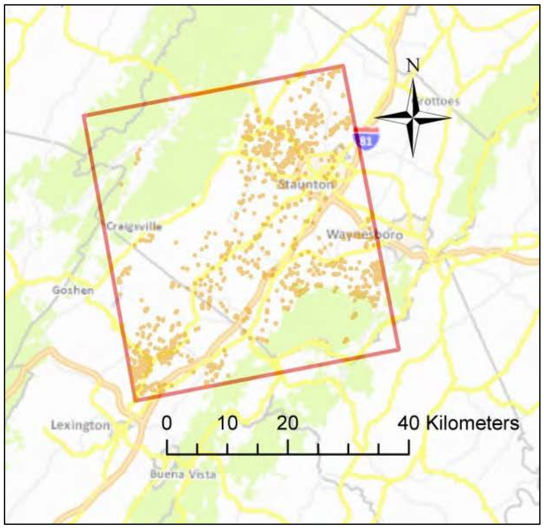

Figure 4 – Known Sinkhole Locations (Orange) and AOI

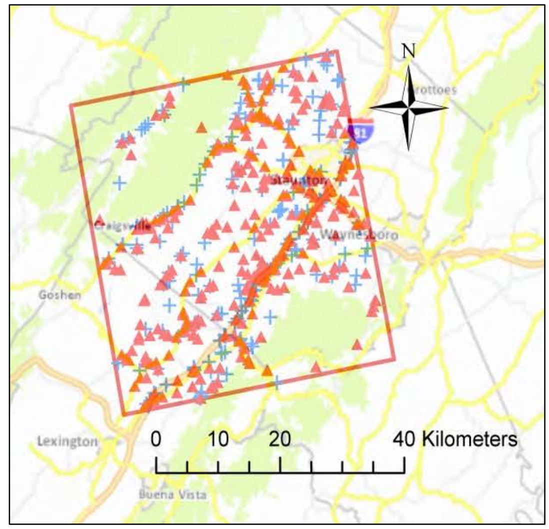

Figure 5 – Bridges and Box Culverts within AOI (Red Triangles and Blue Crosses, Respectively)

The authors selected COSMO-SkyMed, a constellation of four identical satellites built and operated by the Italian Space Agency, for data acquisition. Each satellite is equipped with an X-band SAR operating at 9.6 GHz. Between August 29, 2011 and October 25, 2012, 32 SAR scenes were acquired and were processed by TRE-Canada, Inc. The resulting dataset consisted of 298,954 PS and DS scatterers. The size of the AOI and a densely vegetated swath running through the AOI necessitated data processing in two clusters. Figure 6 represents the processed InSAR scatterers. Heavily vegetated areas proved to be an obstacle to InSAR data collection; however, such areas tend to have limited human population and infrastructure, and are therefore of lesser value in terms of surface analysis.

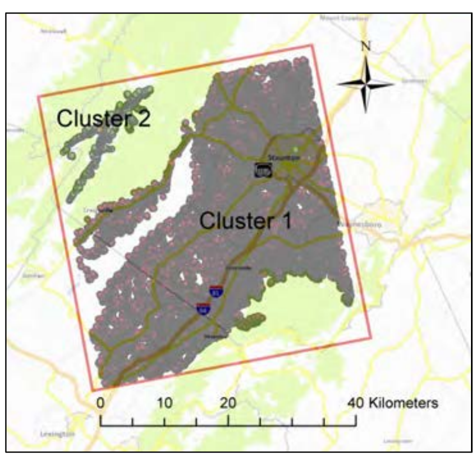

Figure 6 – Processed InSAR Scatterers and AOI

Each scatterer is associated with an identifier and a location consisting of latitude, longitude, and elevation, for each acquisition date. Each scatter is also associated with an effective area (EA), with PS having an effective area equal to zero, and DS having an effective area greater than zero. Additionally, each point is associated with a value for coherence (C), which is a representation of the stability of the point through time and with respect to its nearest neighbors. C values generally are considered to be reliable in the range of 0.8 to 1. Scatterers which have a motion greater than one-half a wavelength lose coherence entirely and are generally lost from the dataset. The data allow generation of a time-series of movement at each scatterer, with the time series of a PS indicating consistent and coherent movement at a very small geographic area, and the time series of a DS indicating movement over a larger area. Figure 7 represents such a time series. The negative slope of the time series indicates that the point is undergoing sinkhole-like subsidence.

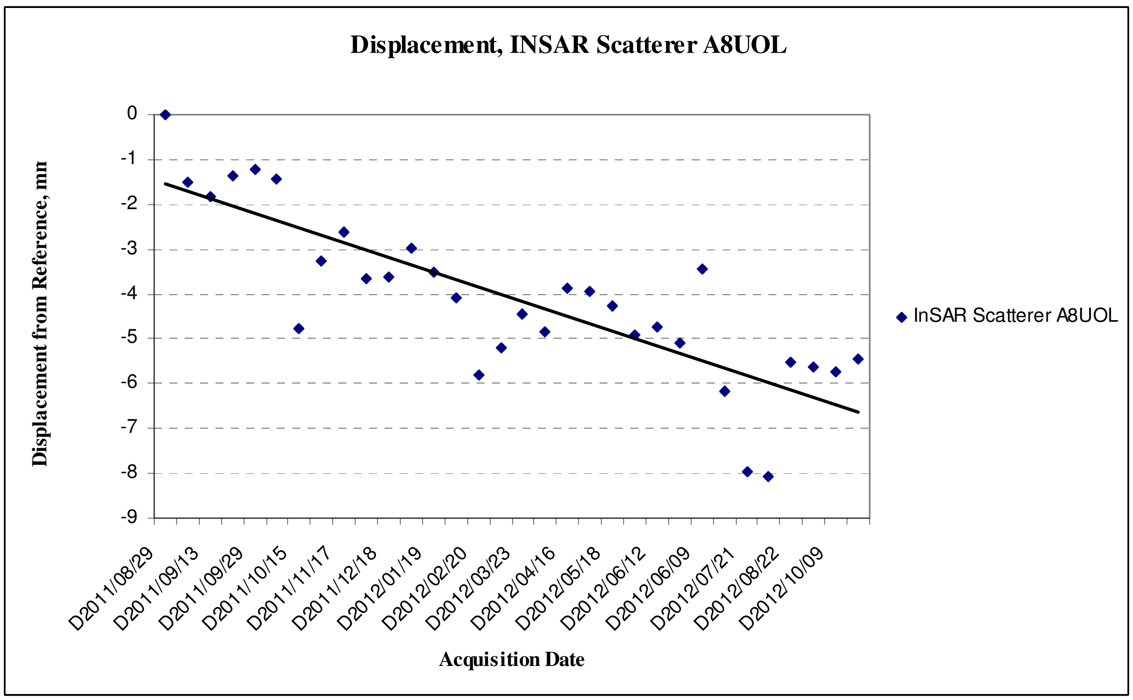

Figure 7 – Time Series of InSAR Point A8UOL

Time series for each, or a set, of InSAR points can therefore be evaluated for absolute motion – subsidence or rebound – as well as the velocity of that motion relative to surrounding points or with respect to their proximity to other features.

InSAR Validation: Karst Geohazards

The relative motion of the points is highly variable across the AOI. Areas of anthropomorphic activity, such as agriculture, quarrying, or construction may show a positive velocity, suggesting rebound due to stockpiling or staging activities, negative velocity, suggesting subsidence or settlement, or some combination of patterns. Isolating scatterers with respect to proximity to mapped sinkholes yields the data in Table 1.

Table 1 – Scatterer Velocity With Respect to Proximity to Mapped Sinkholes (mm/yr)

Cluster Number	Proximity to Mapped Sinkhole			
	Within100 ft	100 to 200 ft	200 to 300 ft	300 to 400ft
Cluster 1	-0.21	-0.07	-0.03	-0.045
Cluster 2	None	None	None	None

Evaluation of the InSAR scatterers yielded several phenomena proving to be developing sinkholes. Figure 8 shows the growth of a sinkhole, represented by InSAR points AO96K and AO96J, which developed during the data collection period.

Figure 8 – Sinkhole Identified by InSAR Points AO96K and AO96J

The average velocity of all scatterers in Cluster 1 was 0.22 mm/yr, reflecting a slight rebound running southwest to northeast across the AOI, possibly correlating to fault activity. The increasingly-negative velocity with increasing proximity to mapped sinkholes suggests very strongly that the InSAR data is reflecting true sinkhole activity, rather than a false-positive result. The velocity inverts at approximately 300 feet from the center of the mapped sinkholes, suggesting that this may represent the maximum average area of influence of sinkholes or sinkhole clusters in the Valley and Ridge Physiographic Province of Virginia. That there are no scatterers intersecting with mapped sinkholes in the region of Cluster 2 reflects the fact that Cluster 2 is largely outside of the area susceptible to karst geohazards (see Figures 2 and 6).

InSAR Validation: Rock Slopes

One large rock slope within the AOI had a geometry and radar reflectance characteristics suitable for InSAR data analysis. Field observations of the slope, Site Number RS-600-001 (Virginia State Route 600, River Road in Augusta County, Virginia) indicate dip slopes of dark blue-gray, fine- to medium-grained, cherty limestone belonging to the Licking Creek Limestone (Silurian-Devonian). The slope height and angle are approximately 120 feet and 40 degrees, respectively. A joint set meets the slope at a steep angle, resulting in slab failure where these joints intersect bedding planes, which range from 4 to 12 inches in thickness (*13*). The lithotectonic conditions result in small-scale, continuous, very wide-angle wedge failure along the entire length of the rock slope. The clasts resulting from the wedge failures are small, generally in the gravel- to cobble-size range. The slope behavior was characterized by digital photogrammetry and terrestrial LiDAR, which allowed the behavior of the slope as characterized by InSAR to be evaluated against the activity characterized by site-specific data collection. Figure 9 is a site image. The red circle is a figure for scale.

Figure 9 – RS-600-001 Site Conditions

Digital photogrammetry and LiDAR are both point-cloud data collection methods, which yield an XYZ file that can be brought into a GIS (or other geospatial) dataframe. This allows three-dimensional analysis of the rock slope. The authors used Sirovision® (version 4.1, 2011), a geology / geotechnical mapping and analysis system, to generate scaled 3D images of rock faces from stereo photographs. A second module, Sirojoint®, was used for limited geotechnical and structural analysis of the 3D images. The data resulting from Sirovision® was then brought into ArcMap® 10.0, and surface analysis was used to interpret the kinematics and geomechanics. Figure 10 is an aggregate of the digital photogrammetry data and interpretation brought into an ArcMap® dataframe, and relates the field conditions to the GIS analysis. The surface analysis highlights portions of the slope of different azimuthal aspect. The yellow wedges are surfaces formed by the intersection of the joints and the bedding. The purple colors represent incoherent slope aspect along the entire toe of the slope, indicating a broad failure mode along its entire length.

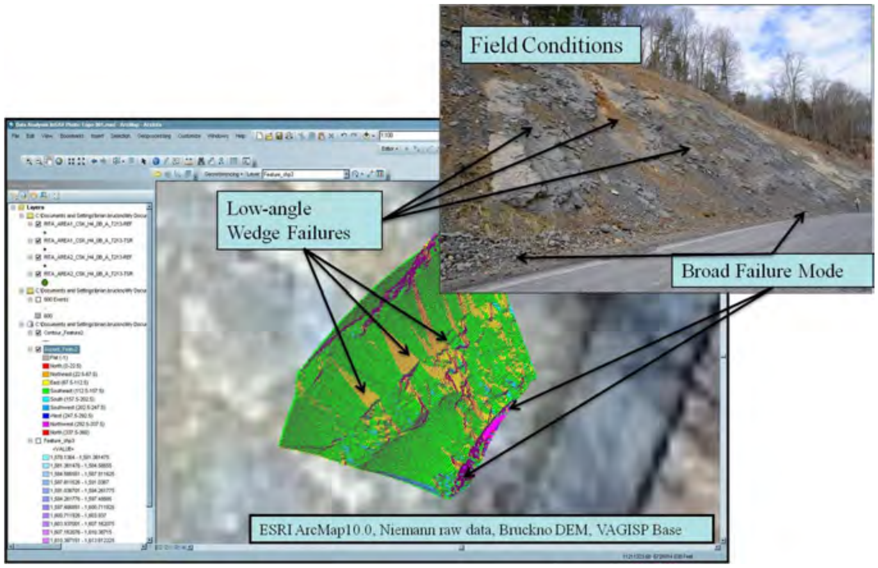

Figure 10 – RS-600-001001 Digital Photogrammetry Data

Figure 11 is an aggregate of the digital photogrammetry data and interpretation brought into an ArcMap® dataframe, and the stereonet represents of site kinematics.

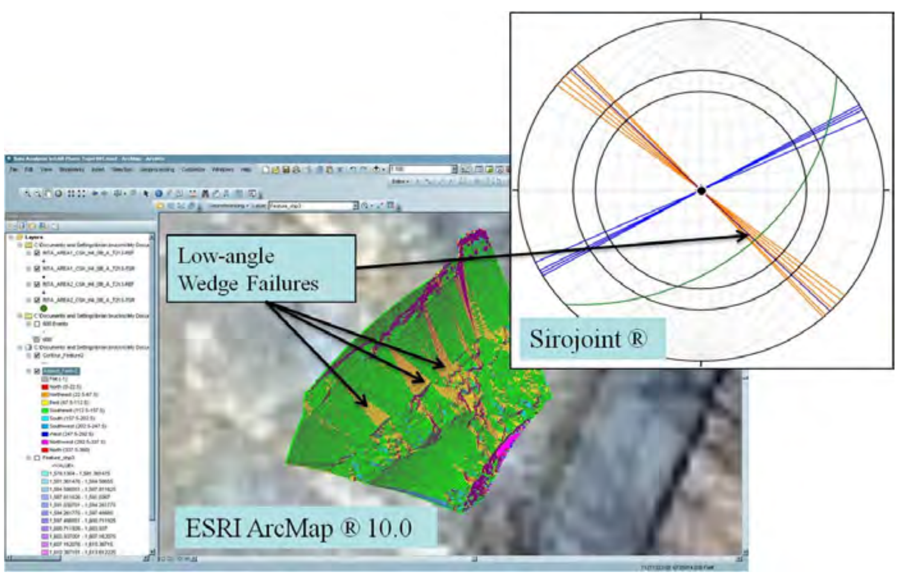

Figure 11 – RS-600-001 Digital Photogrammetry Data and Analysis

Both the digital photogrammetry and the GIS interpretations agree well with the field conditions: Sirojoint® reveals a systematic set of wedge failures formed by the intersection of moderately-dipping bedding and high-angle joints. The GIS surface aspect analysis reveals the wedge failures to be pervasive along the rock slope surface. The data yielded by the LiDAR consist of a set of point cloud data overlapping the digital photogrammetry data and yielded similar results and interpretations.

The InSAR data agrees with the field conditions as characterized by GIS and digital photogrammetry. Figure 12 illustrates data of selected InSAR scatterers falling on RS-0600-001.

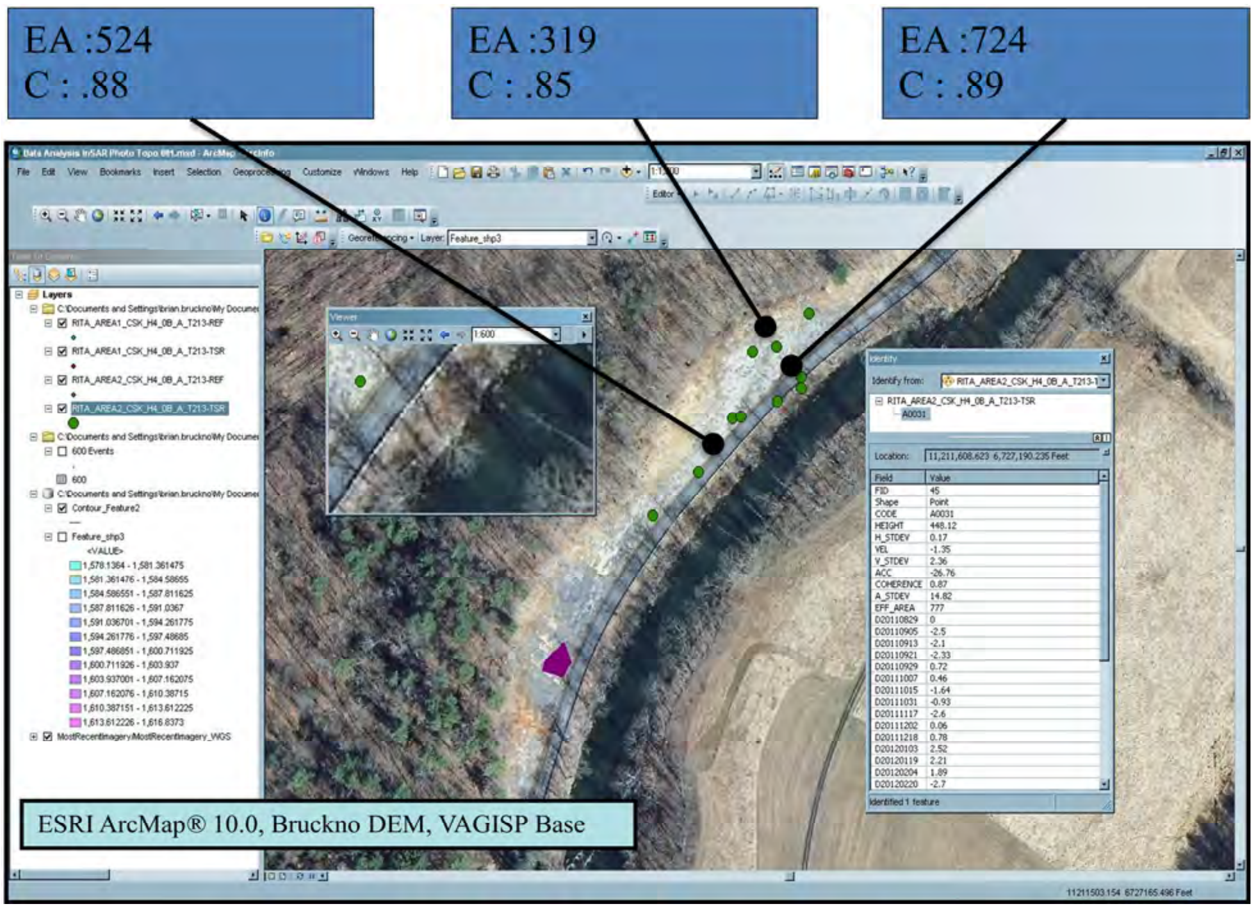

Figure 12 – RS-600-001 InSAR with EA (m^2) and C (dim.)

The scatterers falling on RS-0699-001 are all DS, i.e., they represent movement over a large area. This agrees well with the field observations and the surface analysis rendered by GIS and digital photogrammetry, in that the failure is occurring over the entire slope in small individual areas. Were the slope absolutely stable and undergoing no weathering whatsoever, there would have been no phase changes detected, and therefore a lack of data. Were the slope undergoing severe weathering, losing very large clasts (on the order of boulder-size) the individual scatterers would have lost coherence entirely. Furthermore, InSAR points A002Z and A003F, both DS, yielded vertical settlement of 0.6 and 0.7 in, respectively (i.e., rock face unloading), which agrees well with field observations of activity at this slope.

While only one rock slope within the AOI had characteristics conducive to analysis by InSAR, LiDAR, and digital photogrammetry, and neither digital photogrammetry nor GIS analysis yielded results in terms of quantifiable volumetric loss that the authors considered reliable, the general agreement between the observed behavior of the slope and the InSAR data suggests that the method may be useful for remote monitoring of slope activity and discrimination of rock slope hazards based on C and EA values.

InSAR Validation: Geotechnical Infrastructure

The AOI contains 408 bridges and 224 box culverts, 94 rail crossings, and 1 active municipal landfill, as well as an unknown number of rock buttresses and soil slopes. Each bridge and box culvert is associated with a location and quantifiable inspection data. While the bridges and box culverts are inspected on a frequency of no less than 24 months, rock buttresses and soil slopes are not inventoried, nor are they associated with any performance metrics or specifications, nor are they subject to an inspection program. Rock buttresses are considered to be an inherently reliable design and are considered to require no post-construction inspection.

A systematic evaluation of bridge sufficiency data and inspection reports with respect to InSAR data is underway as of the date of this article; for the purposes of the preliminary validation of in InSAR data, various InSAR points showing motion near or on infrastructure were selected for field inspection. Where possible, areas of two bridges in close proximity or sistered bridges of different ages, one with InSAR scatterers and the other lacking scatters, were chosen in order to minimize the potential for confirmation bias. InSAR scatterers were validated according the rubric in Table 2.

Table 2 – Selected InSAR Validation Rubric

Validation	Typical Validation Evidence	Validation Value
Absolute	Cracks, settlement, recent unvegetated scarps	1.0
Strong	Distortions or cracks, overgrown scarps	0.75
Weak	Repairs or cracks, geomorphology indicates activity	0.5
Possible	Near existing active region In correct terrain, presence of pinnacles	0.25
None	No or negative confirmation	-1.0

Figure 13 is an example of a field verification site. The location is a sistered bridge, with a modern structure to the right in the photograph, and an older structure to the left. The InSAR data includes scatterers indicating settlement at the older structure, but no scatterers located on the modern structure. This data is validated by field observations, which include evidence of damage and deterioration over the older, but not the modern structure.

Figure 12 – Field Validation Site

Table 3 contains a partial set of data relating selected infrastructure, InSAR points, and notes on validation or refutation of the InSAR data related to actual asset condition. Because bridge condition data is not public information, location data is not included

Table 3 – Selected InSAR Scatterer Data Related to Infrastructure Condition and Validation Type

Site ID	Validation Data	
	Validation Value	Validation Evidence
001SL	1.00	Distortions to rock buttress
002RA	0.75	Recent addition to farm waste pile
003RA	1.00	Quarry spoils pile
004RA	1.00	Active auto junkyard
005SH	1.00	Recently decommissioned landfill cell
006RA	0.75	Recent addition farm waste pile
007NC	-1.00	No confirmation
008SL	1.00	Bent trees, slope sloughing near creek, and settlement in drainage basin
009SL	1.00	Distortions to rock buttress
010SL	1.00	Distortions to rock buttress
011SL	1.00	Bent trees, slope sloughing near creek, and settlement in drainage basin
012SL	0.75	Noted wetlands at toe of slope
013SL	1.00	slope drainage pipe had broken
016SL	0.50	Noted spring/wetlands/drainage at toe of slope
017SL	1.00	Recent Burn Area
021SL	0.50	Noted shallow failure on slope
022SL	1.00	Noted leaning signal pole
023PV	0.50	Distortion and Cracking in Pavement
024BR	0.50	Distortion on Erosion and Scour Protection

Validation is ongoing; as of the date of this article, the overall validation value is 0.6, strongly suggesting a positive correlation between displacement activity identified by InSAR scatterers and distortion or damage to infrastructure.

One area of InSAR scatterer data was noted early in the investigation; this area corresponded to a rock buttress within the AOI. Figure 14 represents the motion of the scatterers located on the surface of the rock buttress.

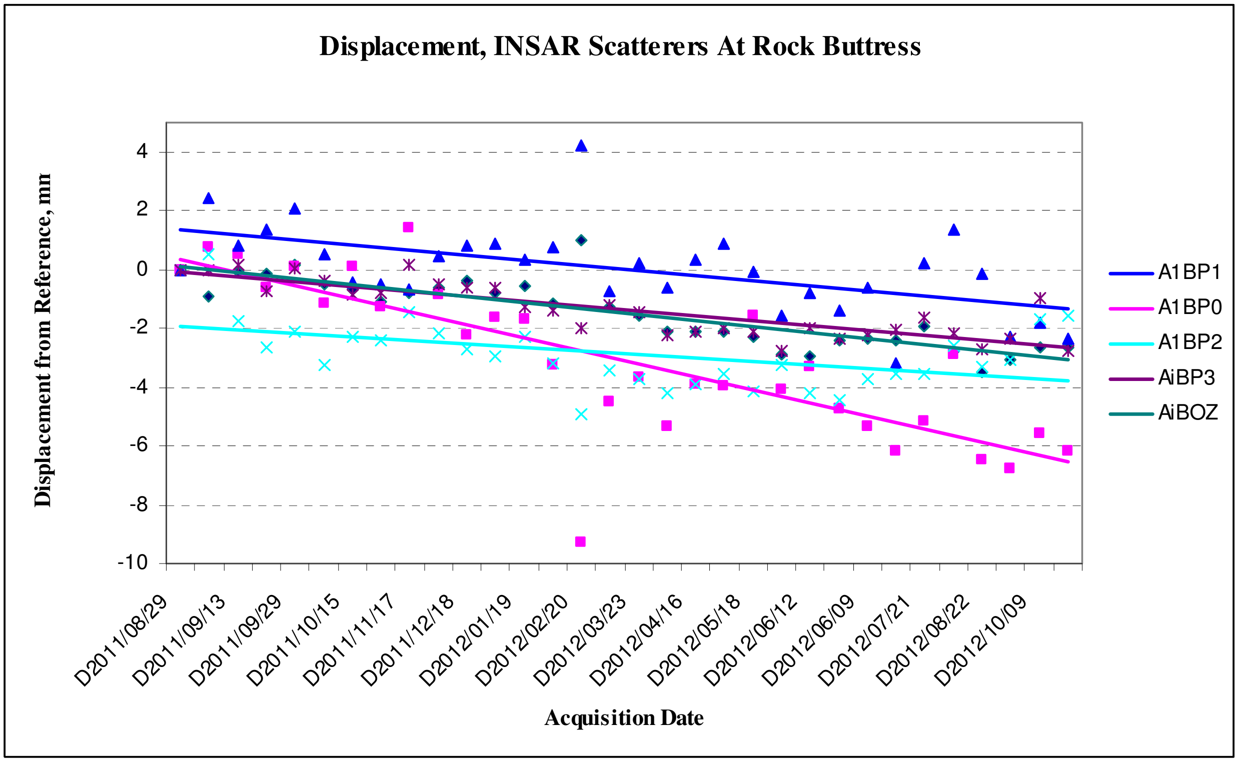

Figure 14 InSAR Scatterer Data at Rock Buttress

The AOI contains a number of rock buttresses. Because the locality represented in Figure 14 was the only rock buttress which demonstrated consistent negative-trending displacement, the authors decided to further investigate its behavior. Several site visits, as well as two episodes of digital photogrammetry data collection, were conducted. Figure 15 is an image of the digital photogrammetry data rendered by Sirovision® along with a site image.

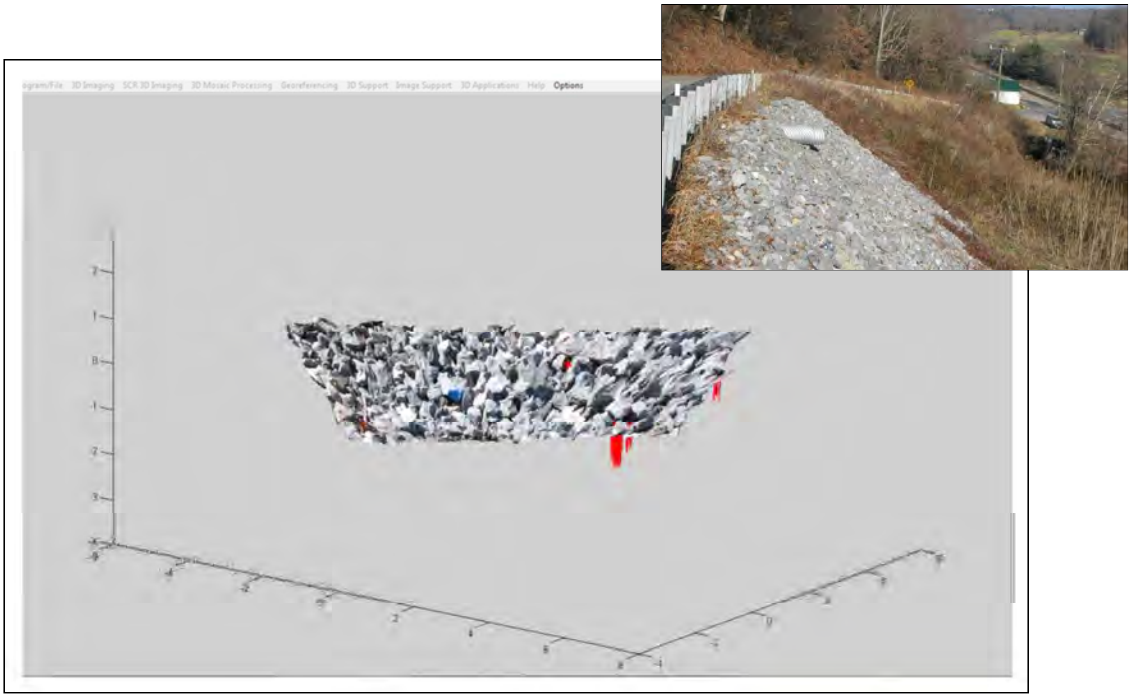

**Figure 15 - Sirovision®
Photogrammetric Image at Rock
Buttress**

The red lines show area of maximum calculated displacement at the rock buttress slope between September and November 2012. While the digital photogrammetry was able to image the rock buttress, the results were not deemed by the authors to be sufficiently reliable to create a time-series of movement along the slope; however, because of the minimal cost, ease of use, and compatibility of the dataset with other types of software, the authors consider digital photogrammetry to be an attractive method of rock buttress characterization for future research.

**Figure 16 – Site Conditions and
Deterioration at Rock Buttress**

Field instigations of the site suggested that a combination of internal settlement and blocked drainage is causing the surface of the rock buttress to distort, and may indicate future failure risk. While the InSAR signal cannot be used to quantify motion along the rock buttress, field investigations strongly suggest that the InSAR scatterer data did reveal previously-unidentified motion along the face of the rock buttress.

DISCUSSION

The authors evaluated the value of InSAR scatterer data applied to evaluation of geohazards and infrastructure condition. The authors determined that velocity measurements of InSAR scatterers were most strongly negative nearest to mapped sinkholes, whereas the overall average velocity of all scatterers in the karst-prone areas was slightly positive. While the AOI allowed analysis of only one rock slope by InSAR and ground-based methods, the coherence and effective area data yielded by the InSAR agreed with field observations and measurements made by digital photogrammetry and terrestrial LiDAR. Lastly, the InSAR scatterer data was positively correlated with field evidence of infrastructure damage or distortion on a range of geotechnical assets including soil slopes, bridges, pavement, and rail crossings. Additionally, a rock buttress displaying motion was identified by InSAR scatterers, and degraded performance of the rock buttress face was confirmed by field investigation.

Validation data collection is ongoing as of the date of this article. Next steps include a systematic evaluation of geotechnical assets which lack InSAR scatterer data in order to evaluate the potential of false negatives, and inclusion of the scatterer data in the bridge inspection program. This may prove to be the best implementation of the InSAR data collection, in that it may reveal damage or distress to bridges between scheduled inspections, and may allow better allocation of staff hours for bridge inspections and include an element of performance-based bridge inspection. Plans are underway to include condition data derived from the InSAR into a new, GIS-based geotechnical asset management system, which will be delivered to field inspection personnel via handheld devices.

Major challenges to the full implementation of InSAR data collection remain. Among the greatest of the challenges is the loss of coherence in areas of sudden ground or infrastructure motion. New methods of identifying scatterers which have coherence for a period of time and then suddenly lose coherence, suggesting a break in the rate-of-change of the motion, are being developed. Regardless of the challenges, the authors view the application of InSAR to remote detection and early warning methods for geohazards and infrastructure failures as highly promising. The InSAR data collection and interpretation lends itself to wide-scale scanning and monitoring at the transportation-corridor level, particularly in areas of very dense transportation infrastructure such as roads, bridges, rail lines, and embankments. Wide implementation of InSAR monitoring may yield more comprehensive and integrative asset management and inspection programs, and, by revealing early signs of failure on critical assets, may be a source of considerable return on investment and mitigation of liability.

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Use of Multi-Electrode Electrical Resistivity to Define the Depth of the Landslide and the use of Isolated Tie-Back Plates to Stabilize the Landslide in Steep Terrain

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Disclaimer

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ABSTRACT

An existing landslide located at the southeast corner of the Telluride Regional Airport has represented an on-going liability for the Airport, the Federal Aviation Administration and the Colorado Department of Transportation. A catastrophic failure of this landslide occurring in a manner similar to that which occurred at the Airport in 1987 has posed an on-going threat to closing Colorado State Highway 145 which is located below the slide area. The existing landslide was characterized as a series of multiple failed block areas located downhill of the airport runway that have occurred in severely weathered Mancos Shale.

A total of 12 alternatives were evaluated to mitigate the landslide, including the preferred alternatives of either partial or total landslide removal. However, prior development left essentially no place on the airport property that would allow for the disposal of the landslide debris, and the closest off-site disposal area was approximately 40 miles from the site. As a result, in-situ stabilization of the landslide, including a primary system of isolated tie-back anchor plates with strand anchors, and a secondary system of high strength steel mesh and intermediate anchors was selected for the ultimate design to stabilize the landslide in place.

For design purposes, geotechnical characterization of the slide area was accomplished through geological mapping, conventional borehole exploration and geophysics using multi-electrode resistivity (MER). The paper discusses the benefits of using MER and isolated tie-back anchor plates, particularly after discovery of survey error required redesign of the entire stabilization system half way through the project.

INTRODUCTION

Telluride Colorado is located at the toe of the Rocky Mountains on the western slope of Colorado. The Telluride Regional Airport is located on Deep Creek Mesa about 5 miles west of the town of Telluride.

The landslide is located at the southeast corner of the airport just below an abandoned portion of the old quarry access road on a south facing slope. The old quarry access road is located below the elevation of the runway and outside the elk fence that surrounds the air traffic area of the airport property. Since the reconstruction of the runway, the new access road to the quarry was rerouted to stable ground north of the old quarry access road location and off of the head scarp of the landslide (See Fig. 1).

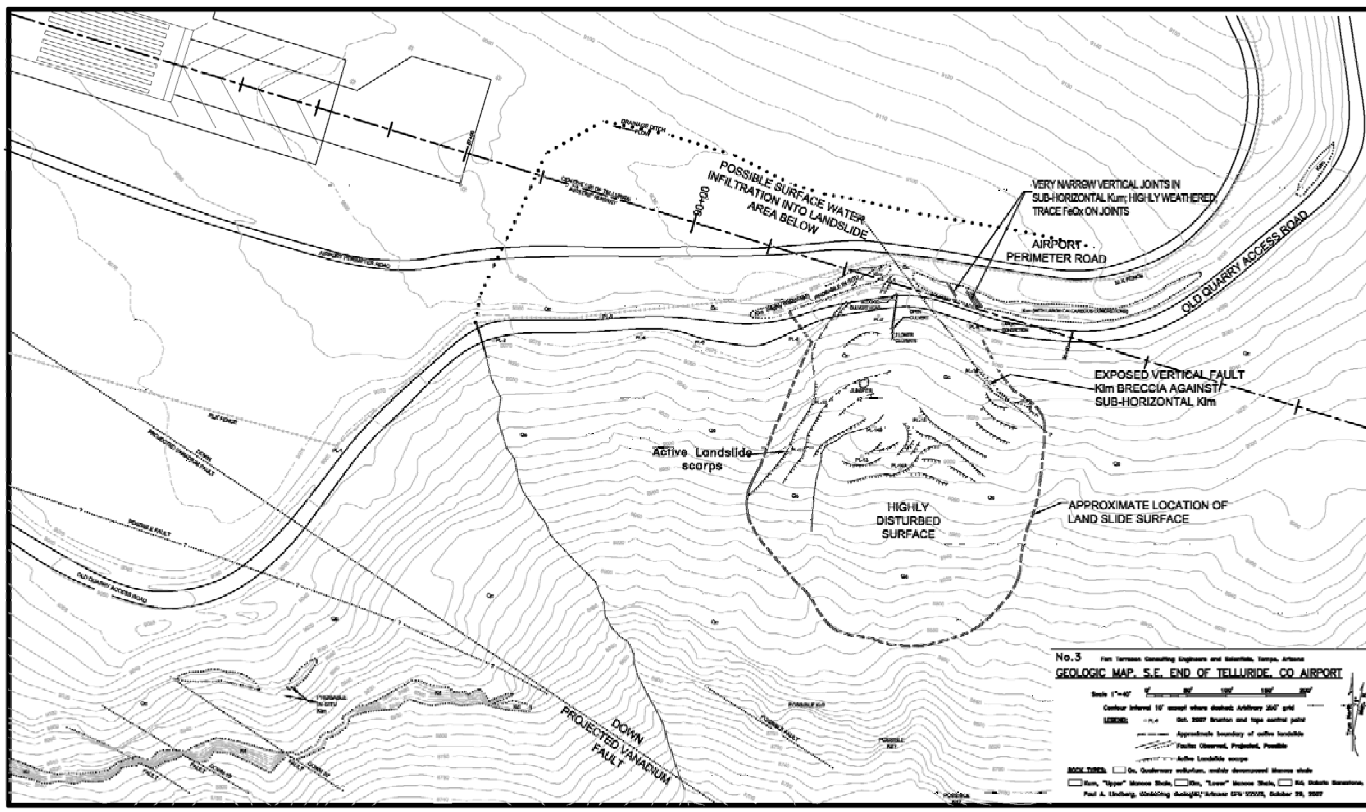

Figure 1 – Plan View of East End of Runway including Landslide Area, and the Old Quarry Access Road

The landslide has been in an active state of failure since at least 2000 when clear evidence of movement was observed by the presence of tension cracks in the surface of the abandoned portion of the quarry access road (Fig. 2).

Fig. 2 - Tension Crack at Head Scarp.

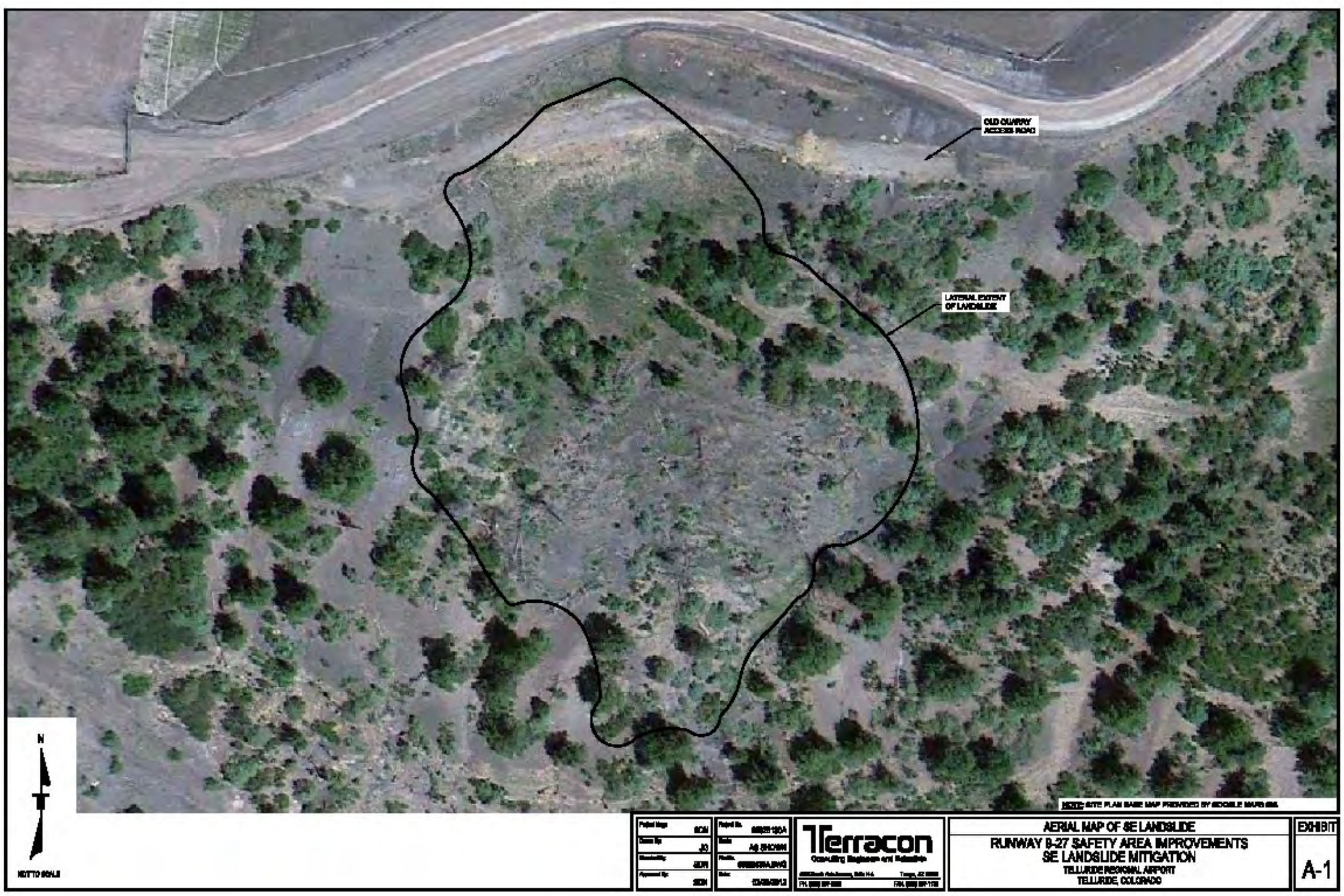

Figure 3 - Aerial View of Landslide

The landslide in plan area is approximately 460 feet north to south and 360 feet east to west (see Fig. 3). The surface of the landslide slopes at about 3H:1V (horizontal:vertical) to the south.

GEOLOGIC SETTING

Deep Creek Mesa and the immediate vicinity are underlain by sedimentary bedrock consisting of Mancos Shale and Dakota Sandstone. These geologic formations were deposited in a marine environment during the late Cretaceous Period. The younger Mancos Shale formation overlies the Dakota Sandstone.

The Dakota Sandstone can be generally described as light in color with frequent thin gray to black carbonaceous shale layers occurring throughout the formation. Occasional coal and lignite beds occur in the formation at some locations, however, none have been observed in the explorations or open cuts performed for the Airport on Deep Creek Mesa. The shale lenses are frequently highly plastic and have expansive potential.

The Mancos Shale can generally be described as a thinly bedded, light to dark gray marine shale. Within the formation thinly bedded fine grained sandstone and limestone lenses may also be encountered. Some portions of the Mancos Shale are bentonitic, and potentially highly expansive. The majority of the shale has only moderate plasticity characteristics. Where undisturbed by landslide movement, the Mancos Shale is highly weathered to a depth of about 5 to 8 feet. In the area of the landslide where there are deep crevices due to landslide movement, the Mancos Shale is weathered to depths on the order of 40 to 50 feet (see Fig. 8 for typical cross section through landslide).

The major geologic structural feature on Deep Creek Mesa is the Vanadium Fault which crosses the mesa in the area of Deep Creek and generally trends across the site from east to west. (See Fig. 1) Published geologic maps of the area by the U.S. Geological Survey (USGS) ([1]Bush, et al,), indicate that the portion of Deep Creek Mesa south of the Vanadium Fault has been upthrust approximately 80 feet relative to the north side of the fault. According to the Colorado Geological Survey (CGS) ([2]Stover et. al.) the up-thrown side of the fault is up approximately 350 feet relative to the down-thrown side.

An unidentified fault was mapped in the immediate area of the bedrock exposures uphill of the quarry access road with an average strike orientation of N 37^0 W. This fault defines the eastern edge of the landslide. See Figure 1 for the location of the fault in plan view.

Deep Creek is the major drainage feature in the area. Deep Creek crosses the central portion of the Airport runway and flows to the north.

LANDSLIDE CHARACTERIZATION

The landslide extremity was easily identified and mapped in the field. In general, the upper portion of the landslide was depressed in relation to the surrounding native intact ground surface (see previous photograph), and the lower portion was raised above the surrounding ground surface. The toe of the landslide daylights at the interface of the Mancos Shale and Dakota Sandstone at the head of a ravine (see Fig. 4).

Fig. 4 - Toe of Landslide

The margins of the landslide were mapped by taking GPS coordinates at 42 locations using a handheld Trimble XH with horizontal accuracy on the order of 1 foot. This mapping exercise became a key and vital part of the engineering analyses and design as it determined the lateral extent of the landslide area.

One boring had been drilled near the center of the landslide mass using remote access equipment. The information from this boring provided only one discreet point for characterizing the subsurface conditions within the landslide.

Based on previous explorations throughout the airport property, and confirmed by the one boring performed in the middle of the landslide, the subsurface conditions were known to consist of weathered Mancos Shale, overlying less weathered Mancos Shale. Dakota Sandstone was encountered below the less weathered Mancos Shale. However, because there was only one boring, and additional borings would be expensive and provide only discreet points of information, multiple electrode resistivity (MER) methods were selected to further explore the subsurface conditions, and to specifically determine the depth of the landslide material.

To further determine the vertical extent of the landslide material, the assumption was made that the landslide materials would have a higher moisture content and corresponding lower electrical resistivity compared with the underlying undisturbed Mancos Shale. This difference could be measured and would present itself using MER methods of exploration. Seven MER traverses were performed across the slide area in an east to-west direction. The MER results were calibrated with the data obtained from the test boring to adjust the MER lines completed at other locations. The GIS mapping was also used to determine the lateral extent of the landslide and to assist in developing the cross sections.

On the basis of the MER results, a three-dimensional model of the landslide mass was developed for design purposes. The results of a typical MER line and the interpreted vertical extent of the landslide along that line are shown on Figure 5.

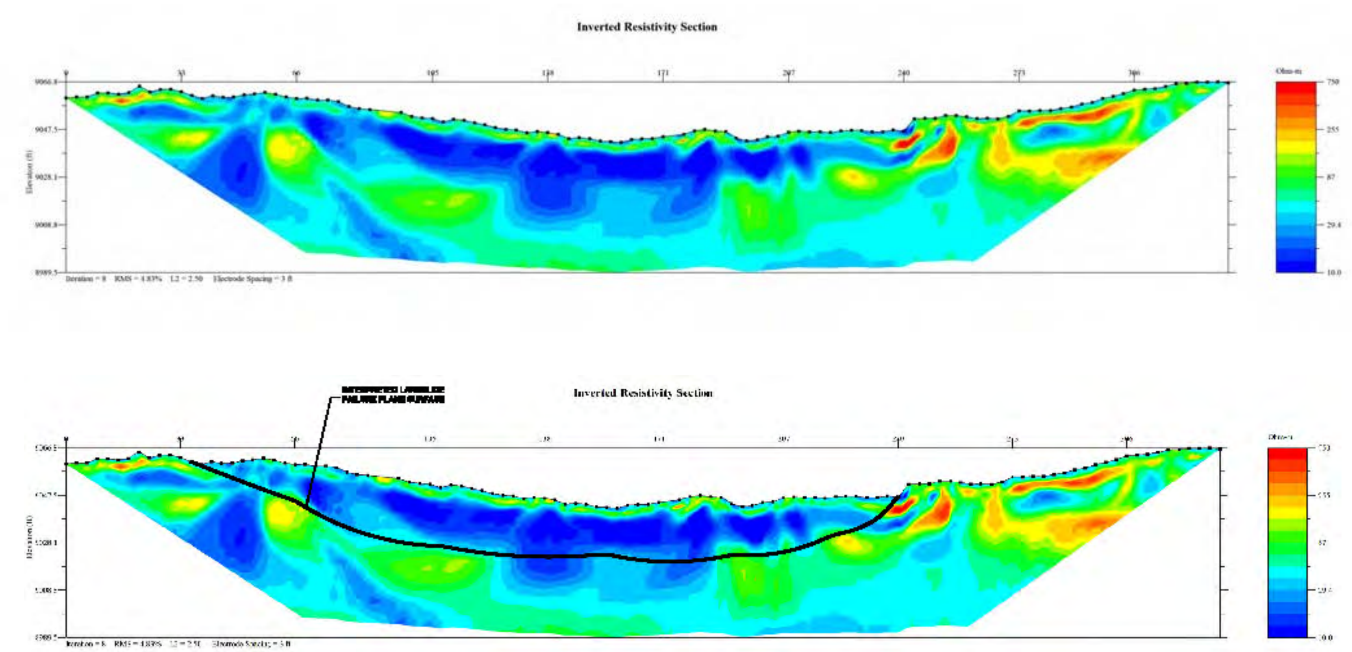

Fig. 5 – TOP - Typical Cross Section along an MER survey line BOTTOM – Typical Cross Section along an MER survey line with interpreted depth of landslide material shown in heavy black line.

LANDSLIDE MITIGATION ALTERNATIVES

Figure 6 presents a summary matrix of the mitigation alternatives that were considered during the course of completing the landslide study. Engineering evaluation of each of the alternatives and sub-options were categorized with regard to:

- Reliability
- Cost
- Constructability
- Maintenance/Monitoring
- Aesthetic Impact
- Inter-Agency Issues (CDOT)
- Further Considerations

Based on the initial engineering evaluation, total or partial landslide removal was determined as the preferred option for landslide mitigation. However, no suitable disposal site could be found in close proximity to, or on the airport property. The closest disposal site was determined to be at a location approximately 40 miles from the airport. Consequently, the preferred alternative to remove the landslide was eliminated as an option due to the anticipated deterioration and cost to rehabilitate local and state highways and the potential political consequences of trucking materials through the residential subdivision which surrounds the airport. As a result, an alternative that would stabilize the landslide in-place was considered as the next best alternative. This alternative consisted of the installation of tiebacks and anchors and was further evaluated in the engineering analyses.

Telluride Regional Airport
Southeast Landslide Stabilization Decision Matrix

	No Removal	Partial Removal	Total Removal	Retaining Walls	Drilled Shafts	Driven Piles	Tie Back Anchors	Reinforced Fills	Key Trenches	Buttress Fill	Debris Flow Control	Property Protection Structure
Reliability	Low	Moderate	High	Moderate	Moderate	Low	Low	Low	Moderate	Moderate	Low	High
Cost	Low	Moderate	High	High	High	High	High	High	Moderate	Low	High	High
Constructability	SF	SF	MD	MD	D	D	D	D	D	D	SF	MD
Maintenance/Monitoring	Moderate	L/M	Low	Moderate	Low	Low	Low	Moderate	Low	Low	M/H	High
Aesthetic Impact	Low	Moderate	Moderate	High	Moderate	Moderate	Moderate	Moderate	Low	Moderate	High	High
Inter-Agency Issues (CDOT)	High	Moderate	Low	Moderate	Moderate	High	High	High	Moderate	Moderate	High	High
Further Consideration	Yes[1]	Yes	Yes	No	No	No	No	No	No	No	No	No

Notes:
SF=Straight Forward Construction
MD=Moderate Difficulty of Construction
D=Difficult Construction
L/M=Low to Moderate Maintenance/Monitoring Issues
M/H=Moderate to High Maintenance/Monitoring Issues
[1]This Alternative will be further considered because of the lowest project cost alternative
Indicates a positive result (green) Indicates a cautionary result (yellow) Indicates a negative result (red)

Fig. 6 - Decision Matrix

The proposed mitigation alternative that was evaluated and used for design of the project consisted of a mesh slope stabilization system in combination with ground anchors and anchor tieback plates. This mitigation alternative eliminated the need to remove any of the landslide debris from the airport property

STABILITY ANALYSES/SYSTEMS DESIGN

The ground anchor system with concrete anchor tieback plates was designed to increase stability of the bulk of existing landslide debris. The mesh slope stabilization system was designed and included to increase stability against shallow and localized failures in the surface located between the rows of the anchor tieback plates and at locations outside of the area of the ground anchors where landslide debris is on the order of 10 feet or less in depth.

To obtain an order of magnitude for the anchor sizes and spacing, a simple plane failure analyses was performed as shown in the following equation and Figure 7:

$$F = \frac{cA + (W cos\psi + T cos\theta) Tan\phi}{W sin\psi - T sin\theta}$$

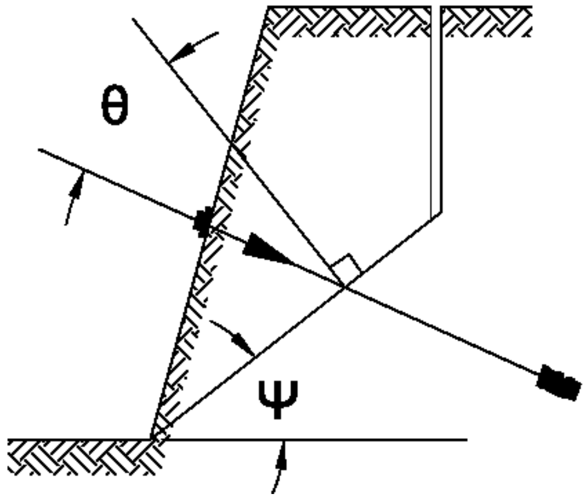

Fig. 7 - Reinforced Slope Schematic

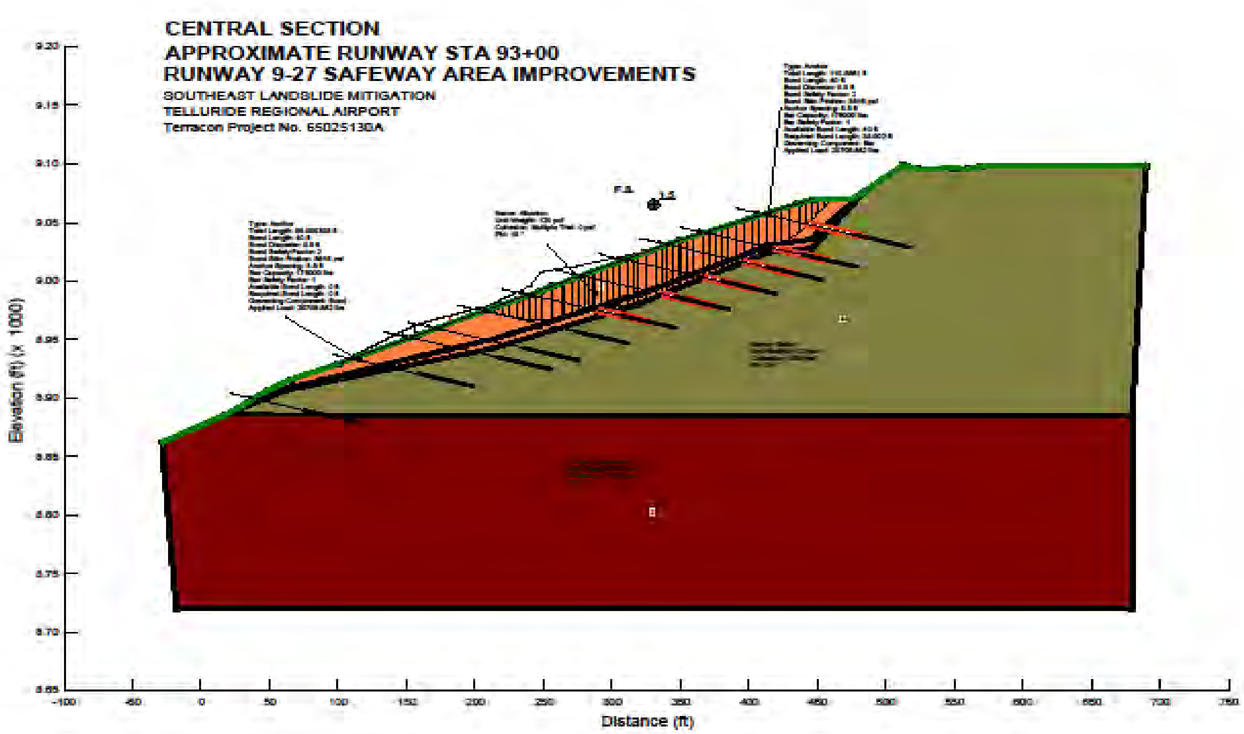

Fig. 8 – Typical Cross Section through middle of landslide

The spacing determined from this approach was adjusted during the slope stability analyses using Slope/W slope stability software and the general limit equilibrium method (GLE) of analyses. Multiple cross-sections were modeled across the location of the landslide area (see Fig. 8 for typical cross section) in order to determine the required number and depth of anchors, tieback loading and locations.

The target factor of safety was a minimum of 1.5 against global instability for the re-graded landslide mass. A factor of safety of 2 was used to determine the allowable load of anchors in the bonded zone within the Mancos Shale.

The design of the mesh slope stabilization system was based on the computer program RUVOLUM developed by GEOBRUGG of Romanshorn, Switzerland. In general, this program models shallow infinite and localized slope type failures. Stability is increased when the mesh slope stabilization system and intermediate nails/anchors are applied in the model.

Shear strength parameters used for the stability analyses of existing and conceptual slope configurations were based upon laboratory data developed during previous explorations, laboratory test data previously developed at the location of this project, correlation to field and laboratory test data, back-calculation of shear strength along failure planes beneath the landslide debris, experience with similar soils/bedrock and end use conditions. The shear strength data used in the engineering analyses is summarized in Table 1.

TABLE 1

Material Type	**Drained Shear Strength Parameters**		**Ultimate Grout to Ground Bond Stress (psi)[3]**
	c' (psf)	**ϕ'**	
Landslide Debris/Alluvium	0	19	15[1]
Compacted Embankment Fill	150	24	N/A
Mancos Shale	1,000	24	47[2]
Dakota Sandstone	10,000	0	N/A

[1] Bond stress in the alluvium only used for the design of the intermediate nails/anchors for the mesh stabilization system. The portion of the ground anchors in the alluvial soils is part of the unbonded anchor length.

[2] The bonded portion of the ground anchors will be developed entirely in the underlying Mancos Shale. None of the ground anchors are expected to encounter the lower Dakota Sandstone Formation.

3 Ultimate bond stress values will be verified by conducting sacrificial verification tests during construction in accordance with the G-801 specifications.

The stability analyses were completed for static conditions using drained shear strength parameters. Undrained shear strength was not considered in the analyses since the existing landslide was in an active state of failure with residual shear strength being developed on the failure plane.

In regard to groundwater, the subsurface conditions were modeled for the completed condition without the inclusion of groundwater in most all of the models. This design assumption is predicated on eliminating perched water within the landslide debris and on top of the underlying shale with the installation of horizontal drains and by reducing infiltration of surface water into the debris by re-grading and placing compacted embankment fill at the surface. However to verify adequate stability of the anchor tieback system, two critical cross sections were modeled.

Each cross section was modeled geometrically on the basis of the proposed grading of the embankment and based on the depth of the failure surface determined from the geotechnical and geologic studies conducted at the location of the project. To provide for additional conservatism in the design, the depth to the contact (i.e. failure surface) between the landslide debris and the underlying Mancos Shale was increased by 25% over that determined from the MER surveys. Three cases were analyzed, including:

***Case I*:** End of Construction associated with drained shear strength of embankment and foundation material failures along a pre-determined failure plane (at an increased depth of 25%);

***Case II*:** End of Construction associated with drained shear strength of embankment and foundation material failures incorporating underlying shale and anchors (generally to the tail end of the anchor system); and,

***Case III*:** End of Construction associated with drained shear strength of embankment and foundation materials and perched groundwater to five (5) feet above the contact between the landslide debris and the underling shale (note the contact was considered at a failure plane increased by a depth of 25% as previously discussed).

The results of the stability analyses, expressed in terms of Factor of Safety for the most critical failure surfaces, are summarized in Table 2.

The design of the mesh slope stabilization system considered the potential of developing shallow infinite slope type failures along the surface of the completed embankment and the potential of localized failures between the locations of anchor tie back rows and at locations of relatively thin sections of landslide debris (i.e. less than 10 feet). Infinite slope type failures were modeled at an approximate depth of 10 feet from the surface of the completed embankment between anchor rows. Outside of the anchor tieback system, the mesh slope stabilization system was designed on the basis of stabilizing landslide debris to a depth of approximately 10 feet or less. Drained shear strength parameters, as previously outlined, were used for these analyses.

The anchor tieback system design and the stability evaluations have been based on the use of strand anchors with allowable design loading of either 141 or 211 kips.

TABLE 2 – SUMMARY OF FACTORS OF SAFETY					
Runway Station	**Project Station**	**Minimum Factor of Safety**			
		Case I	**Case II**	**Case III**	**Other Cases[1]**
91+45	3+25	1.6	1.9	--	
91+70	3+50	1.5	1.5	--	
92+20	4+00	1.5	--	1.4	1.5
92+75	4+50	1.6	1.7	--	
93+00	4+75	1.5	1.6	--	
93+50	5+25	1.6	--	1.4	1.5
94+00	5+75	1.6	1.5	--	

[1]Other cases reported here represent failure surfaces contained within the landslide debris but not along the entire extent of the failure surface.

Pull out resistance of the anchors was developed in the underlying intact shale beneath the landslide debris. An ultimate bond stress of 47 psi and allowable bond stress of 23.5 psi were considered in the analysis of the geotechnical capacity of each anchor and the grout to ground resistance to be developed along the sides of each drill hole. The selection of the design bond stress was made based on the results of shear strength testing of the shale conducted on previous studies at the airport. The bond stress was confirmed during construction by verification testing on two sacrificial anchors.

The verification tests consisted of two separate anchors, each drilled to a depth of 30 feet and 6 inches in diameter. A maximum number of 7 anchor strands was used in each anchor and was limited due to drill tooling. The bonded lengths were 15 feet and 20 feet. For the bonded length of 15 feet the anchor was loaded to about 120 kips which is 150% of the allowable bond stress. The 20 foot bond length anchor was loaded to 160 kips. Both verification test anchors showed nearly linear displacement vs. load graphs and did not have any creep during the 10 minute hold interval. After the last cycle, each anchor was incrementally loaded to about 320 kips which is slightly less than the maximum jacking load of 328 kips in an attempt to determine the ultimate bond stress of the Mancos Shale. The graphs of the displacement vs. load followed the same trend as obtained for the verification testing indicating the bond stress in the Mancos Shale was higher than the 47 psi that had been estimated for design purposes.

Continuous engineering inspection was conducted on the tendons during the installation of the anchors to verify that the bonded length was achieved in the proper bearing materials.

The concrete anchor tieback plates were designed for the two loading conditions of the anchors that were used on the project (i.e. either 141 or 211 kips). The plates were designed based on the procedures in ACI 318 and ASCE 7. Each plate was analyzed for bending, punching and shear.

STABILITY ANALYSES/SYSTEMS DESIGN - REDESIGN

At about one-third of the way through the installation of the tie-back anchors, an error in the surface topography was identified in the field, and confirmed by the project surveyor.

At the time the error was discovered, all of the anchors had been fabricated based on the original design, and most had already been delivered to the site. Based on these fixed lengths and capacities, an attempt was made to use the existing anchors in a redesign of the system. Since the capacity and length of each tendon anchor was fixed, the only modification to the stability analyses was the east-west and north-south spacing of the anchors.

The contractor (DBM) by this time in the project had two drill rigs installing anchors and was routinely drilling 500 to 700 feet per day with each rig. Consequently, about 6 to 9 anchors were being installed each day at this time in the project. Working with the contractor on a frequent basis to determine their planned construction and anchor bench sequencing, interim drawings were developed during the redesign process in order to avoid a long delay in the project schedule.

Based on the close working relationship developed with the contractor after the survey error was discovered, interim plans were provided within four days for the spacing and location of the next row of anchors they planned to install. Approximately seven days later, a full re-design of the stabilization system for the project had been completed.

The survey error resulted in a larger amount of material that had to be stabilized in the lower portion of the landslide mass than that which was included in the original engineering design. This resulted in the need to increase the number of anchors at lower elevations on the landslide mass and a reduction in the number of anchors at higher elevations on the landslide. As a result of the redesign effort, additional stability was achieved by adding a new row of anchors below the previously lowest row of anchors, and by increasing the spacing between anchors higher on the landslide.

The flexibility of the tie-back anchor system allowed for rearrangement of the system in order to keep anchor changes to a minimum and reduce the requirement for additional anchors and tie-back plates.

In less than 11 days from delivery of the corrected topographic survey, a complete set of revised plans for the anchor system was delivered. The re-design of the mesh slope stabilization was completed 10 days later.

At the conclusion of the redesign, only six additional anchors were required, all of which were 141 kips and only 60 feet in length (some of the shortest on the entire project).

Fig. 9 – View of Anchor Placement

CONCLUSIONS

- The use of geophysical surveys was instrumental in obtaining the three dimensional geometry of the landslide mass.
- The use of individual anchor plates was a key factor in flexibility of construction of the project.
- Off-site fabrication avoided the placement of concrete plates on the landslide slope and increased the overall safety of the project.
- The individual anchors allowed redesign to be completed in a relatively easy manner after discovery of the topographic survey error.

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“Large”-Scale Seismic Reflection for Infrastructure Projects

Not just for Oil and Gas Anymore

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ABSTRACT

The seismic reflection method is one of best established geophysical techniques taught in introductory geophysical courses. A common misconception of the method is that it is solely a tool for mapping deep geologic structure and stratigraphy. This perception is unfortunately associated with the level of cost and scale required for petroleum exploration.

Modern engineering-scale seismographs (12-48 recording channels) have been used to a varying degree of success in mapping shallow geology with seismic methods. Recent advancements developed for the petroleum industry in instrumentation and data acquisition are being co-opted by the shallow geophysics community with tremendous success. Wireless sensors, very large seismic sources, and professional-level data processing services are now being applied beyond the oil patch and incorporated into small engineering-scale projects.

In this paper, we will show several examples where the utilization of hundreds of recording channels was capable of providing high-resolution geophysical data for a fraction of the exploration costs required only 10 years ago. Project examples include identifying karst features, mapped and unmapped fault structures, and general geologic structure. These examples are completed, ground-truthed engineering projects. Additionally, we present one example where the seismic reflection method was only marginally successful at achieving project goals, as well as a discussion about the drawbacks and limitation of the method.

Finally, as an industry we can safely state that seismic reflection surveys are no longer "just for the big boys" and can provide added benefit to the shallow engineering community.

INTRODUCTION

Geophysical surveying is becoming increasingly commonplace as a means of supporting environmental, geotechnical, and exploration projects. Not surprisingly, most geophysical methods can trace their roots and research to petroleum and mineral exploration. However, the availability of smaller, simplified instrumentation has accompanied a boom in client education with a suite of applications moving beyond the oilfield and into the realm of the civil engineer.

Seismic methods are identified as one of the most popular geophysical tools utilized by transportation geologists and engineers (*1*). The seismic refraction method is used to map depth to bedrock and the water table, as well as measure compressional-wave velocity (Vp) of unsaturated sediments and rock (*2*) to determine bedrock rippability. Seismic surface waves are used in 1D to model the shear-wave velocity (Vs) profile of soils to aid in seismic site class designation and in 2D to identify fractured and weak zones in rock (*3*). Seismic cross-hole and down-hole measurements provide in-situ measurements of Vp and Vs allowing the calculation of elastic moduli for critical structures.

However, the best-researched seismic technique is the seismic reflection method. Arguably the most complicated seismic method (in terms of field acquisition and certainly in terms of data processing), this method is traditionally one of the first taught to geologists and geophysicists attending an exploration-steered curriculum. The seismic reflection method benefits from 100 years of continuous, well-funded research. With respect to research and development of the technique, engineering geophysicists have been able to "ride the coattails" of the petroleum industry and take advantage of new instrumentation, new field recording techniques, and advancements in data processing and geologic visualization. The same equipment, processing resources, and interpretational methods of the petroleum industry are today being used on projects where the depth of interest is measured in hundreds of feet, not tens of kilometers.

We present three cases where high-resolution seismic reflection data was used to support engineering projects. Data was collected with small field crews over short time spans, at prices associated with traditional shallow geophysical surveys. We also present one case where site conditions and geologic issues provided problems for the seismic reflection method and serve to highlight the complexities of this geophysical technique.

Reflection Data Processing and Interpretation

It is important to note that the software and methods used in seismic reflection data processing are not common or typical to the engineering geophysics community. In order to responsibly carry out seismic reflection data processing and interpretation, Zonge International regularly partners with Excel Geophysical (processing) and Summit Geoscience (interpretation). We have partnered with these organizations for two decades on federal, state, and private projects. We believe an integrated team-based methodology for seismic reflection is the most responsible approach to solving these problems.

Though small-scale reflection processing software does exist and has been used by the engineering geophysics community with some degree of success, it cannot match the capabilities of a large scale seismic reflection processing regime. Our team's approach to seismic data processing includes dedicated experience with the ProMAX® interactive seismic data system. ProMAX®, developed by the Landmark Graphics Corporation, is a specialized family of dedicated reflection processing software commonly used for geophysical exploration. The processing routine used to handle shallow reflection data is similar to that used for high-resolution 2D data processing within the deep-basin community. Shot records are converted to 2D binned common-depth-point stacks (CDP) and follow an iterative processing flow of approximately 20 to 30 steps (more or less depending on the dataset).

The interpretation of seismic reflection data is as specialized as its processing and relies on a combination of geological as well as geophysical knowledge. Proper interpretation generally involves the use of specialty workstations with advanced visualization capabilities not common within the engineering geophysics community. Summit Geoscience utilizes a Seismic Micro-Technology KINGDOM seismic interpretation workstation to generate cross-sections as distance vs. two-way travel time plots. Kingdom software allows for available geologic or borehole information to be integrated into the seismic reflection data to provide a more comprehensive model of the area of interest. Our partnership with Summit Geoscience and Excel Geophysical has allowed Zonge International to bring the resources, research, and knowledgebase of the petroleum industry to the engineering community.

CASE HISTORIES

RTC Southeast Connector – Reno, Nevada

Project Outline

The RTC Southeast Connector is an important 5.5 mile long highway investment in the Truckee Meadows region that addresses long-term transportation needs and improves the mobility of people, goods and services throughout northern Nevada (*4*). Geologic maps of the area (*5*) indicated a projected fault passed near the project area and came within 80 feet of the proposed RTC alignment east of Reno, NV. To aid in the design of the Connector, Zonge International and its partners Excel Geophysical Services and Explortech LLC performed a high resolution 2D seismic reflection survey to delineate faulting along the proposed RTC Southeast Connector route. As this is within the basin and range province, the primary geologic units were quaternary valley-fill sediments within a relatively flat basin. The 2D seismic survey produced good seismic images that extended to over two thousand feet in depth and readily allowed for consistent stratigraphic and structural interpretations. The resulting interpretation identified that the projected fault was not found to be in the mapped position and multiple other faults were identified and located to the west of the mapped fault location.

Survey Setup

Zonge International acquired all seismic reflection data for this survey. The survey was completed using a 15-foot source and receiver (S/R) interval allowing for maximum achievable fold along the majority of each line. Data were obtained with a Wireless Seismic RT-1000 system capable of recording over 400 channels simultaneously. Each receiver station used a single channel Wireless Relay Unit (WRU) data receiver/transmitter connected to a single geophone. A DigiPulse 450 accelerated weight drop mounted on a Kubota ATV served as the seismic source. A wireless trigger was used to activate the seismic system. Source points were located on the half station, mid-way between each receiver. Survey control for source points, receiver locations, and other key site features were acquired with an RTK (real-time kinematic) GPS system. Fieldwork was completed in 2 field days using a 6-person field crew including a source operator.

Interpretation and Results

Prior to picking stratigraphic horizons, several faults were observed and interpreted along each seismic line. These faults are west dipping normal faults (extensional features). Correlating the faults between each line is difficult with 2D data and has some degree of uncertainty. Without additional geologic data, or, even better, 3D seismic data, there can be different possible orientations for these faults.

After interpreting the faults, two seismic events were identified. These two seismic horizons are more than 500 feet below the surface (using the assumed velocities) and appear to correlate from line to line. Currently no well or deep borehole is available to identify these events. Deep well bores with sonic logs or velocity surveys would allow for better determination of the valley-fill sediment velocity regime.

After examining the two sedimentary horizons, it can be shown that the four faults identified on Line 2 are less recent than the three faults identified on Line 1. This is because the faults on Line 2 do not appear to cross-cut the upper sedimentary horizon, only crossing the lower one, whereas the three faults on the more southern Line 1 clearly crosscut both horizons. Because of the different ages of the offsets from line to line it is somewhat uncertain if the faults can be correlated. If they are the same faults they would have to be near-vertical strike-slip faults with no recent vertical displacement on the northern line while having some offset on the southern line. The two seismic reflection profiles showing the final interpretation with identified faulting is shown in Figure 1. The two profiles show locations of the interpreted faults as well as the location of the fault mapped by Ramelli and Henry (2010). Evident within the profiles is the cross-cutting nature of the faults along Line 1 that is not evident within Line 2.

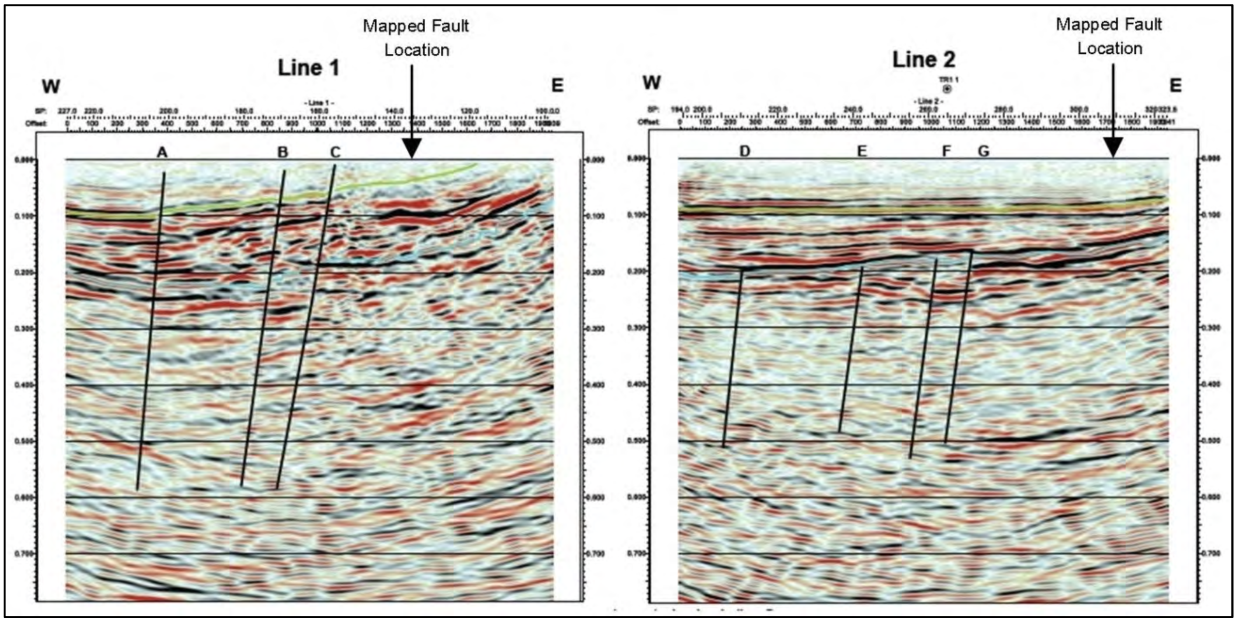

Figure 1: Final migrated seismic sections for RTC Project

The geological mapping of Ramelli and Henry (*5*) projects a concealed fault or fault zone across both of the seismic lines. However, the interpreted faults seen on the seismic lines are clearly to the west of the concealed fault.

Through a combined team approach, Zonge International and its partners have successfully completed a high resolution seismic reflection survey to determine the location of faults and deep stratigraphy within the Basin and Range province outside of Reno, NV. These data provided good seismic images to depths of over two thousand feet. Data quality was very good and faults and sedimentary layering were readily identified. The projected fault was not found to be in the previously mapped location; however several faults were identified to the west and were correlated from line to line. The correlation was non-unique and there are other possible correlation combinations.

Private Dam Site – Missouri

Project Outline

In support of a private facility's proposed new dam location, seismic datasets along two intersecting profiles were acquired. The original geological assessment of this site from widely spaced geotechnical borings suggested a bedrock depth of 40-50 feet below the ground surface (BGS). The general geology of the site consists of alluvial materials of variable thickness in a stream valley with a strong hydrologic gradient. The geologic map of Missouri (*6*) depicts the Eminence Dolomite, a massive bedded dolomite, as the bedrock underlying alluvium in this area. In the year prior to this case history, Zonge conducted a geophysical investigation targeting karst features within the dolomite. Results from the electrical resistivity survey indicated a bedrock profile much deeper than the expected 40-50 feet. Geotechnical borings were drilled to verify those geophysical results. The boring located in the central portion of the area of concern was abandoned at a depth of 80 feet BGS without encountering bedrock.

The primary objective of this geophysical program was to map the depth of bedrock, a secondary objective was to determine compressional and shear-wave velocities at several points along each of the profiles. These datasets would be used in the geotechnical design of a new dam structure. Seismic reflection and refraction methods were selected to map the top of bedrock and the compressional wave velocity of the overburden (alluvial) material. The MASW method was selected to obtain shear wave velocities of the overburden. Using the same seismic setup and geometry, Zonge International collected reflection, refraction, and surface wave (MASW) data concurrently and handled processing of the refraction and MASW data while Excel Geophysical conducted the reflection data processing and Summit Geoscience provided the initial interpretation.

Survey Setup

The seismic reflection survey was designed to have an effective exploration depth exceeding 300 feet while the refraction and MASW methods were designed to achieve depths of approximately 100 feet. A geophysical program of this nature necessitated a high-resolution survey setup using a 10-foot source and receiver interval along each line. Seismic data were collected with a roll-off/split-spread receiver configuration utilizing a series of five Geometrics Geode seismographs, each capable of recording 24-channels, for a total of 120 active recording channels. Shots were collected on the half-station as well as at pre-determined offsets beyond the ends of each line. The primary energy source was a 16-lb sledgehammer supplemented by a 40-kg accelerated weight drop, primarily used for comparison purposes. Survey control for source points and receiver locations was obtained with a Trimble ProXH PPK (post-processed kinematic) GPS capable of field-ready sub-meter horizontal and vertical accuracy. Because of the relatively shallow exploratory depths, as well as a shallow water table, each source produced excellent quality data. All fieldwork was conducted over the course of two days with a four-person survey crew.

Interpretation and Results

Seismic reflection data identified two horizons on Line 1 and three horizons on Line 2 (see Figures 2 and 3). The upper horizon outlines the contact between unconsolidated alluvial sediments and the underlying bedrock. The deeper horizons (more clearly seen on Line 2) identify structural dipping within the bedrock. Additionally, minor faults were identified along Line 1. Most important to the goals of this particular project is defining the apparent bedrock depression visible along both lines and most notable along line 2. On line 1 the depression appears to be bounded by the interpreted minor faults. The seismic refraction data shows good agreement between the interpreted faults and lateral velocity changes evident in the tomogram.

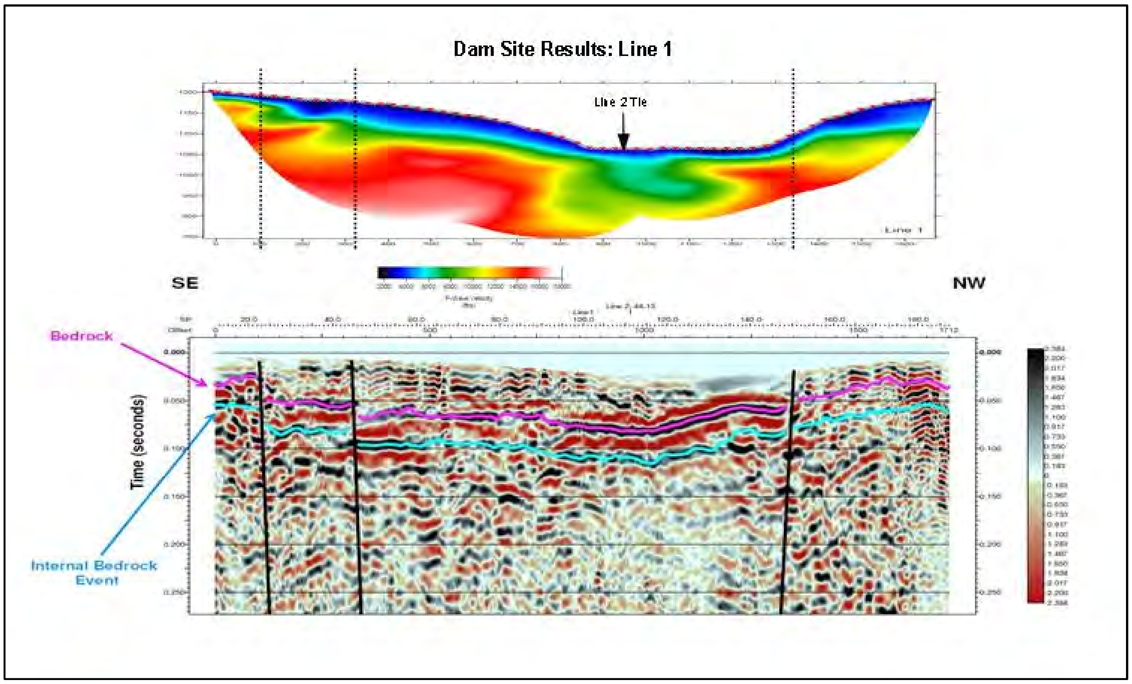

Figure 2: Seismic refraction tomogram (top) and final migrated seismic reflection profile (bottom) for Line 1

Seismic refraction tomograms and 1D MASW profiles were used to aid in the interpretation of the reflection data. The refraction tomograms clearly show a thickened low velocity zone within the central portion of each seismic line with good agreement where the lines intersect. This depression identifies bedrock dropping to depths of at least 80 feet below the ground surface, compared to depths of 10-50 feet nearer to the edges of each line. Figure 3 presents the reflection and refraction results for line 2 where the low velocity zone is more pronounced and the dipping bedrock structure is evident. The location of the low velocity zone within the tomogram agrees well with the bedrock low imaged in the reflection profile. Dipping strata within the bedrock are clearly evident beneath the depression and it has been shown that bedding interfaces can be a contributing factor to the development of karst systems (*7*).

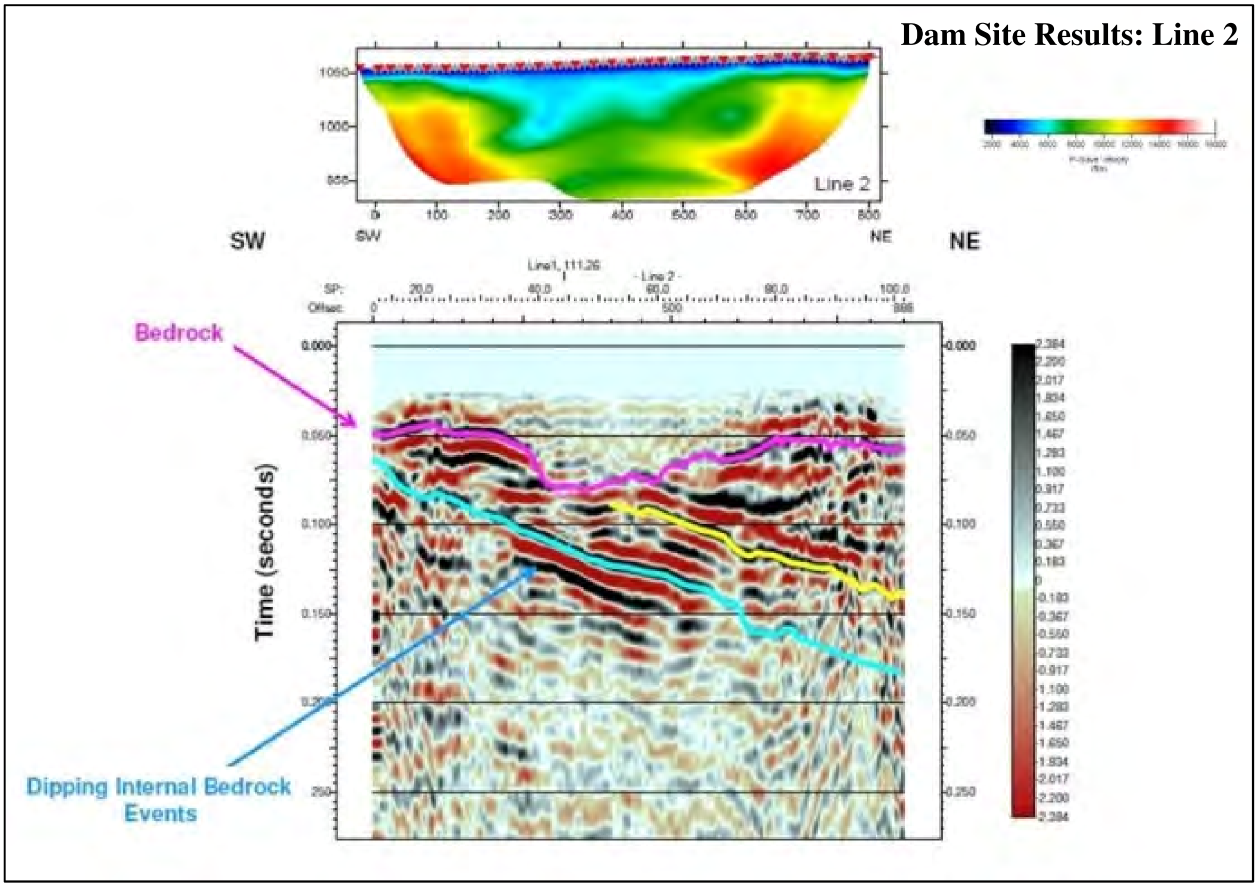

Figure 3: Seismic refraction tomogram (top) and final migrated seismic reflection profile (bottom) for Line 2.

Cave systems, large springs, and other karst features have been well documented throughout South-Central Missouri within the Eminence dolomite (*7*). Considering the strong response and dipping nature of the alluvium/bedrock interface within the reflection data, a thickened central low velocity zone identified within the refraction data, and a repeatable velocity inversion seen in the MASW data within this depression/low velocity zone, we interpret a potential karst-structure given the geology of the area (karst susceptible dolomite). Based on these anomalous results and confirmatory borings, a more detailed geotechnical assessment of the site is currently being conducted prior to dam site selection.

Federal Research Project – Western United States

Project Outline

A suite of seismic surveys were performed as part of a geotechnical assessment of several test sites in the desert southwest of the United States. Expected geologic conditions were Quaternary Alluvium overlying faulted granitic bedrock. Seismic Reflection, 2D and 3D P-wave Refraction, 2D MASW, and 1D Refraction Microtremor data were collected throughout the site.

The focus of this discussion is the seismic reflection study necessary to characterize stratigraphy and faulting in the study area.

This site was located on environmentally and culturally-sensitive land and destructive impact to the soil was closely monitored.

Survey Setup

A RT-1000 wireless seismic system (Wireless Seismic, Inc.) was used for all data acquisition. This system consists of individual wireless remote units (WRUs) forming a local area network transmitting seismic data to a central recording unit in a trailer. Backhaul radios were used approximately every 75 to 100 stations within the network to compress data and speed data transmission. Using this configuration it was possible to shoot the entire project without moving the recording trailer, thereby maximizing acquisition time, reducing site impact, and increasing security.

300 WRUs were deployed on this project. Each WRU was connected to a single geophone. A WRU and string spacing of 30 feet was used for all data acquisition. A US Alliance AF-450 track-mounted impulsive, weight-drop source system was used to generate the seismic energy. Shots were collected at every second geophone location (60-foot spacing). In addition, shots were collected 30, 60, and 90 feet off the ends of each line.

Data for three of the seismic methods (reflection, refraction, MASW) were acquired concurrently by having every WRU active during the shoot (300 recording channels). This resulted in a single command dataset from which individual traces and records could be extracted depending on the method being considered (reflection, refraction, MASW). For the MASW a subset of geophones was selected such that there were 12 or 24 active geophones a set distance from each shot. For the 2D refraction all shots and geophones were selected and the recording window was trimmed. Requirements of the method necessitated that microtremor data had to be recorded separately from the command dataset. Though recorded separately, geophone receivers were not moved from previous locations. The source truck was randomly triggered repeatedly some distance off the end of each side of the reflection lines while recording the individual records.

Due to site impact constraints, each reflection line begins and ends near the position of a receiver. Additionally, the reflection lines often end or begin near shallow granite, which is problematic for reflection based on station spacing. As such, because of these two constraints, data quality and results suffer near the ends of the lines except in situations where bedrock is deep.

Approximately 17,000 lineal feet of high resolution reflection, 2D and 3D refraction, and 2D MASW data were acquired over four days by an acquisition crew of four persons.

Interpretation and Results

Confidentiality agreements do not allow for a detailed presentation of the results for this project. However, broad discussion of the findings has been authorized. The survey was successful in detailing the alluvium bedrock interface to depths in excess of 200 feet and was able to clearly identify offset beds and multiple faults running through the study area. The 2D MASW and P-wave refraction datasets were able to complement the reflection data by providing alluvium thickness estimates, soil stiffness parameters, and through comparison of the P and S-wave results, reporting of elastic constants along each line. All of this information was obtained from a single seismic system, and a single command dataset, on one individual mobilization.

Riley Creek Bridge Replacement Project - Denali National Park, Alaska

Project Outline

A seismic reflection survey was chosen to further characterize the Park Road Fault as part of the design process for a new bridge alignment. At the southeastern extent of Denali National Park, the Park Road fault has been mapped as being near coincident with an existing bridge alignment along the Parks Highway at Riley Creek (*8*). That location is based on a review of LIDAR data as well as exposures of the fault within test pits. The fault is mapped as a high-angle thrust fault with the up-thrown block coming from the north. Fault scarps in the area suggest up to 15 feet of Holocene displacement. The site lies on thick Quaternary alluvial sediments of the Nenana River and Riley Creek overlying Pre-Cambrian to Paleozoic schist to the north and Late Cretaceous sedimentary and volcanic units to the south.

Survey Setup

Performing a high resolution reflection survey within a protected National Park presented special challenges. Though our seismic lines were located within dense forest and floodplain vegetation, the clearing of brush or removal of any vegetation was not permitted. Secondly, the use of explosives or larger scale impact energy sources was not allowed. Finally, fieldwork was conducted during the winter season, with working temperatures between -15°F to +10°F, limited daylight hours, and all equipment had to be hand packed to each line as there was no vehicle access. Regardless of the constraints, a four person crew was successfully able to collect the reflection data within five (short) field days.

A total of approximately 4,000 feet of seismic reflection data was collected along four lines selected to cross the expected fault location and minimize ground impact. Line lengths ranged from 700 to 1,200 feet, with each line crossing the mapped location of the Park Road fault. Using a geophone spacing of 10 feet, with up to five Geometrics Geodes connected in series providing between 96 and 120 recordable channels, it was possible to shoot each line with one single setup. A 30 pound slide hammer served as the seismic energy source. Shot points were located on the half-station of each geophone interval.

Interpretation and Results

Given the mandated restrictions governing the data acquisition process, the quality of the final results was moderate at best and the least successful of the four projects outlined in this paper. A primary factor in the data quality appeared to be the degree of organic cover controlling geophone coupling. Geophones could not be planted firmly into stiff soil because a very thick organic mat was present, and could not be removed. As expected, this served to dampen the recorded seismic signal. Additionally, high winds and swaying trees reduced signal coherency and a tight production/permitting schedule did not allow for project delays due to weather.

Reflection data from the lines where coupling and noise were less significant issues successfully imaged the alluvium/bedrock interface at depths of 100 to 300 feet (using an assumed velocity datum shift of 6,000 ft/sec for alluvium). Interpretation of the fault location was laterally constrained by the previously mapped surficial location. Attitude of the fault was difficult to determine due to the near-vertical nature of the anticipated offset. Additionally, this is a high energy fluvial environment. Alluvium observed within stream and river cut banks was cobble to boulder size gravel. Offset bedding within the alluvium would be ideal to determine fault throw and attitude.

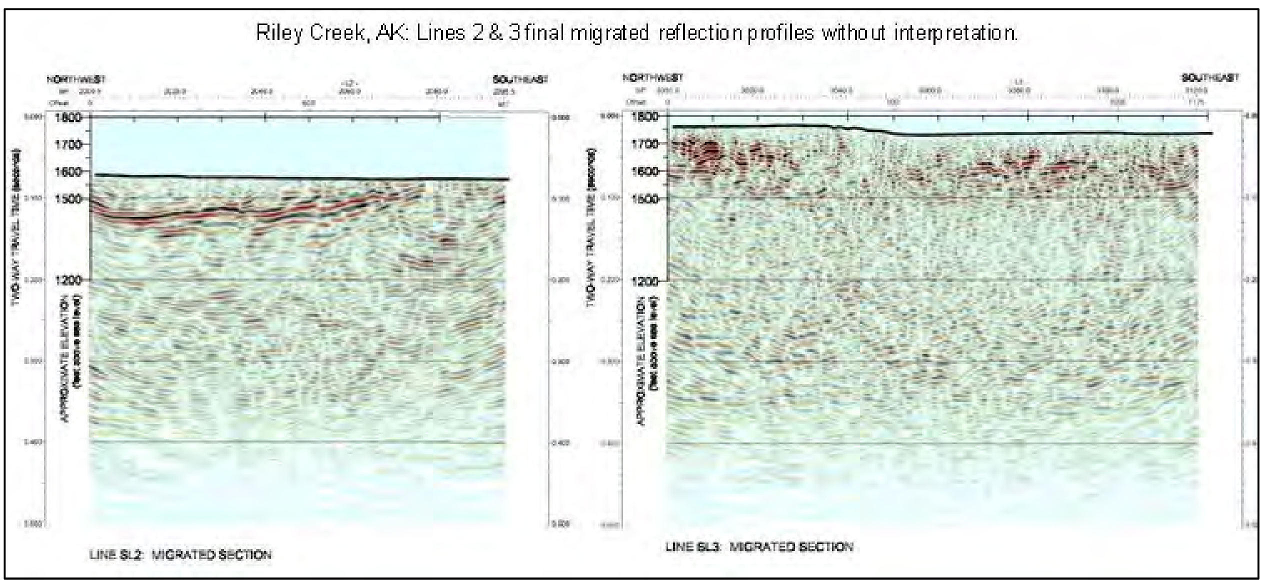

Figure 4: Final uninterpreted migrated sections of Lines 2 and 3 from Riley Creek, AK.

Figure 4 presents the final migrated reflection profiles from Lines 2 and 3 and clearly shows the alluvium-bedrock interface along Line 2. Of particular interest is that this dataset was collected adjacent and parallel (within 20-50 feet) to the fast flowing Riley Creek and yet, even with the stream noise (and periodic highway noise), imaging of the interface was successful across a majority of the line. Line 3, collected in dense forest with thicker organic mat, suffered from a combination of poor geophone coupling and wind noise showing that, in this project, these factors are much stronger contributors to a poor dataset than the constant random noise of a large stream or river.

CONCLUSION

We have shown four example projects where the high-resolution seismic reflection method was used in support of engineering projects. The method was selected because of the resolution requirements and other geophysical techniques (by themselves) would not produce meaningful data at the required depths. The geophysical data was used to reduce the overall number of exploratory borings as well as locating anomalous areas for targeted geotechnical exploration.

100 years of geophysical research aimed at exploiting earth's natural resources has given us the controlled-sweep seismic sources, gas-fired weight drops, multi-component geophone arrays, and higher-resolution digitizers which increase our observed signal-to-noise ratio. New developments in wireless geophone technology and in-field data QA/QC tools have tremendously reduced the time required to collect and produce draft result sections. Professional, trained processing firms have the capability to process shallow reflection data with the same tools and experience used to generate two-and three-dimensional sections for oil and gas exploration. We hope this paper shows that the use of several hundred recording channels and variable seismic sources is becoming more and more commonly applied to environmental and engineering projects, at costs competitive to traditional shallow geophysical surveying.

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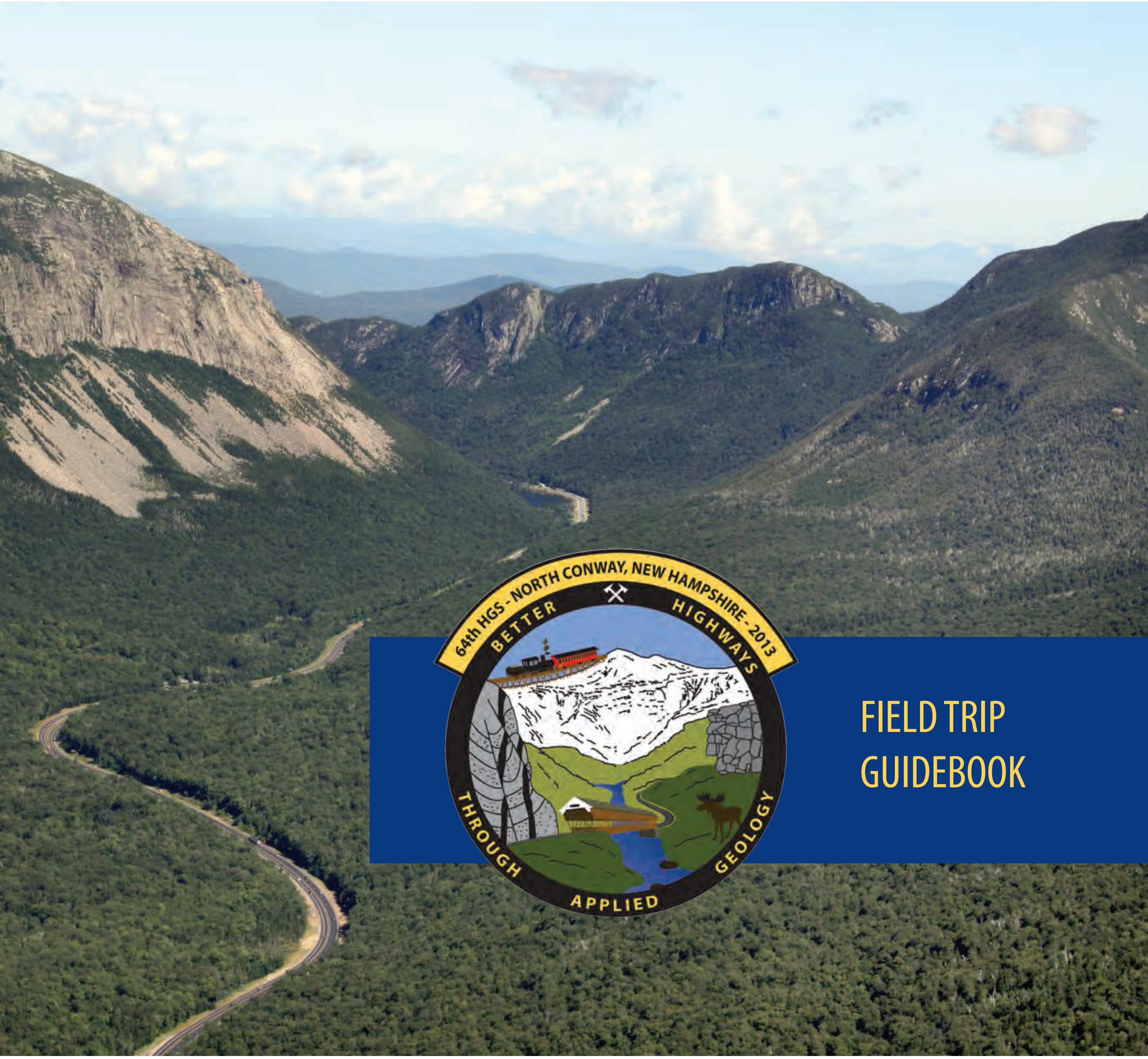
64TH HIGHWAY GEOLOGY SYMPOSIUM

SEPTEMBER 9-12, 2013 | NORTH CONWAY GRAND HOTEL | NORTH CONWAY, NEW HAMPSHIRE



FIELD TRIP GUIDEBOOK

Hosted and Coordinated By:

The New Hampshire Department of Transportation
Bureau of Materials and Research - Geotechnical Section

On Cover - Picture of Franconia Notch Parkway

Table of Contents

Field Trip Logistics

Welcome to the Field Trip for the 64th Highway Geology Symposium, headquartered in North Conway, New Hampshire. All participants will be travelling in deluxe, 50 passenger coaches. Sturdy footwear (sneaker and not sandals) is recommended.

Lunch will be catered at the Peabody Base Lodge at Cannon Mountain.

The lunch is sponsored by **Geobrugg,** and field trip refreshments are sponsored by **Golder Associates**.

GEOBRUGG® Golder Associates

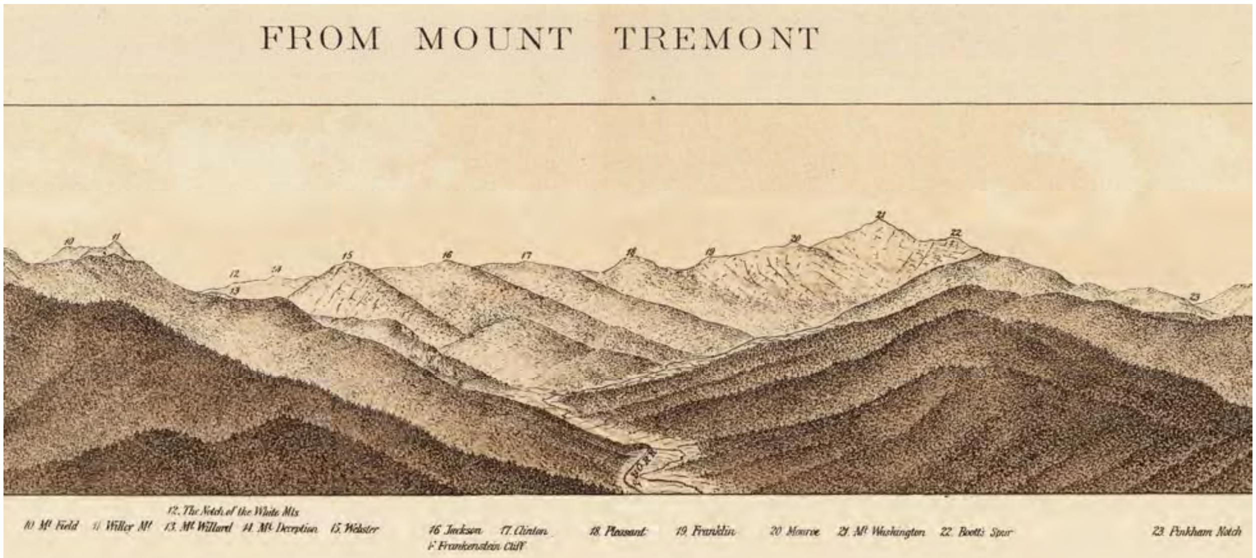
Field Trip Itinerary

Wednesday, September 11, 2013

6:30 AM	Buses arrive at North Conway Grand Hotel for boarding
7:00 AM	Buses leave North Conway Grand Hotel
8:00 - 8:45 AM	Stop 1: Pemigewasset Scenic Overlook
9:15 – 10:15 AM	Stop 2: Barron Mountain Rock Cut
10:30 - 11:15 AM	Stop 3: Old Man Historic Site
11:30 - 1:00 PM	Lunch: Cannon Mountain Lodge
1:35 - 2:05 PM	Stop 4: Carroll Visitors Center
2:15 – 2:45 PM	Stop 5: Mt. Washington Scenic Overlook
3:00 – 4:00 PM	Stop 6: Willey House
4:45 PM	Field Trip concludes at the North Conway Grand Hotel

Note: There are 3 drive-by sites (A, B & C) of interest included in the field trip, which are described in the overview.

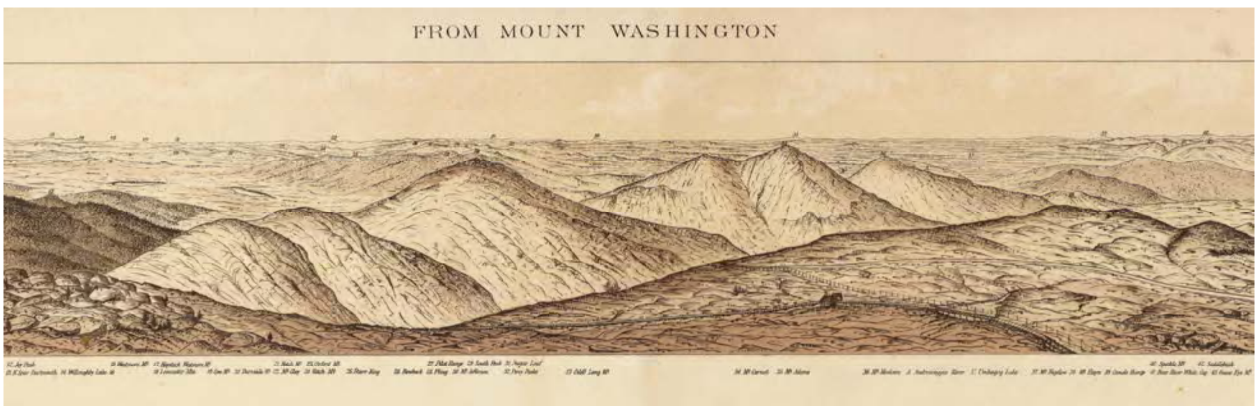
Field Trip Overview



New Hampshire's identity as the "Granite State" dates back to the early nineteenth century, even before the First Geological Survey was authorized by the state legislature in 1839. Although the nickname is well-deserved given the widespread occurrence of granite and early importance of granite quarries as local, then commercial, sources of building stone, it fails to convey the true complexity of the geology that is found here. The rocks of New Hampshire record over 625 million years of earth history. Interpretations of that history, at first by a generation of gentlemen scientists and natural philosophers, reflected the prevailing and later discounted geologic paradigms of their times. Their skilled and careful observations, however, laid the foundation for those who followed them, mapping and re-interpreting in light of a growing and evolving understanding of planet earth. In New Hampshire, such luminaries as Marland P. Billings (Harvard University) and John B. Lyons (Dartmouth College), and their students, led the way into the modern era. General acceptance of the concepts of plate tectonics in the 1970's and development of accurate radiometric age dating techniques provided the foundation for the synthesis of knowledge that is represented by the most recent statewide bedrock geologic map (Lyons et al. 1997). Within the past 20 years, Dykstra Eusden and a host of geology students at Bates College have added significantly to our understanding of the bedrock geology of the Presidential Range of the White Mountains (Eusden, 2010).

At the conclusion of three year's field work that focused on completing a series of transects, several oriented southeast to northwest and perpendicular to the predominant structural trend, the first State Geologist of New Hampshire (1839-1842), Charles T. Jackson, mistakenly reported that granite constitutes the central axis of the White Mountains. The highest peaks in the Presidential Range are actually underlain by metasedimentary rocks of the Littleton Formation. In Jackson's defense, the relatively primitive state of roads and the breadth of unsettled regions beyond a small number of population centers made travel and mapping extremely difficult. The White Mountain region at the time was wilderness, inhabited by a few intrepid pioneers, offering limited accommodations for travelers. Three decades later, state geologist Charles H. Hitchcock recalled his experiences with the Second Geological Survey that was completed in 1875:

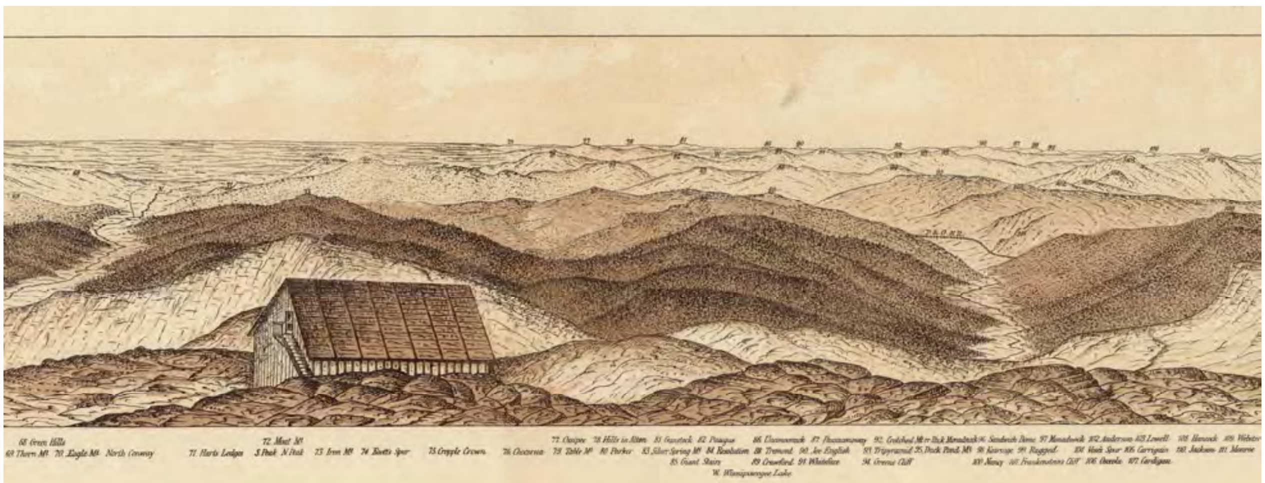
"*The progress of the New Hampshire survey was much retarded by the presence of a dense forest covering an area of 2000 square miles in the northern portion of the state, and by the difficulties of transportation. All this mountainous forest had to be traversed on foot mostly without paths or guides. From the summit of Mount Washington a sea of mountains is visible. Every one of them was visited by some member of the survey, observations made and specimens preserved for study. At the present time [1896] railroads thread three-fourths of this forest country, and by excavations and the removal of the forests, facilities for exploration have been greatly increased. Had the survey of this region commenced fifteen years later, the information acquired could have been gathered in a fourth part of the time actually taken*." (C.H. Hitchcock, The Geology of New Hampshire, The Journal of Geology, Vol. 4, No. 1, Jan. – Feb. 1896, pp. 44 - 62).



From this perspective, the intersections between "highways" and "geology" are quite literally groundbreaking. No doubt Jackson and Hitchcock would have marveled at the field trip itinerary for the 64th Annual Highway Geology Symposium. The 120-mile loop not only traverses spectacular mountain scenery and sites of important historical events, but the Kancamagus Highway [NH Route 112], Franconia Parkway [I-93], and the 10th New Hampshire Turnpike [US Route 302] that comprise much of the route are themselves part of this history.

In the early twentieth century, state tourism officials boasted of the White Mountains region of New Hampshire as the "Switzerland of America", exploiting another nickname for the state that was popularized in Hayward's New England Gazetteer of 1839. While the comparison may be an exaggeration, at least one Swiss connection is worthy of note. Stop 4 introduces the glacial geologic history of the White Mountains and the contribution of the Swiss geologist Louis Agassiz who visited the Bethlehem area in 1847 and recognized some of the same features he had come to know in his beloved Alps. As you will learn, once the glacial origin of "drift" displaced the dilvuialist theories of the day, debates began in earnest over the detailed histories of continental versus alpine glaciation in the Whites. The matter is still not entirely settled, although recent 1:24,000-scale surficial geologic mapping under the auspices of the New Hampshire Geological Survey and the cooperative U.S. Geological Survey STATEMAP program, aided by acquisition of the first LiDAR terrain data within the region, is shedding new light on the subject.

The historical intersection between "highway" and "geology" is apparent once again in the generation of surficial mapping that was sponsored by the NH Highway Department (now Department of Transportation) in the 1930's. The expressed purpose of this project, under the leadership of James W. Goldthwait, the third NH State Geologist, was to locate the materials that were needed to expand the road network and stimulate tourism and economic growth. Subsequent mapping continued this focus on sand and gravel as an economic commodity, but beginning in the 1970's another focus was added, availability of groundwater resources. Cooperative projects with the U.S. Geological Survey produced a statewide series of stratified-drift aquifer maps that progressed from 1:125,000 to 1:24,000 scale before the final report in the latter series was published in 1997. These maps are still being widely used today as the basis for local groundwater protection.



New Hampshire History

New Hampshire can only claim 18 miles of Atlantic coastline as its own, the least of any coastal state in the United States, but that limited stretch of real estate has played a disproportionate role in its history. Explorers early in the 17th century recognized the potential of the deep-water harbor at the entrance of the Piscataqua River, leading into the tidal waters of the Great Bay and its major tributaries. English fisherman gained the first foothold, setting up temporary outposts from which to exploit the bounty of the rich coastal waters. The Isles of Shoals, a cluster of islands 10 miles offshore from the Piscataqua harbor (now divided between New Hampshire and neighboring Maine), supported active fishing communities.

The first organized attempt to create English settlements on the mainland came after Captain John Mason and his partner Sir Ferdinando Gorges (who eventually settled Maine) received land grants in 1622 from the Council of New England under the authority of the Crown. In 1623, David Thompson established the first settlement in what later would become New Hampshire at Ordiorne Point several miles south of the Piscataqua River in the present-day town of Rye. His Pannaway Plantation only lasted four years but several of its original inhabitants, brothers Edward and William Hilton, moved seven miles upriver to form their own settlement (now the City of Dover).

Captain Mason invested heavily in the company that he formed to establish and sustain settlements within his lands, a province which he named "New Hampshire" after the English county of Hampshire where his family seat was located. In 1630, the settlement of Portsmouth (originally known as Strawbery Banke) was founded on the west bank of the Piscataqua River harbor. This strategic location insured its continued growth and eventual prosperity as a center of maritime trade and shipbuilding. Mason died suddenly in 1635 at the age of 49 without ever setting foot in his province or seeing his investments become profitable.

The fur trade with the native inhabitants never became the profitable venture that was imagined by the early settlers. Relations with the indigenous tribes, the Pennacook and Abenaki (meaning "people of the dawnlands"), were peaceful at first, but hostility grew as more and more settlers occupied ancestral Indian lands and European diseases decimated their villages. Many of the new settlers came north from the Massachusetts Bay Colony, perhaps to escape the strictures of Puritan society in the largely ungoverned province of New Hampshire. Almost certainly they were lured by the plentiful land, timber, and fish and the economic opportunities that this natural wealth represented. The Puritan authorities in the Bay Colony coveted the same resources and alliances formed with some of the leading investors in the Piscataqua region to merge the two colonies.

After Mason's death, questionable claims arose regarding titles to the early settlements, compounded by a gross misconception of the true course of the Merrimack River that was specified as the southern boundary of his grant. Despite the efforts of Mason's heirs, the four plantations then existing in New Hampshire fell under the political control of Massachusetts until 1679. At that time, the Crown, already suspicious of the Bay Colony's expansionist aspirations and weary of the squabbling over governance, declared that New Hampshire constituted a separate colony and established a royal governor, an appointed council, and an elected assembly to govern it. The assembly was the precursor to the New Hampshire General Court that, with its 424 members, is the largest state legislature in the United States and one of the largest elected bodies in the world today.

Subsequent successors to the Crown allowed the balance of political power to shift back in favor of Bay Colony allies when jurisdiction to govern both colonies was granted to a royal governor with authority over his lieutenant who served in New Hampshire. However, the tide began to turn once more when John Wentworth, a native of Portsmouth, was appointed as lieutenant governor of New Hampshire in 1717, beginning a dynasty of royal appointments that included two more generations of Wentworths. By skillfully courting the favor of the Crown and wealthy merchants within the province, John Wentworth expanded his family's influence and further frustrated ambitions of the Bay Colony to assert control over New Hampshire.

The disputed boundary between the two colonies finally emerged as a major political battleground in the 1720's. John Wentworth died in 1730 before seeing the issue settled, but an agent that he enlisted to plead the case before authorities in London, Captain John Thomlinson, ultimately prevailed. In March of 1740, a measure advantageous to New Hampshire was passed setting the southern boundary along a line due west from the southerly curve of the Merrimack River at Lowell, Massachusetts. Thomlinson achieved a final coup in 1741 by successfully lobbying King George II to appoint Benning Wentworth, John Wentworth's son, as royal governor of New Hampshire, separate from and independent of Massachusetts.

The political power struggle between these two New England colonies was not the only source of conflict in the region during the early period of settlement. New Hampshire occupied the frontier between British and French territorial claims in North America which insured that it would be a battleground in the protracted struggle for domination. A succession of wars between the English settlers and the French and their Indian allies began as early as 1675 and lasted almost one hundred years, with depredations on both sides. Dover was raided by several hundred Abenaki and Pennacook Indians in June 1689 under the command of chiefs Kancamagus and Mesandowit. More than 20 settlers were killed and 29 more taken captive and marched to New France to be sold or held as hostages.

Numerous chilling accounts of such attacks and heroic defenses exist from this period, creating their own literary genre, the "captivity narrative." The story of Hannah Dustin (a.k.a. Hannah Duston), who was captured in Haverhill, Massachusetts along with her newborn daughter and her nurse during a raid in 1697, is especially compelling. On the way north, the Indians murdered the baby and several other captives. Hannah Dustin, with the aid of the nurse and a teenage boy, were able to overpower their captors while they camped on an island in the Merrimack River, killing two adult men, two adult women and six children before scalping them and escaping downriver in a canoe.

Source: http://en.wikipedia.org/wiki/File:Hannah_Duston,_by_Stearns.jpg

The site in Boscawen, New Hampshire is commemorated by a statue of Hannah Dustin wielding a hatchet, which was erected in 1874, the first publicly funded statue in New Hampshire.

Settlement continued, despite the dangers. The vast timber resources of the virgin forests provided an irresistible economic incentive. By the 16th century the English homeland had been largely denuded of its own forests and the British navy required a secure and steady supply of exceedingly straight, tall trees, "mast trees", to maintain its maritime superiority. The settlers required lumber for buildings, barrels, and household tools of all descriptions, as well as up to 40 cords of firewood each year for fuel. The old-growth forests, with their giant white pine trees up to 230 feet tall, met both requirements.

The first pine masts were shipped out of the Piscataqua region bound for British ports in 1634 and Portsmouth came to dominate the lucrative masting trade until shortly before the American

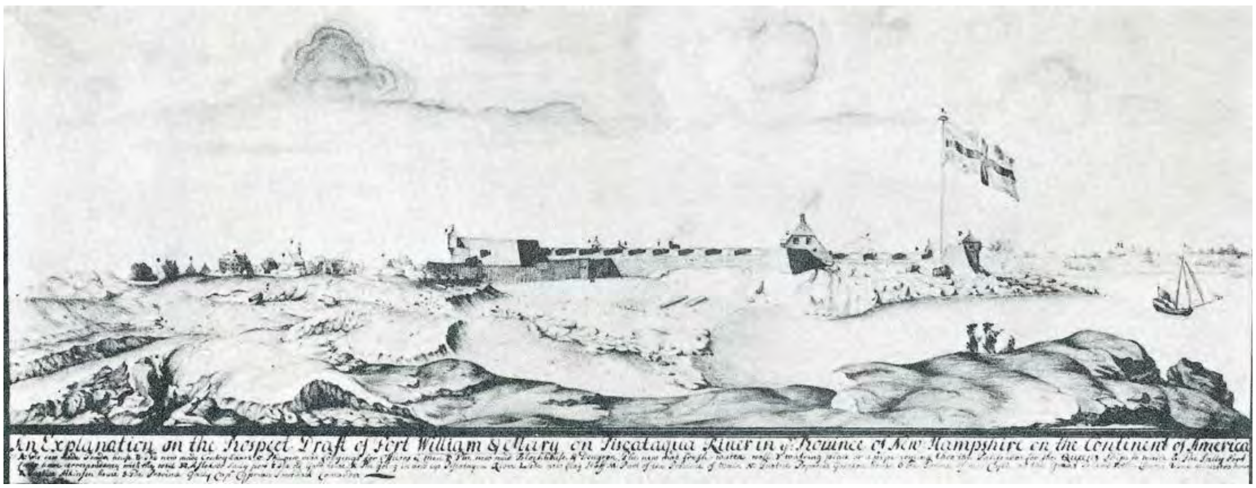
Revolution. The glaciated landscape of New Hampshire provided abundant opportunities to develop water power and sawmills and grist mills sprang up as a vital part of almost every new settlement. The settlers could turn the surrounding forest into products to meet their own needs, but also soon realized that there was an enormous export market for lumber, clapboards, shingles, and barrel staves throughout the British colonies.

As the easily accessible timber in the Piscataqua region was cut over, competition rapidly increased between the settlers and their merchant middlemen and the powerful colonial agents of the masting trade. Restrictions on the cutting of pine soon followed in 1691, enforceable by Crown-appointed Surveyors of His Majesties Woods and Forests. Pines more than 24 inches in diameter at 12 inches above the ground were branded with the King's Broad Arrow as potential mast trees and property of the Crown. With three quick strokes of an ax, surveyors appropriated the best pines for the Royal Navy. Settlers routinely poached these pines and sawed them into boards that were no more than 22 inches wide to avoid being discovered by the King's agents.

Enforcement of the white pine laws was lax under royal governors John Wentworth and then his son Benning Wentworth. Although they both profited immensely from the masting trade, they also benefited both politically and financially from the success of merchants in the lumber trade. A delicate balance was required to maintain the favor of the Crown as loyal subjects while encouraging the entrepreneurial spirit of the colonists under their direct authority.

Prospects for increased settlement improved after the Treaty of Paris was signed in 1763, ending the French and Indian Wars. Benning Wentworth took full advantage of confusion over the western boundary of New Hampshire and began to charter new towns on both sides of the Connecticut River. Under his skillful leadership, which came to an end in 1767 when he lost favor with the Crown and relinquished the governorship, the colony expanded and became more secure and prosperous. Political maneuvering at the court of King George III resulted in Benning's nephew, John Wentworth II, being named as his successor. Unfortunately, John II lacked the friends in high places that his kinsman had so effectively cultivated in London and could not afford to be so cavalier about enforcing the laws against smuggling and cutting of the King's pines. His heavy-handed approach, however, did not win him friends among the colonists either. What became known as the Pine Tree Riot transpired during April of 1772 in the town of Weare after one of Wentworth's inspectors charged a number of local men with cutting a large number of the King's pines, marked the illegal logs for seizure, and fined all of the offenders. In an act of open defiance, a mob of more than 20 men with faces covered in soot to hide their identities assaulted the government officials the following morning and sent them packing toward the Mast Road and out of town. Unrest in the colony would only increase and lead to the opening volleys of the American Revolution.

While the midnight ride of Paul Revere in April 1775 has achieved mythical proportions, dramatic events that occurred in New Hampshire during the previous December have received much less public notice. Paul Revere served as a courier from Boston to Portsmouth to deliver an urgent message that the British had banned export of military stores to America. More alarmingly, he reported that troops were already en route to occupy Castle William and Mary on Newcastle Island in Portsmouth harbor, intent on securing all its arms and ammunition. The British were coming. A preemptive strike was organized, and on December 14 four hundred patriots under the command of Captain Thomas Pickering and Major John Langdon overwhelmed the five British defenders and liberated all of the gunpowder that they then distributed in nearby communities for safekeeping. The following night they came back for more, this time hauling away cannons, muskets, and other military hardware.



Source: http://en.wikipedia.org/wiki/File:Fort_William_and_Mary,_1705.jpg

The entire affair was a significant embarrassment for Governor Wentworth who was residing in Portsmouth at the time. An angry mob showed up on his doorstep in June 1775 to confront a friend of the governor, Colonel John Fenton, who was staying there. They brought a cannon with them and positioned it in front of the door, threatening to open fire if Colonel Fenton was not handed over. Realizing that resistance was futile, Fenton surrendered. Wentworth got the message and made arrangements to retire his family to the safety of Castle William and Mary, now guarded by two British warships, thence to Boston and finally to Nova Scotia, never to return to New Hampshire. So ended the royal Wentworth dynasty and began New Hampshire's struggle for independence from the Crown.

In January 1776, New Hampshire's became the first colony to write its own constitution and formalize its independence. In Philadelphia on July 4, 1776, New Hampshire delegates were accorded the honor of being the first to vote for the Declaration of Independence. No battles of the American Revolution were fought on New Hampshire soil, but the state contributed three regiments to the Continental Army. Native son and renowned Indian fighter, General John Stark, came out of retirement in July 1777 to lead New Hampshire troops to victory at the Battle of Bennington in southwestern Vermont. As a result of his victory, he decisively blocked the strategic offensive of British General Burgoyne who was attempting to cut off New England from the other colonies. Stark was remembered as rallying his troops on the battlefield by declaring with much bravado, "There, my boys, are your enemies, the red-coats and Tories; they are ours or this night Molly Stark sleeps a widow." The famous declaration "Live Free or Die", which is attributed to Stark and became New Hampshire's state motto in 1945, was actually never spoken by him during the conflict. Rather he penned the words as part of a toast he sent to his former soldiers in 1809 upon declining their invitation to participate in a reunion thirty-two years after the Battle of Bennington.

Portsmouth's long experience with seafaring and shipbuilding proved to be a major asset in the war effort. Skilled shipwrights produced numerous vessels for the Continental Navy but also for an intrepid navy of privateers bankrolled by local merchants and venture capitalists. After the war, tribute to this shipbuilding heritage was bestowed by the official seal of New Hampshire that depicts the 32-gun frigate Raleigh while still on the shipyard stocks in Portsmouth. (Major elements of the original seal, created in 1775 by the First Provincial Congress, included a pine tree and an upright fish, acknowledging the natural resources that supported New Hampshire's economy during the previous century). American naval hero John Paul Jones supervised the Raleigh's construction, along with that of another man-of-war, the 18-gun Ranger, which he later commanded. One of these two ships was the first to fly the Stars and Stripes after it was adopted as national ensign by an act of Congress in June 1777. On June 21, 1788, New Hampshire became the ninth state to ratify the Constitution of the United States, providing the final vote needed for it to become the law of the land. Delegates acted deliberately to achieve that distinction, beating Virginia. [New Hampshire's tenacious hold on its First in the Nation Primary status would appear to have deep historical roots.]

The 1817 edition of Merrill's "The Gazetteer of the State of New Hampshire" observed that: "Within the last twenty years, the roads of this state have been much improved, so that communication between the distant parts of it is much facilitated. Much however remains to be done, especially in the northern part of the state…. From the best information I can obtain, we have now open for travel 300 miles of turnpike road, and 300 more will soon be opened." (Page 12). A new era of economic development was beginning as the transportation network expanded. The corridors for settlement and the main arteries for trade and commerce were no longer defined by the major rivers. Those yeoman farmers who settled in the relative isolation of the uplands began to have access to more distant markets and they became a market for goods produced beyond their local communities. When the first railroad line was completed from Lowell, Massachusetts to Manchester, New Hampshire in 1836, the pace of economic change rapidly accelerated.

Manufacturing began on a scale that dwarfed that of the early sawmills and grist mills. Cotton and woolen mills and shoe factories attracted growing numbers of workers from the family farms or newly arrived immigrants to satisfy demand for products in far-flung markets. Two years after the railroad reached Manchester, construction of the Amoskeag Manufacturing Company began. The world famous gingham cloth and cotton ticking that flooded from the looms of Amoskeag gave Manchester its identity for almost a century. By the middle of the 19th century, the Amoskeag Manufacturing Company was the largest producer of cotton textiles in the world. In its heyday at the turn of the 20th century, the manufacturing complex included thirty major mills covering a total of 8,000,000 square feet of floor space and employed up to 17,000 workers.

Source: http://linguistlist.org/fund-drive/2011/hometowns/danielle/history.cfm

Writing in 1817, Eliphalet Merrill clearly understood the importance of transportation infrastructure as the precondition for development, but he likely never could have imagined the impact that the railroads would have on New Hampshire. The initial 35 miles of track grew to 92 miles in 1845, then 467 miles in 1850 and 661 miles in 1860. During that period Franklin Pierce, "the young hickory of the Granite Hills", was elected as the fourteenth president of the United States, the only New Hampshirite to hold that office. Another 239 miles of track were laid during the 1860's bringing the total mileage to 900 in 1870. During the Civil War years, railroad technology advanced significantly while New Hampshire mustered 18 regiments of volunteers in answer to the call for troops to preserve the Union. Of these, the Fifth Regiment is widely recognized for its hard fighting and number of battlefield casualties in all the major engagements of the Army of the Potomac.

By 1870, the year after State Geologist Charles H. Hitchcock began the Second Geological Survey of New Hampshire, the railroads had united the previously separated regions of the state, including the North Country. The story continues where the tracks ended, in the White Mountains.

White Mountain History

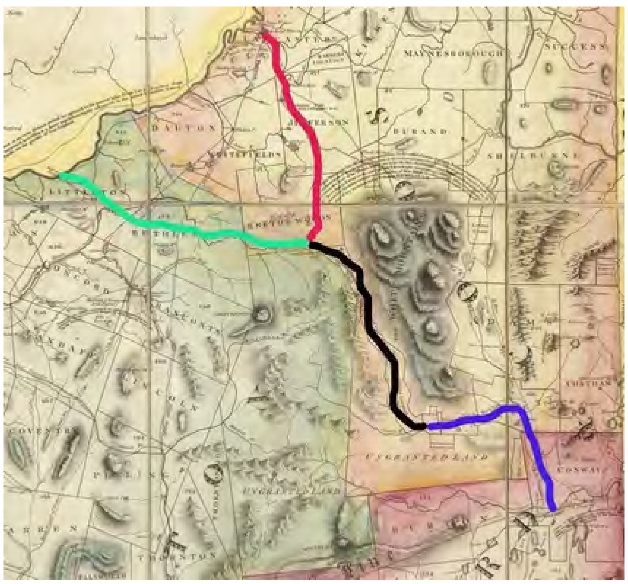
The White Mountains of New Hampshire have long held the fascination of residents and visitors alike. The earliest explorers off the coast of New England reported seeing these high mountains in the distance rising above the seemingly endless green of the virgin forests. Because this region of the state was so rugged, remote, and difficult to access, early explorers and settlers remained on the periphery and relatively few were bold enough to venture into the imposing notches.

The "discovery" of Crawford Notch by Europeans in 1771 or 1773 (depending on sources) is attributed to two hunters, Timothy Nash and Benjamin Sawyer. Governor John Wentworth II, upon learning of this discovery, is said to have offered Nash a grant of land if he could bring a horse through from Lancaster and prove that the route had potential to open up trade with the upper Connecticut Valley. The existence of Nash and Sawyer Location on maps today attests to their success in meeting the governor's challenge.

Rev. Guy Roberts in his booklet "The Willey Slide: Its History, Legend and Romance" (1925) describes the first "rude road" through the notch that was built with funds that were supposedly obtained from the sale of a confiscated Tory estate: "*In places it was so steep that horses and wagons had to be drawn up or let down with ropes. 'Sawyers Rock' being one such place. The first merchandise to go over the road after its completion was a barrel of tobacco taken down from Lancaster to Portsmouth by one Titus Brown. This was followed by a barrel of rum going in the opposite direction, it being a gift from a Portland firm to any one who would get it thru the*

Notch. Captain Rosebrook accomplished the feat, tho most of it was consumed en-route by 'those who helped to manage the affair'."

Publications such as this and the many that preceded it during the 19th century did much to popularize the region and attract and charm tourists, although the historical accuracy of events as reported may have suffered in the service of literary license. However, we do know with certainty that Abel Crawford was one of the first settlers, building a log cabin near the present location of the Fabyan Station Restaurant sometime around 1792. Shortly thereafter, he moved 12 miles down the Saco valley to the vicinity of Notchland and sold his log cabin at Fabyan to his father-in-law, Eleazer Rosebrook. Both locations became natural stops for teamsters hauling freight and other travelers along the road, so that Crawford and Rosebrook eventually found themselves in the hospitality business, providing food, spirits, and lodging to an increasing number of wayfarers. Their once humble accommodations become worthy of being called taverns, a pattern of development that repeated itself all over the White Mountain region in later years as trade, but especially tourism, increased.



The early notch road was succeeded by the 10th New Hampshire Turnpike, which was chartered by the New Hampshire Legislature in December 1803. [The route of the field trip traces the original 20 miles of turnpike from near Sawyer's Rock to the intersection of the Cog Railway Base Station Road with US Route 302. Cherry Mountain Road, which also intersects US Route 302 and is only open for seasonal use, could be the longest, mostly original section of a 19th century turnpike still in existence]. Construction of this and other early turnpikes was paid for by investors because the state did not have the means to fund such enterprises. Shareholders hoped to recoup their investments by charging tolls and furthering their business interests in the area.

Source: http://whitemountainhistory.org/Tenth_New_Hampshire.html

Despite Governor Wentworth's expectation of forging an efficient transportation link to Portsmouth, the 10th Turnpike proved to be a better stimulus for trade with Portland, Maine. Not surprisingly, as local tavern owners and innkeepers, Rosebrook, Abel Crawford, and Abel's son, Ethan Allen Crawford, were major proponents of this and related road building projects during the first decades of the 19th century. Not only were they stockholders, but they also assumed roles as directors, builders, and toll collectors. Abel and Ethan Allen Crawford are remembered as well for clearing a path to the treeline near the top of Mt. Clinton in 1819. The completed trail, known as the Crawford path, extends a total of 8.2 miles over the southern Presidential Range to the summit of Mt. Washington from Crawford Notch and is the oldest maintained foottrail in the United States. The Crawfords improved the trail as a bridal path in 1840. Charles

T. Jackson, first State Geologist of New Hampshire, made the first ascent on horseback that year with Abel Crawford as his guide. Jackson made the following observations:

"*The geological features of Mount Washington possess but little interest, the rocks in place consisting of a coarse variety of mica slate, passing into gneiss, which contains crystals of black tourmaline and quartz. The cone of the mountain and its summit are covered by myriads of angular and flat blocks and slabs of mica slate, piled in confusion one upon the other. They are identical in nature with the rocks in place, and bear no marks of transportation or abrasion by the action of water*." (Page 78) [Note: Jackson was an ardent believer in diluvialist theories and dismissive of any notion of widespread glaciation.]

Jackson did concede, however, that "the geologist will be fully rewarded for his toil in ascending this mountain, by the magnificent and comprehensive view which may be obtained of the surrounding country." Eminent British geologist Sir Charles Lyell and his wife made the trip in October 1845, in the company of an accomplished botanist from Boston, a gentleman and his wife visiting from Maine, a young New England artist, and three guides. The different interests of the various participants exemplify how appealing a visit to the White Mountains had become.

Road construction over the steep and rocky terrain of the Crawford Notch was challenging enough but maintenance of the road was equally if not more challenging. The rainstorm in August of 1826 that was responsible for the tragedy of the Willey family [Stop 6] washed out parts of the road and buried others under many tons of debris. The damage that occurred to the roadbed and bridges probably has a good analogue in the havoc wreaked in the same general area by Tropical Storm Irene in August 2011. [Bridges damaged by Tropical Storm Irene are located at the second and third drive-by sites.] The tragedy was sensationalized in the newspapers of the day and ironically became a boon to tourism as people were drawn to the scene of the disaster. To add to the sense of pathos and moral ambiguity of the event, chroniclers likely embellished the tale with details of questionable veracity, such as providing the image of a burnt out stub of a candle on a table beside the family bible in a hastily abandoned room, the bible open to the 18th Psalm which begins "The Lord also thundered in the heavens". The road was re-opened and by 1830 could be readily negotiated by stagecoaches, the preferred method of mass transit at the time. The Abbott Downing Company in Concord, New Hampshire manufactured some of the most widely used passenger models (the premier "Concord coach") on the road. Travel to Fabyan and other White Mountain destinations took less time and became at least less arduous if not more comfortable.

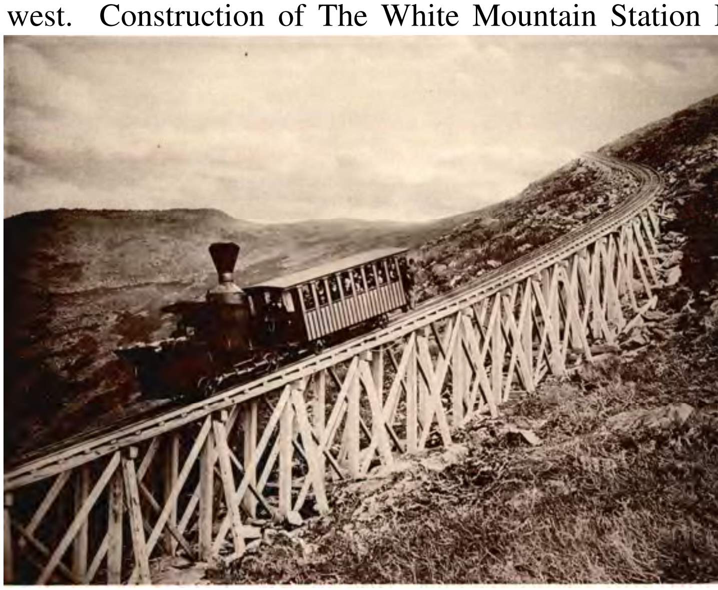
The approach of the first railroad lines ushered in a new era of comfort and convenience for the traveler. One could leave Boston or Portland in the morning and dine in the White Mountains that evening, perhaps even having time to take in a few of the sights before dinner. Stagecoach lines still provided links to the nearest rail depots. Sir Charles Lyell, shared his personal perspective on traveling by rail in 1845:

"*It is an agreeable novelty to a naturalist to combine the speed of a railway and the luxury of good inns with the sight of the native forest – the advantages of civilization with the beauty of unreclaimed nature – no hedges, few plowed fields, the wild plants, trees, birds, and animals* *undisturbed.*" (A second visit to the United States of North America, vol. 1, New York: Harper and Brothers, 1850, page 41.)

The popularity of the White Mountains as a travel destination grew steadily as more and more visitors came and shared their experiences. Praises were sung in the popular media of the day, tour guidebooks of all kinds abounded, inns and taverns aggressively promoted themselves, and artists gave expression to the majestic and picturesque landscapes that they encountered. A tour of the White Mountains, encompassing many of the remarkable geologic features and views that were widely publicized, soon became fashionable for those with financial means and leisure time. Sights such as the Old Man of the Mountain [Stop 3], the Flume, and the Basin in Franconia Notch were high on the list of what to see. In his popular book "The White Hills: Their Legends, Landscape and Poetry" published in 1859, Thomas Starr King appealed to visitors to stay in one location long enough to appreciate the effects that different qualities of atmosphere and light had on the scenery, rather than rushing from place to place heeding the itineraries promoted by the guidebooks. [Today's equivalent practice might be "bagging peaks", the attempt to climb all 48 of the peaks that exceed 4,000 feet in elevation]. King advocated returning to the same places in all seasons, even winter.

The many landscape painters who came to the Whites created a body of work (loosely referred to as the "White Mountain School") that essentially reflected King's aesthetic, although works depicting winter scenes are relatively rare. Painting flourished during the latter half of the century as some artists established studios or became artists-in-residence at the various hotels. [Two different exhibits of White Mountain art are currently open and highly recommended: "Passing Through: The Allure of the White Mountains" at the Museum of the White Mountains in Plymouth, NH (http://www.plymouth.edu/museum-of-the-white-mountains/exhibitions/) and "Mountain Scenery" at the New Hampshire Historical Society's museum in Concord, NH (http://www.nhhistory.org/museum.html).]

In 1851, rail service reached Gorham, at the gateway to Pinkham Notch, via the Atlantic & St. Lawrence Railroad that connected Portland, Maine in the east to Island Pond, Vermont in the west. Construction of The White Mountain Station House (later better known as the Alpine House) was completed that year, providing plenty of accommodations for passengers. A stage road to the future site of the Glen House at the base of Mt. Washington had already been completed the previous year. It was far from coincidental that the era of the grand hotels coincided with development of rail lines into the interior of the White Mountains. The tourist economy literally picked up steam as rail service arrived from different directions throughout the next quarter century, bringing an impressive number of visitors to the region.



A carriage road to the summit of Mt. Washington was completed in 1861, providing even easier access to the ultimate destination of many tourists than was offered by the already existing bridal paths. Attention shifted to the west side of the mountain once the Cog Railway (picture on previous page) was completed in 1869, an ambitious project that was the vision of Sylvester Marsh. Marsh received a legislative charter for the Mount Washington Railway Company in 1858 but didn't break ground for the project until 1866. The novelty and efficiency of this marvel of engineering proved to be as much a tourist attraction as the mountain itself.

The Portland & Ogdensburg Railroad (P&O RR) built one of the last lines into the interior of the Whites, confronting the considerable engineering challenges posed by Crawford Notch. Having reached Conway in 1871, track was extended to Bartlett in 1873 [the route followed by the Conway Scenic Railroad dinner train] and then on to Notchland one year later. Because of the steepness of the grade and the narrowness of the notch beyond Notchland, track was laid on a shelf that was blasted and excavated out of the sides of Mounts Bemis, Willey, and Willard to reach the "gate of the notch." Trestles were constructed to carry the tracks over gorges and ravines in the mountainsides. Frankenstein Trestle was named after an artist who frequented Notchland and not Shelley's monster.

The first train to reach Fabyan from Portland arrived on August 7, 1875, essentially completing the passenger rail network in the region. As momentous as this occasion might have been, an event that occurred eight years earlier in Concord would have far more extreme and lasting consequences. In 1867, the administration of Governor Walter Harriman authorized the sale of the state's extensive land holdings in the White Mountains (172,000 acres) to local landowners and speculators. The sale generated an estimated $25,000 in revenue deposited in a "literary fund" to support the financing and maintenance of schools. [Ironically, school funding remains a hotly contested issue in the state today.] A virtual land grab ensued, with large parcels of virgin forest being acquired by timber interests. The railroads too realized the opportunity to get a piece of the action, having by this time already amassed immense wealth and political and economic influence in the state. The miles of "mountainous forest" that Charles H. Hitchcock had experienced during the Second Geological Survey now came under the lumberman's axe and saw on an industrial scale.

To access all of those trees, timber barons such as James E. Henry of Lincoln contracted with the railroad companies to lease the equipment needed to build and operate their own network of logging railroads throughout the Whites. Seventeen spur lines emerged to carry the logs from the woods to the mills and then deliver timber products to market. Regrettably, the science of forestry was in its infancy at this time. Whole mountainsides were completely clear-cut in one area and then the operation pulled up stakes (and tracks) and moved on to the next. Vast tracts were left with nothing but slash everywhere, just waiting for a spark or lightning strike to set them aflame.

As this environmental catastrophe was unfolding, guests at the many grand hotels began to complain that their much-loved views were becoming blighted. Some days there were no views at all because smoke from fires obscured everything and soot and ashes rained down on the spacious hotel verandas, keeping guests indoors. In 1903 alone, over 12,000 acres burned. Needless to say, all of this was very bad for the businesses that depended on tourism. A public outcry against the logging abuses gained strength, joining the voices of hotel owners, conservationists, and even large mill owners beyond the region. The latter group recognized a threat to the sustained flow of water in the rivers that generated power for their machinery. The Society for the Protection of New Hampshire Forests was founded in 1901 to advocate for better forestry practices and land protection. Action finally came in the form of the Weeks Act that was passed by the U.S Congress in 1911 and was named for the senator who sponsored the legislation, John Wingate Weeks. The precedent setting argument was successfully made that the federal government had the right to purchase and own private property for the purpose of protecting the headwaters of navigable streams. This was the impetus for the creation of the national forest system. The White Mountain National Forest was officially recognized in 1918.

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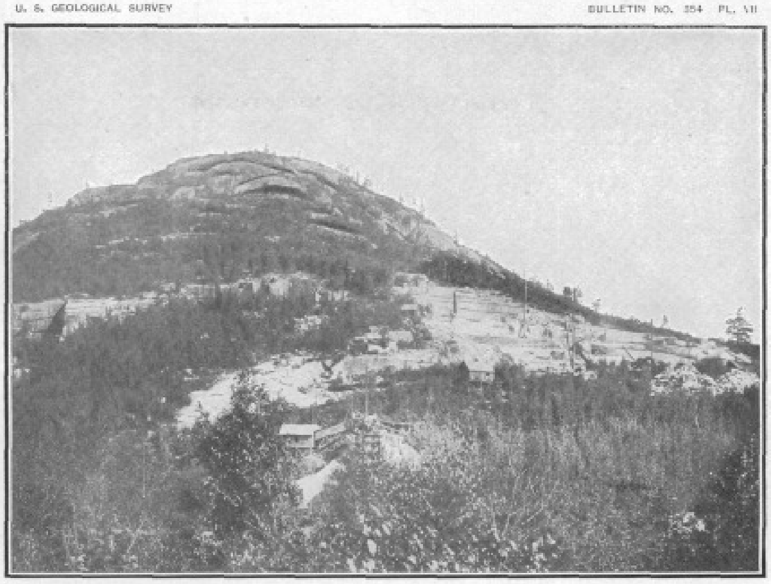

A. REDSTONE LEDGE, CONWAY, N. H., LOOKING N. 30° E.

Showing, on the left, quarry in green granite and, on the right, one in pink granite; also the sheets dipping away from axis of hill. Boundary between the two granites runs diagonally up the hill. In foreground is stone crusher for waste.

Timber was not the only resource of interest in the Whites. When the P&O RR reached Conway in 1871, it laid track at the base of Rattlesnake Mountain and made use of the large granite boulders found there, which could be easily split to provide dimension stone for the railroad. The quality of the Conway Granite was soon recognized and the Redstone Quarry opened in 1886. Much of the product was shipped elsewhere for use as paving stones, but the railroad itself had an enormous need for granite blocks for constructing abutments and also for architectural use in building many of the grand stations along its lines. However, the market for Redstone products was more extensive than this. Grant's Tomb in New York, the National Archives building in Washington, and the George Washington Memorial Masonic Temple in Alexandria, VA were built mostly of Conway pink granite. [A 3,000-foot borehole was drilled in 1975 at the site of the quarry in an effort to assess the heat flow and geothermal energy potential of the Conway Granite. This remains the deepest hole ever drilled in the state].

In the early 1900's, the automobile began to displace the railroads as the preferred method of travel and the railroads and grand hotels went into slow decline. Tourism continued to thrive even as these venerable institutions faded from the scene. The first steam-powered automobile climbed to the summit of Mt. Washington in 1899 on what was to become the Mt. Washington Auto Road. The first gasoline-powered car "summited" in 1902. [Today, car bumpers bearing "This Car Climbed Mt. Washington" stickers are a common sight. Events sponsoring contests of various forms of human-powered locomotion are also a regular occurrence on the Auto Road]. Echoing the turnpike statistics cited by Eliphalet Merrill one hundred years earlier, the State Highway Department reported in 1919 that:

"*The system of trunk line highways alone comprises 1,300 miles of which 909 have been already built and 391 are about to be built. Most of New Hampshire's roads are gravel and no better riding surface has been designed than a substantial, smooth gravel road. There are large deposits of gravel in the state that have been made available for this purpose, which fulfill all the requirements and have the advantage of being the cheapest road material under the conditions*." (New Hampshire: A pamphlet concerning the activities of certain of the State Departments, Concord, September 15, 1919, p. 22)

The Civilian Conservation Corps (CCC), with camps throughout the White Mountains, added to this network during the 1930's. Tripoli Road was built from North Woodstock to Waterville Valley and an 8-mile segment of US Route 3 in Pittsburg that reached the Quebec border, creating the state's first and only port of entry to Canada in 1940. Work crews from Camp Peabody worked to clear blow-downs from the Mount Washington Auto Road after the Hurricane of 1938. Altogether, CCC enrollees built 277 miles of new roads and truck trails across the state. At that time, no improved road existed between the former logging camp site at Passaconaway on the Swift River and the village of Lincoln on the East Branch of the Pemigewasset River. The gap was closed in the 1950's with the construction of the Kancamagus Highway. The Franconia Notch Parkway became part of the interstate highway system [I-93] in 1988 and is the only stretch of interstate in the country constructed without a median strip.

The forests of the White Mountains have regenerated and geologists have found plenty to interest them in the time since Charles T. Jackson looked at the rocks on the top of Mt. Washington and yawned. Scientists still inhabit these storied mountains as the Mt. Washington Observatory continues the legacy of discovery that State Geologist Charles H. Hitchcock began during the winter of in 1870-71 when he maintained a meteorological station on the summit, the first high-mountain observatory in the United States.

WHITE MOUNTAIN NOTCH.

New Hampshire Geologic History

Notes: The recently published book, "The Geology of New Hampshire's White Mountains" (Durand Press, 2013) is highly recommended as an excellent source of additional information.

Specific rock units that are traversed by the route of the field trip are printed in bold where first introduced in the narrative below.

The role of plate tectonic theory in enabling geologists to piece together the geologic history of New Hampshire cannot be overestimated. The entire rock record, as documented by scores of geologists over the past 175 years, can be understood in the context of a model of crustal evolution whereby new plates and crust are created by extension at divergent boundaries while other plates are being enlarged by accretion and/or consumed by subduction at convergent boundaries. Ocean basins open and accumulate sediment eroded from continental highlands and deposited by volcanic activity associated with island arcs and continental plate margins. Basins close as plates collide, culminating in a new episode of mountain building. Deformation and metamorphism are pervasive during orogenesis, accompanied by partial melting of crust and extensive formation of granites together with intense volcanic and seismic activity. Plates coalesce by accretion only to be split apart to form new configurations as rifting is renewed in response to dynamic changes in the driving forces. Tectonic lithofacies mapping provides the key to plate reconstruction.

The bedrock of New Hampshire is the product of a succession of tectonic events that occurred along the continental margin of the Laurentian plate beginning with the Taconic orogeny in middle Ordovician time. Collision and accretion added to the continental mass, processes that were repeated during the Siliurian Salinic and mid-Devonian Acadian orogenies, deforming the older rocks and producing new ones. The culminating Permian Alleghanian Orogeny created the supercontinent Pangea, a "backbone" of which is the Appalachian Mountain chain. The northern portion of that chain includes the White Mountains of New Hampshire. Early Mesozoic rifting associated with the breakup of the Pangean supercontinent is evidenced by the presence of north to northeast trending normal faults and basalt dikes and the intrusion of large composite igneous bodies, ring dikes, and associated explosive volcanic rocks.

The lithotectonic associations of the oldest rocks in New Hampshire remain uncertain despite considerable study. The Massabesic Gneiss Complex of migmatitic gneisses is Late Proterozoic in age. The main body of Massabesic trends northeasterly across southeastern New Hampshire, aligned with the pronounced regional structural grain and bounded by the Silurian Berwick Formation of the Merrimack belt to the southeast and rocks of the Central Maine terrane to the northwest. Dorais et al. (2012) now characterize the Massabesic as an inlier of the Gander terrane which is more prominently exposed farther northeast in the Maritimes of Canada where it was first recognized. Earlier interpretations include the Massabesic as part of the more outboard Avalon terrane of southeastern Massachusetts and Rhode Island.

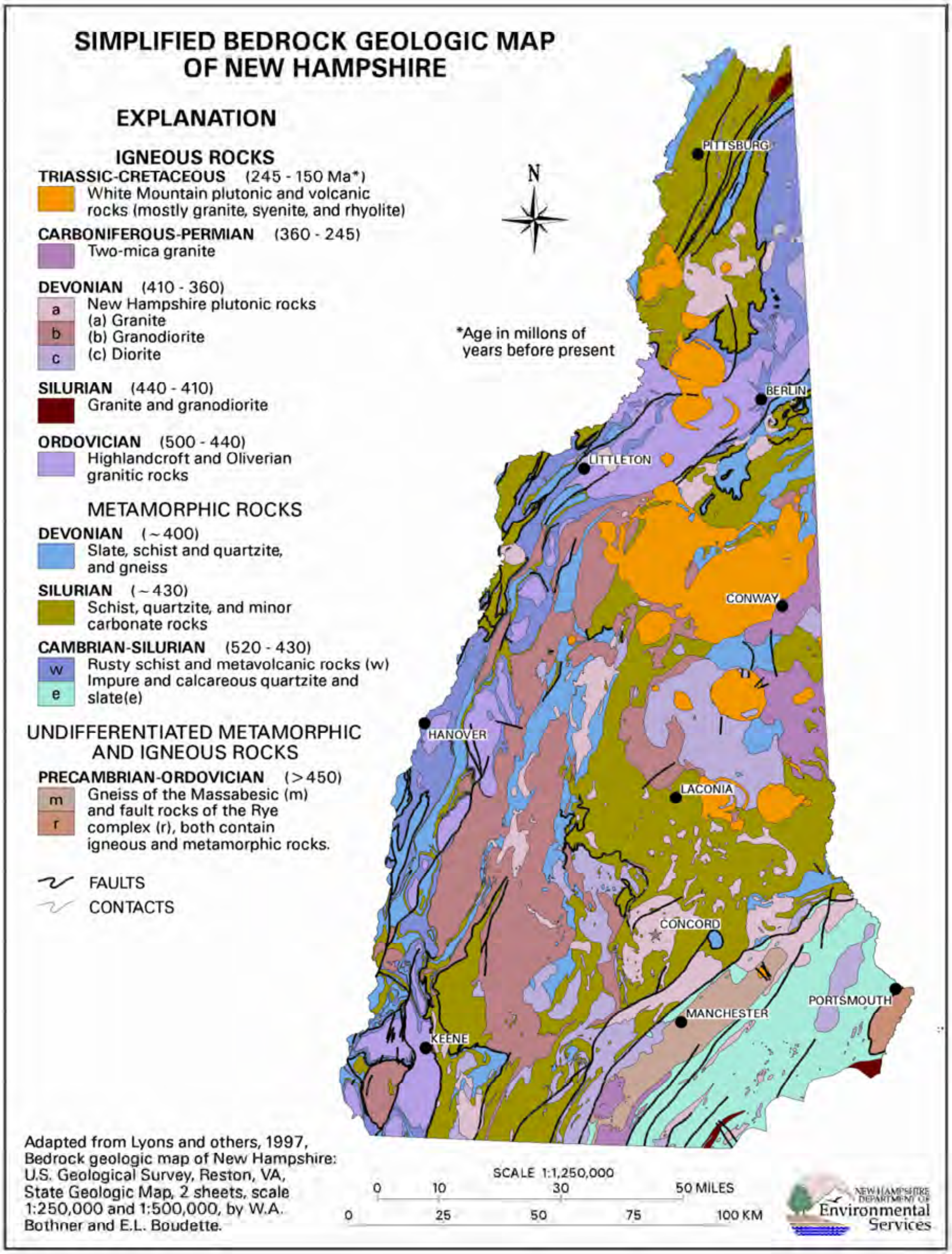

Figure 1 – *Simplified Bedrock Geologic Map of New Hampshire*

Gander and Avalon originated as several elongated segments that rifted from the Gondwanan plate when it was on the far side of the Iapetus Ocean from the Laurentian plate. The geologic history of New Hampshire during the Paleozoic is defined by the ultimate closing of this basin, bringing segments of the Gondwanan plate (Gander, then Avalon, finally Meguma), sediments from their intervening ocean basins and a succession of volcanic island arcs associated with subduction zones into contact with the evolving Laurentian plate margin. All of these events are believed to have occurred while the plates where located near the equator.

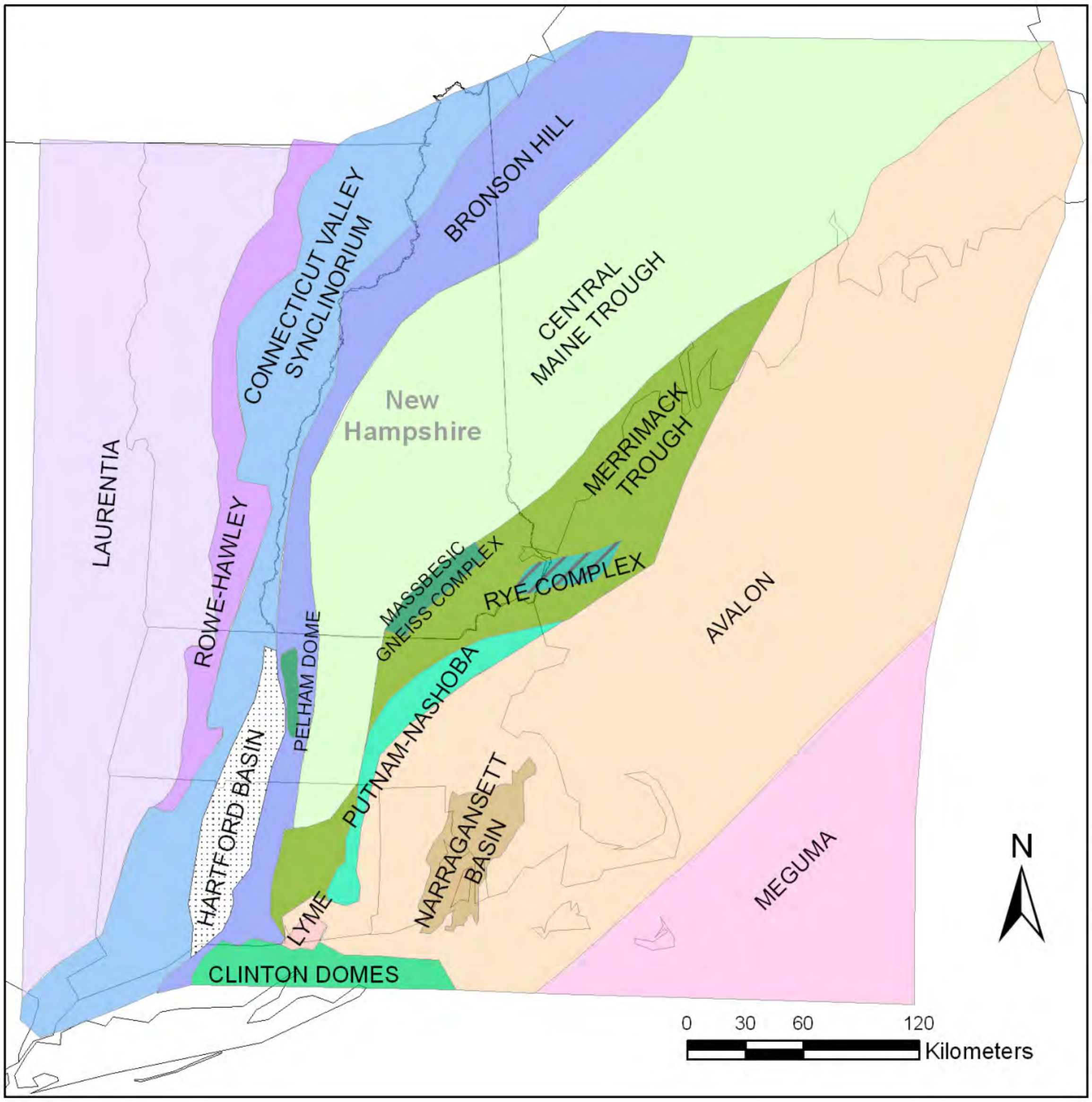

Figure 2 - *Generalized map of lithotectonic terranes in New England; modified from Figure 1 of Dorais, et al. 2012. Note: Plutonic bodies within the Merrimack Trough are not shown.*

Laurentia grew by accretion during the Ordovician with first the collision of the Shelburne Falls volcanic arc, marking the beginning of the Taconic Orogeny about 480 Ma, and then the Bronson Hill arc about 450 Ma. The main mountain building that resulted, the formation of the Taconic Mountains, was to the west of present-day New Hampshire in eastern New York and western Massachusetts and Vermont. In New Hampshire, the Ordovician rocks occupy a belt along the border with Vermont that coincides with the Bronson Hill volcanic arc and includes an assemblage of sediments, volcanics, and igneous intrusions having an affinity with the Gander plate. These rocks represent the oldest rocks in the White Mountains, among which the metamorphosed sedimentary rocks, mostly shales and sandstones, of the Albee Group (correlated with the Dead River formation of Lyons et al. 1997) of Late Cambrian to Early Ordovician age are the very oldest. This characterization derives from recent mapping within the Connecticut River valley between New Hampshire and Vermont in conjunction with compilation of the new statewide bedrock geologic map for Vermont (Ratcliffe et al. 2011). Stratigraphic nomenclature is in the process of being redefined as a result of this more recent work.

The **Ammonoosuc Volcanics (Oam)** unconformably overlie the Albee and record the eruptions of basaltic and rhyolitic material within the Bronson Hill island arc. Rusty weathering metamorphosed black shales of the Partridge formation lie conformably above the Ammonoosuc Volcanics. This sequence of Ordovician metavolcanics and metasedimentary rocks is intruded by the Oliverian Domes of the **Oliverian Plutonic Suite (Oo1b)** and the Highlandcroft Plutonic Suite that are interpreted to be the magma chambers for the Bronson Hill island arc volcanoes. These granitoid bodies are exposed as elongated elliptical masses enveloped by the Ammonoosuc Volcanics and trending north to northeast along the structural axis of the Bronson Hill Anticlinorium.

At the conclusion of the last of the Taconic compressional events during the Middle to Upper Ordovician, two ocean basins existed off the margin of the Laurentian plate. The Central Maine basin was proximal to the recently accreted Bronson Hill volcanic arc while the Merrimack basin was farther to the east where it was bounded by the Coastal Maine arc. In latest Ordovician time, both basins began accumulating significant deposits of shale, siltstone, and sandstone eroded from the adjacent landmasses. Deposition ceased in the Merrimack basin at the end of the Middle Silurian, but continued through Early Devonian time in the Central Maine basin.

Rocks of the Merrimack basin include the Kittery Formation and Eliot Formation that today underlie the lowlands in the seacoast region of the state. The Kittery is notable for its preservation of primary sedimentary structures such as graded bedding, cross-bedding, and small-scale channel cut and fill structures. The oldest stratified rocks exposed in this region are the mylonitinized metasedimentary rocks of the Rye Complex (presumably Ordovician or older) that occupy the immediate coastal zone. They are separated from the Kittery and Eliot formations to the northwest by the Portsmouth fault. Because of this contact relationship, the basin where the Rye sediments originally accumulated cannot be readily determined (Hussey et al. 2008).

The source of sediments for the Silurian-age Berwick Formation has been attributed to erosion of the Bronson Hill terrane with deposition occurring in the Central Maine basin. Historically, however, the Berwick Formation has been included within the Merrimack Group and the

Rangeley Formation (Srl and Sru) is identified as the oldest unit of Silurian age (430 Ma) within the Central Maine stratigraphy of New Hampshire. The Rangeley Formation is characterized as variably bedded deepwater deposits (pelitic and psammitic gneiss, schist, and granofels) derived from sediment eroded from the Bronson Hill terrane. Eusden et al. (2013) report the occurrence of olistrostromes within the Rangeley Formation throughout the Presidential Range as evidence for active subduction during the Early Silurian. More quiescent conditions prevailed during deposition of the quartzites of the Perry Mountain Formation (Spm), followed by the rusty-weathering, sulfide-bearing schists of the Smalls Falls Formation (Ssf). The contrast between units records a transition to an euxinic depositional environment as circulation within the closing ocean basin became increasingly restricted. More open circulation was resumed during deposition of the calc-silicates constituting the Madrid Formation (Sm), the youngest sedimentary unit in the Silurian sequence.

Sedimentation in the Central Maine basin continued into the Early Devonian with the deposition of the **Littleton Formation (Dl)** that is believed to have been derived from sources to the east and deposited from east to west. The change in presumptive sediment source has been attributed to the approach of the Avalon plate from the present-day east. The metapelites and metawackes with interlayers of rocks of volcanic origin that constitute the Littleton Formation lie conformably above the older Madrid Formation. Rates of sedimentation and intensity of volcanic activity increased as the collision progressed with subduction of the Avalon plate, initiating the most pronounced mountain-building episode recorded in New Hampshire, the Acadian Orogeny.

The New Hampshire Plutonic Suite of Devonian-age synkinematic and postkinematic granites and granitoids is related to the Acadian Orogeny. The Concord Granite, Spaulding Tonalite, Winnipesaukee Tonalite, **Bethlehem Granondiorite (Db2b)**, and **Kinsman Granodiorite (Dk2x)** are included within this group of widely distributed plutonic bodies. Their origin has been linked to the significant thickening of continental crust inboard of the margin of the composite Laurentian terrane as the continental portion of the Avalon plate was subducted beneath it. The increased crustal thickness drove up temperatures sufficiently to cause the rocks to melt, forming magmas that rose through the overlying crust and crystallized as intrusive bodies. Deformation and metamorphism associated with the Acadian Orogeny strongly overprints pre-existing evidence, making the pre-Acadian geologic history far more challenging to decipher.

During Late Devonian to Early Carboniferous time, a period of crustal instability and magmatism referred to as Neoacadian, less dense rocks of the Oliverian Domes rose upward as remobilized solids, displacing the more dense Ammonoosuc Volcanics and other overlying Silurian metasedimentary formations. The contact relationships create a map pattern where the older gneissic domes appear as “islands” surrounded by younger metasediments, hence the term “mantled gneiss domes.” A number of light gray to white, fine-grained two-mica granites were also emplaced during this same period, cross-cutting all the Acadian metamorphic rocks and the Acadian folds and faults. Named bodies include the Alderbrook, **Bretton Woods**, Bickford and Peabody granites (unit **D1m** of Lyons et al., 1997). The Permian-age Sebago pluton represents the last magmatic episode in the White Mountains during the Paleozoic.

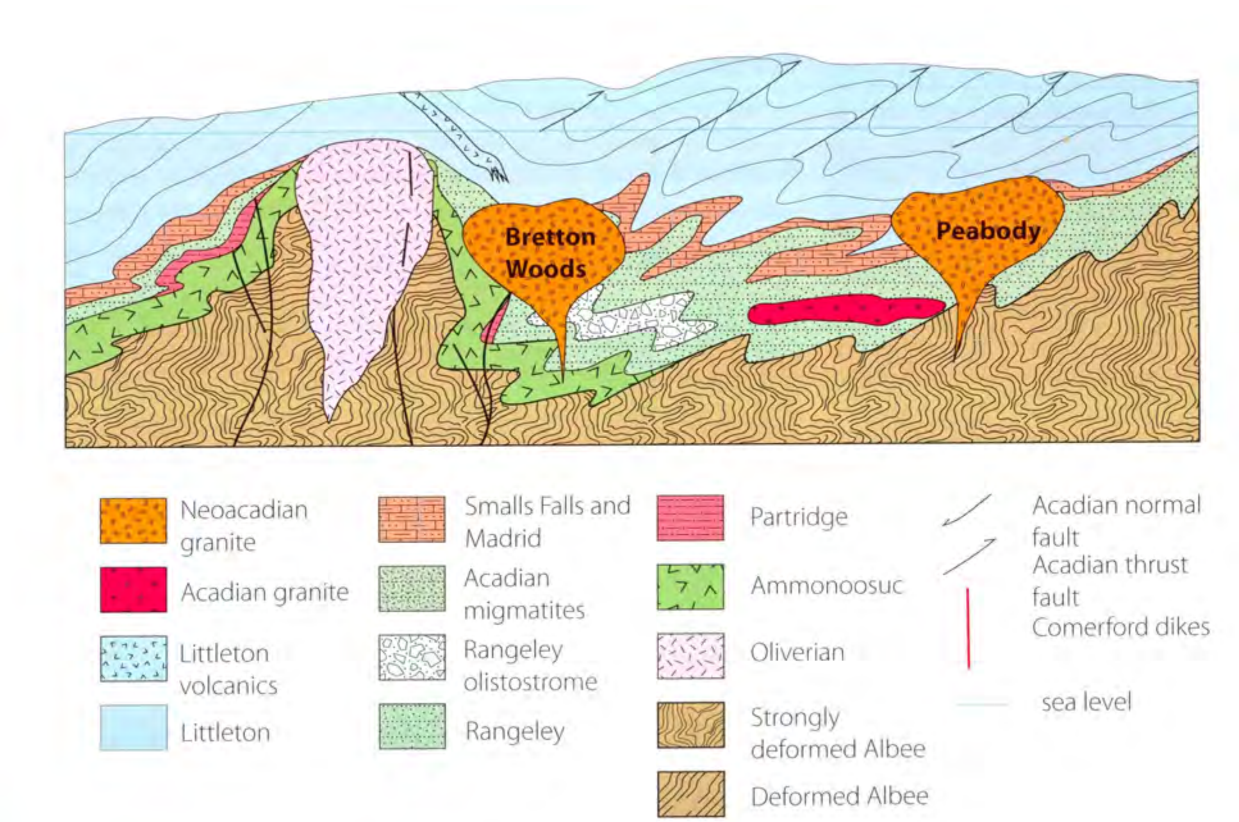

Figure 3 - *The figure above is an idealized cross-section through the complete Late Cambrian to Early Devonian stratigraphy within the White Mountains (Figure reproduced from "The Geology of New Hampshire's White Mountains", Durand Press, 2013, with permission of the authors.)*

By the end of the Paleozoic the composite Laurentian terrane was united with the Gondwanan plate to create the supercontinent Pangea as the Iapetus Ocean finally closed. This configuration was short-lived. Changes in the underlying geodynamic forces initiated large-scale fracturing and rifting during Late Permian through Early Triassic times (240 and 210 million years ago). The new supercontinent began to break up. Extension of the crust resulted in a series of north- to northeast-oriented fault-bounded basins along the margin of the proto-North American plate. The Ammonoosuc fault, located along the eastern boundary of the Connecticut River valley in New Hampshire, has been identified with this episode of Mesozoic rifting. To the south from northern Massachusetts through central Connecticut, the Northfield – Hartford basins preserve huge volumes of Triassic-Jurassic arkosic sandstones and abundant basaltic volcanic rocks that reflect failed continental extension. In addition, swarms of northeast-trending basalt dikes along coastal New England, and more sporadically (forest cover) throughout inland New England, provide further evidence.

Crustal extension has been proposed as a significant factor in explaining the active magmatism that occurred in parts of east-central and northern New Hampshire (as well as elsewhere in New England and Quebec) during the Triassic, Jurassic, and Cretaceous time periods. The rocks associated with this activity in New Hampshire are represented by the White Mountain Plutonic-

Volcanic Suite, mostly identified with Early Jurassic granite plutons. These overlapping centers of magmatic activity stand out boldly on the simplified bedrock geologic map (Figure 1) as golden yellow areas with roughly circular outlines.

Some of these intrusive bodies with their associated arcuate ring dike geometries provide a revealing view into the deep magmatic "plumbing" systems that sustained active volcanism during the Jurassic. They have long attracted the interest of geologists. The ring dike complexes in the Ossipee Mountains and Belknap Mountains near Lake Winnipesaukee are regarded as classic localities and continue to be favorite destinations for geologic field trips. Other notable examples are found in the White Mountain region, including the Pliny Range immediately north of the field trip route and several localities that are even closer. Indeed the entire White Mountains composite batholith can be viewed as a series of overlapping caldera complexes. Some of the most notable are defined by the syenite ring dike opposite the Cannon pluton, at **Lower Falls** on the Swift River near the west side of Moat Mountain, Mt. Tripyramid, Hart Ledge, and Jackson Falls; we drive through most of these on the trip.

The model for the genesis of ring dike complexes invokes an initial doming of overlying rocks in response to the intrusion of a significant volume of magma at a relatively shallow depth. The associated stresses result in the formation of ring-shape fractures that encircle the magma chamber below them and often become outlets for minor eruptions. This stage is a prelude to major explosive eruptions that expel great quantities of hot ash, gases, and volcanic debris and partially empty the magma chamber in the process. The accumulation of volcanic ejecta that blankets the surrounding landscape may be preserved as thick, extensive layers of pyroclasic rocks. Because the volcanic edifice is no longer fully supported, the roof of the magma chamber collapses along the ring fractures to form a steep-sided caldera. The caldera is subsequently filled by the eruption of new ash flows from the ring fractures. The **Moat Volcanics (Jmv)**, underlying the three peaks of the Moat Range and also Big Attitash Mountain to the west of North Conway as well as Kearsarge North and Bartlett Mountain to the northeast, originated in this manner. Lithologies range from fine grained tuffs to coarse breccias. Volcanic activity eventually subsides and ring dikes are formed as magma within the ring fractures cools and solidifies, in this case forming **Albany Porphyritic Quartz Syenite** (unit **J4hx** of Lyons et al., 1997). This distinctive unit has an overall pink to gray appearance with larger crystals of quartz and feldspar more or less uniformly distributed within a fine-grained matrix. The sequence may conclude with the intrusion of new granitic magma into the assemblage of older volcanic rocks, creating discordant contacts between resulting rock units.

The Jurassic-age **Conway Granite (Jc1b)** and **Mt. Osceola Granite (Jo1b)** are widely exposed throughout the White Mountains and were largely derived from resurgent magma sources as described above. The White Mountain Batholith is a composite of the magma bodies from which these two dominant plutonic rocks were formed. The mineralogy of the two granites is distinctly different but distinguishing them in the field can be challenging because each is subject to variations in its composition and appearance. The typical Conway Granite is a medium- to coarse-grained, pink, biotite two-feldspar (pink orthoclase gives it its color) granite, whereas the Mt. Osceola Granite is often greenish (Billings described it as "dirty" green) and contains only one feldspar plus amphibole and pyroxene.

A period of relative inactivity followed the intense magmatism and volcanism that characterized the Jurassic, lasting from 130 to 108 million years ago. One final episode of magmatic activity during the Cretaceous resulted in the intrusion of roughly east-west striking basaltic dike swarms and a few minor plutons of the same age. In the absence of significant geologic events since that time, the inexorable forces of weathering and erosion have outpaced gradual uplift to exhume the geologic record that has been described in this section. The Pleistocene Epoch added a final flourish to the story.

New Hampshire was over-ridden multiple times by continental ice sheets as the climate cooled beginning approximately 2.6 million years ago. Evidence for each episode of glacial advance and retreat was largely erased by the next one. Today we are left mostly with the remnants of deposits that resulted from the last event, the Wisconsinan stage of the Laurentide ice sheet that reached its maximum extent about 20,000 years ago. The ice margin melted back through coastal and southern New Hampshire beginning about 16,000 years ago; the northernmost landscape was virtually ice-free by about 11,000 years ago.

Upland areas were left with a relatively thin covering of glacial till, varying in density from a compact silt- and clay-rich lodgment till to a sandier, and more permeable ablation till. In places, till from the older Illinoian glaciation is preserved underneath the Wisconsinan deposits, but these "two till" exposures are relatively rare. The stony and nutrient-poor nature of the soils that developed from these parent materials made cultivation difficult and relatively unproductive. The extensive network of stonewalls that bound the fields and pastures of the early hill farms, now largely abandoned and reforested, are a testament to the hard labor involved in clearing and working the land.

The numerous large boulders were formidable obstacles, but also a source of curiosity. Attempts to explain their presence eventually led to acceptance of their glacial origins, once it was recognized that many boulders differed from the underlying bedrock and must have been transported significant distances from points north where outcrops of similar rock type could be seen. Perhaps the most famous glacial erratic in New Hampshire is the Madison Boulder which is composed of Conway Granite, resting on Concord Granite.

Many of the lowlands filled with stratified meltwater deposits that created broad, flat intervales that were far more fertile and easier to farm. An extensive stratified-drift aquifer in the Saco River valley supplies the 4 production wells operated by the North Conway Water Precinct, the source of water for the North Conway Grand Hotel. Figure 4 (next page) provides an overview of the extent of these valley-fill deposits in the area of the field trip based on 1:24,000-aquifer mapping completed in the mid-1990's. In the White Mountains, these areas contrast sharply with the narrow, steep-walled "notches" that were scoured through the mountain fronts once they had been over-ridden by the continental ice sheets. The field trip route takes us through Franconia Notch and Crawford Notch, both famous for their dramatic scenery which is a legacy of their geologic history. Stops 3 and Stop 4 will build on the story of the glacial geology of the White Mountains that has been introduced here.

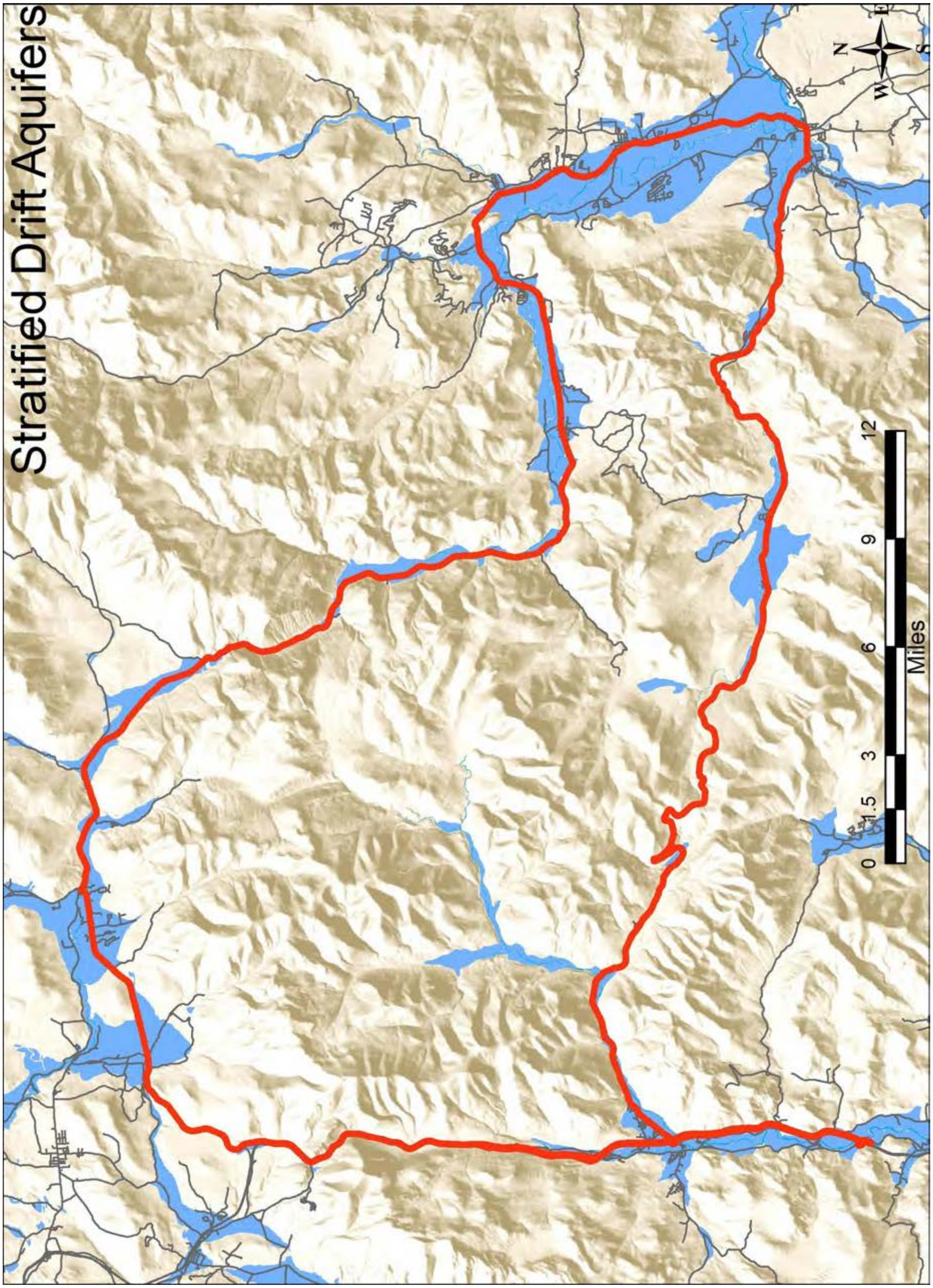

Figure 4 - *Stratified Drift Aquifers in the White Mountain Region*

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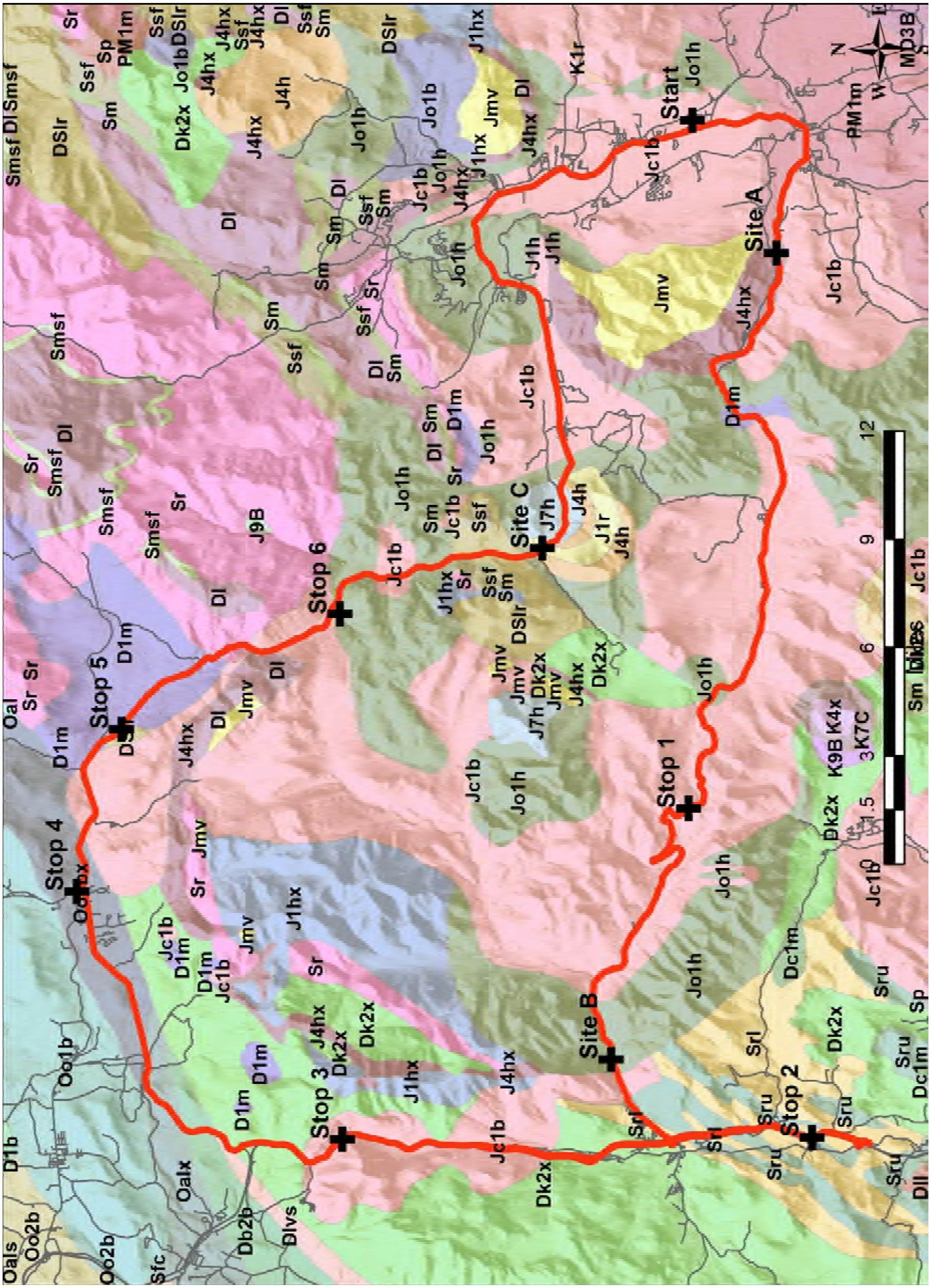
Roadside Geology of the 2013 HGS Field Trip

The following text and maps are provided as an accompaniment to the detailed narratives for each of the stops along the field trip route, presented sequentially following this section. The map on page 31 provides an overview of the bedrock geology traversed along our 120-mile loop through the White Mountains, clockwise from North Conway and back. The explanation for all the corresponding map units appears on page 32. Each of the subsequent 6 maps highlights the bedrock geology and other points of interest along a segment of the route centered on one of the field trip stops.

Page 31: Overview map showing filed trip travel route, field trip stops and drive-by sites.

Page 32: Explanation of bedrock unit symbols shown on field trip map.

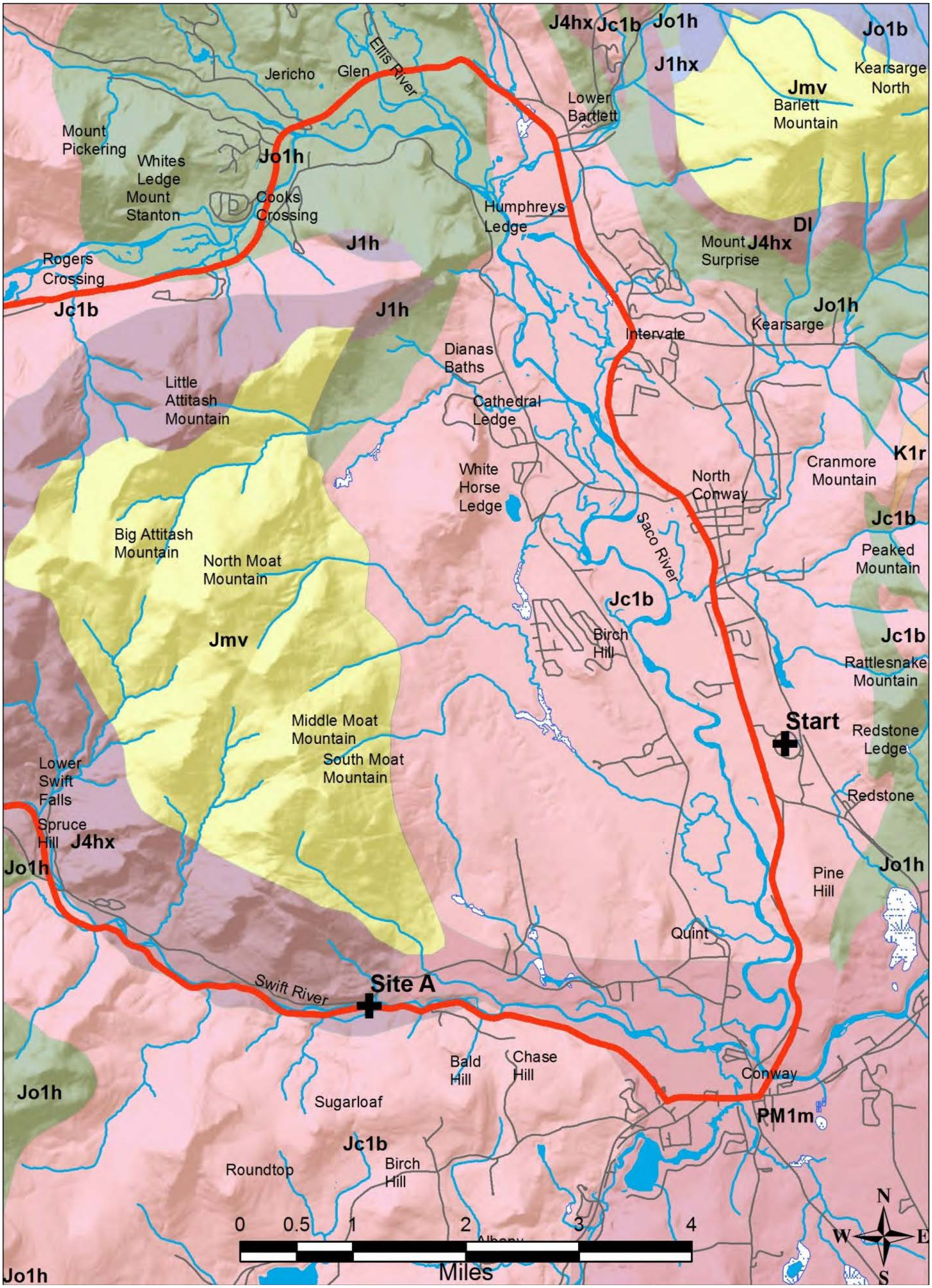

Start
Site A
Site B
Site C
Stop 1
Stop 2
Stop 3
Stop 4
Stop 5
Stop 6
N
0 1.5 3 6 9 12
Miles

Legend

Bedrock Geology

D1b>Pink equigranular biotite granite (Late Devonian)
D1m>Two-mica granite of northern and southeastern New Hampshire
D3Ab>Biotite tonalite
DSlr>Migmatitic rocks (Devonian and Silurian)
Db2b>Bethlehem Granodiorite (Early Devonian) (Bethlehem Gneiss of Billings, 1955)
Dc1m>Concord Granite (Late Devonian)
Dk2x>Kinsman Granodiorite (Early Devonian) (Kinsman Quartz Monzonite of Billings, 1955)
Dl>Littleton Formation, undivided (Lower Devonian; Siegenian)
Dlc>Littleton Formation
Dll>Littleton Formation
Dlu>Littleton Formation
Dlv>Littleton Formation
Dlvs>Littleton Formation
Ds1-6>Spaulding Tonalite (Early Devonian) (Spaulding Quartz Diorite of Fowler-Billings, 1949)
Dw3A>Winnipesaukee Tonalite (Early Devonian) (Winnipesaukee Quartz Diorite of Billings, 1955)
J1h>Hastingsite granite
J1hx>Granite porphyry (Early Jurassic)
J1r>Mesoperthitic granite
J4h>Hornblende or alkalic amphibole quartz syenite
J4hx>Porphyritic hornblende or alkalic amphibole quartz syenite
J4x>Quartz syenite
J7h>Hornblende (or alkalic amphibole) syenite
J7x>Porphyritic syenite
J9B>Gabbro
Jc1b>Conway Granite (Late? and Middle Jurassic)
Jmv>Moat Volcanics (Middle Jurassic?)
Jo1b>Mount Oscelola Granite (Middle and Early Jurassic)
Jo1h>Mount Oscelola Granite (Middle and Early Jurassic)
K1r>Medium-grained mesoperthitic granite containing riebeckite and (or) hastingsite
K4x>Large mesoperthitic phenocrysts in a pink to gray quartz syenite porphyry of "Albany type"
K7C>Gray augite-hornblende-biotite monzonite
K9B>Augite-hornblende-biotite gabbro
MD3B>Trondhjemite and abundant pegmatite
Oal>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oalb>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oalf>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oalg>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oals>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oalx>Ammonoosuc Volcanics (Upper and Middle Ordovician)
Oo1b>Biotite granite
Oo1bx>Porphyritic (alkalic feldspar) biotite granite
Oo1h>Hornblende-biotite granite
Oo2b>Biotite granodiorite
Oo2h>Hornblende granodiorite
Op>Partridge Formation, undivided (Upper to Middle Ordovician}
PM1m>Two-mica granite of the Sebago batholith and Effingham pluton of eastern New Hampshire
Sc>Clough Quartzite (Lower Silurian; upper Llandoverian)
Sf>Fitch Formation (Upper Silurian; Pridolian and Ludlovian)
Sfc>Fitch and Clough Formations, undivided (Silurian)
Sm>Madrid Formation (Upper Silurian?)
Smsf>Madrid and Smalls Falls Formations, undivided (Silurian)
Sp>Perry Mountain Formation, undivided (Middle? to Lower? Silurian)
Spr>Perry Mountain and Rangeley Formations, undivided (Silurian)
Sr>Rangeley Formation, undivided (Lower Silurian; Uandoverian)
Srl>Rangeley Formation
Sru>Rangeley Formation
Ssf>Smalls Falls Formation, undivided (Upper to Middle Silurian; Ludlovian and Wenlockian)

Page 34: **Hotel to Drive-By Site A (North Conway to Soil Nail Walls Kancamagus Highway) –** Travel south from conference center to Conway on NH Route 16. Beneath us is typically pink coarse-grained Jurassic Conway biotite two-feldspar granite. It also underlies Cranmore Mountain to the east and reappears at Redstone to the south. Between these two mountains are Peaked and Rattlesnake Mountains, underlain by coarse-grained, greenish Jurassic Osceola (Jo1h and Jh) amphibole or pyroxene-bearing one-feldspar granite. To the west is Moat Mountain, the type locality of the Jurassic Moat Volcanics (Jmv). They consist of rhylolite flows, ash deposits, and breccias, some of intermediate composition. As elsewhere in the state, the Moat Volcanics are in contact with ring dikes supporting a cauldron subsidence as the principal means of preservation; see also Bartlett Mountain to the north.

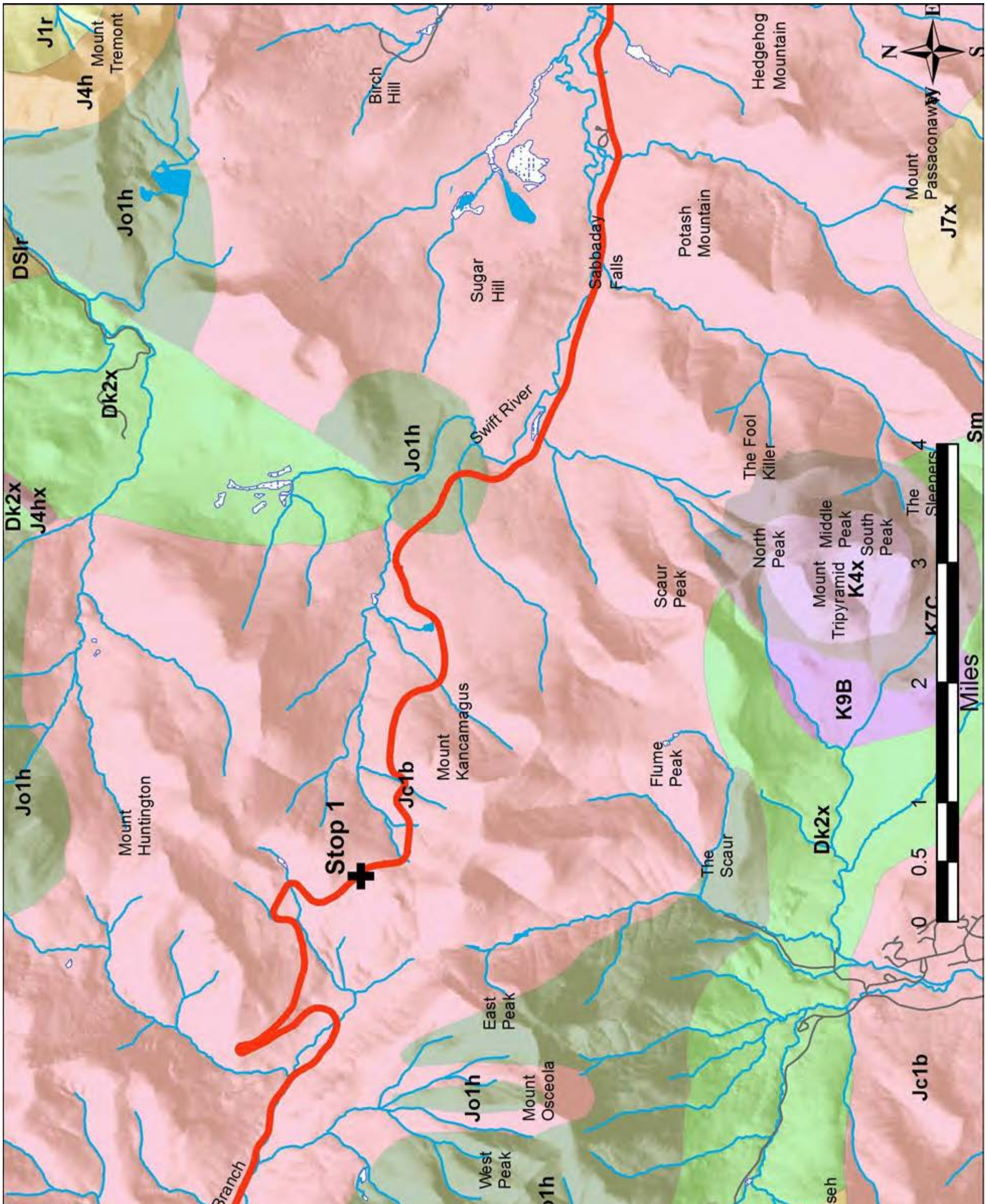


J4hx
Jc1b
Jo1h
Jo1b
J1hx
Jmv
Kearsarge North
Barlett Mountain
Jericho
Glen
Ellis River
Lower Bartlett
Mount Pickering
Whites Ledge
Mount Stanton
Cooks Crossing
Humphreys Ledge
J1h
Mount Surprise
Dl
Rogers Crossing
Intervale
Kearsarge
Dianas Baths
Little Attitash Mountain
Cathedral Ledge
White Horse Ledge
North Conway
Cranmore Mountain
K1r
Saco River
Big Attitash Mountain
North Moat Mountain
Peaked Mountain
Birch Hill
Rattlesnake Mountain
Start
Redstone Ledge
Middle Moat Mountain
South Moat Mountain
Lower Swift Falls
Redstone
Spruce Hill
Pine Hill
Quint
Site A
Swift River
Bald Hill
Chase Hill
Conway
PM1m
Sugarloaf
Roundtop
0
0.5
1
2
3
4
Miles
N
W
E
S

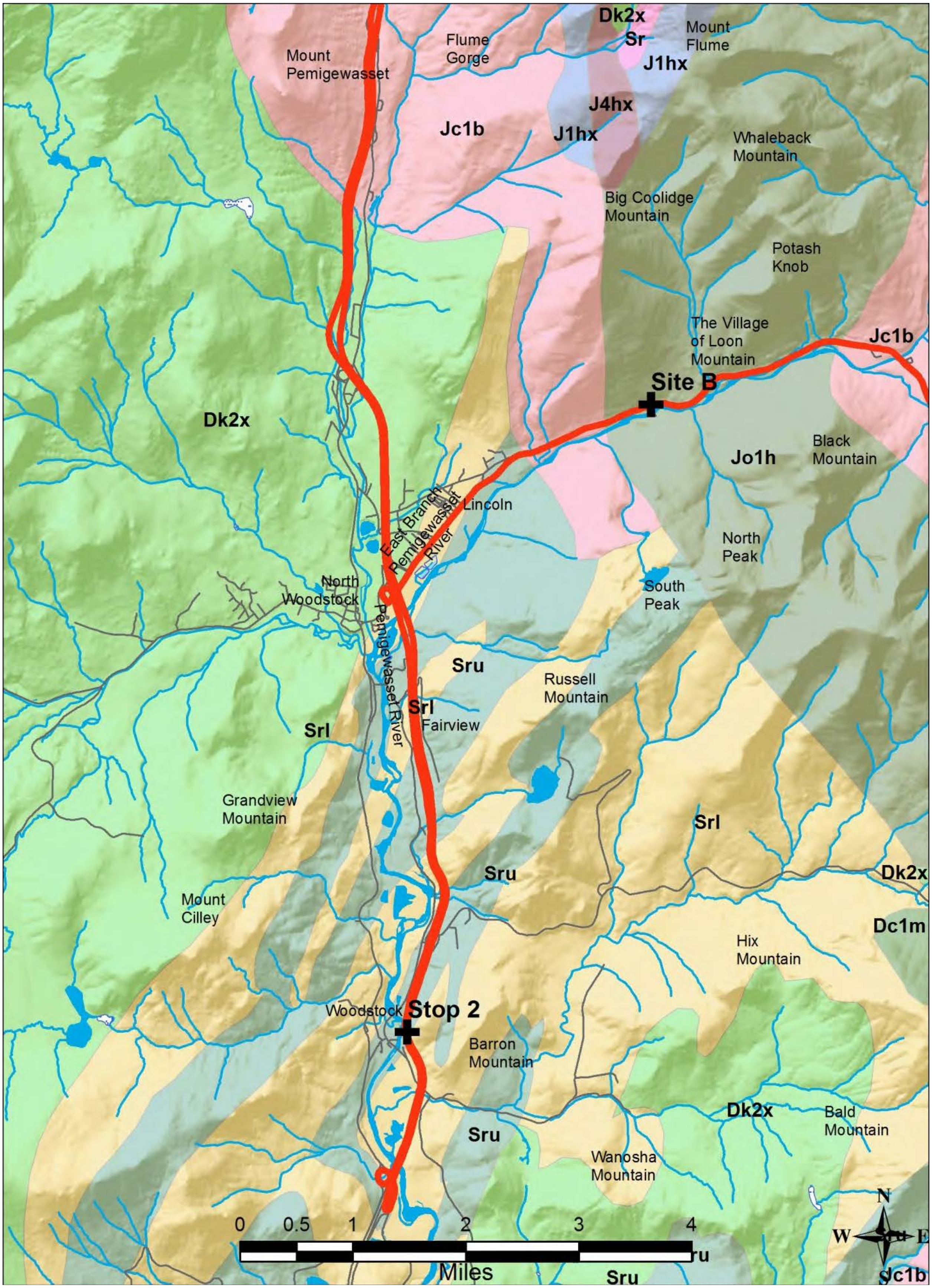
Page 36: **Drive-By Site A to Stop 1 (Kancamagus Highway Soil Nail Walls to Pemigewasset Scenic Overlook)** - We will see some of the Albany Porphyritic Quartz Syenite (J4hx) as we drive past Lower Falls of the Swift River heading west on the Kancamagus highway. We will also drive past Champney Falls trail leading to the top of Mt. Chocorua (Jo1h), and Sabbaday Falls trail (a must if you have a hour to spare!) that exposes typical Conway granite and a Jurassic (or younger?) diabase dike elegantly exploited by the stream. For the enthusiast this trail connects with another to the top of Mt. Tripyramid, underlain by one of the youngest central complexes with several ring dikes of syenite and an unusual and relatively rare gabbroic body, all of Cretaceous age.

At the Pemigewasset Scenic Overlook, Stop 1, note the beautiful coarse grained, well-jointed Conway biotite two-feldspar granite in nearby road cuts and Mt. Osceola, the type locality for that one-feldspar granite (of the same age) is visible to the southwest. Here we are near the center of the White Mountains composite batholith.

You should be able to see several landslide scars on the north flank of Mt. Osceola. Continue west and southwest crossing more Conway granite, then an outer belt (ring?) of Mt. Osceola Granite to about Lincoln where shortly w leave the White Mountains batholith.

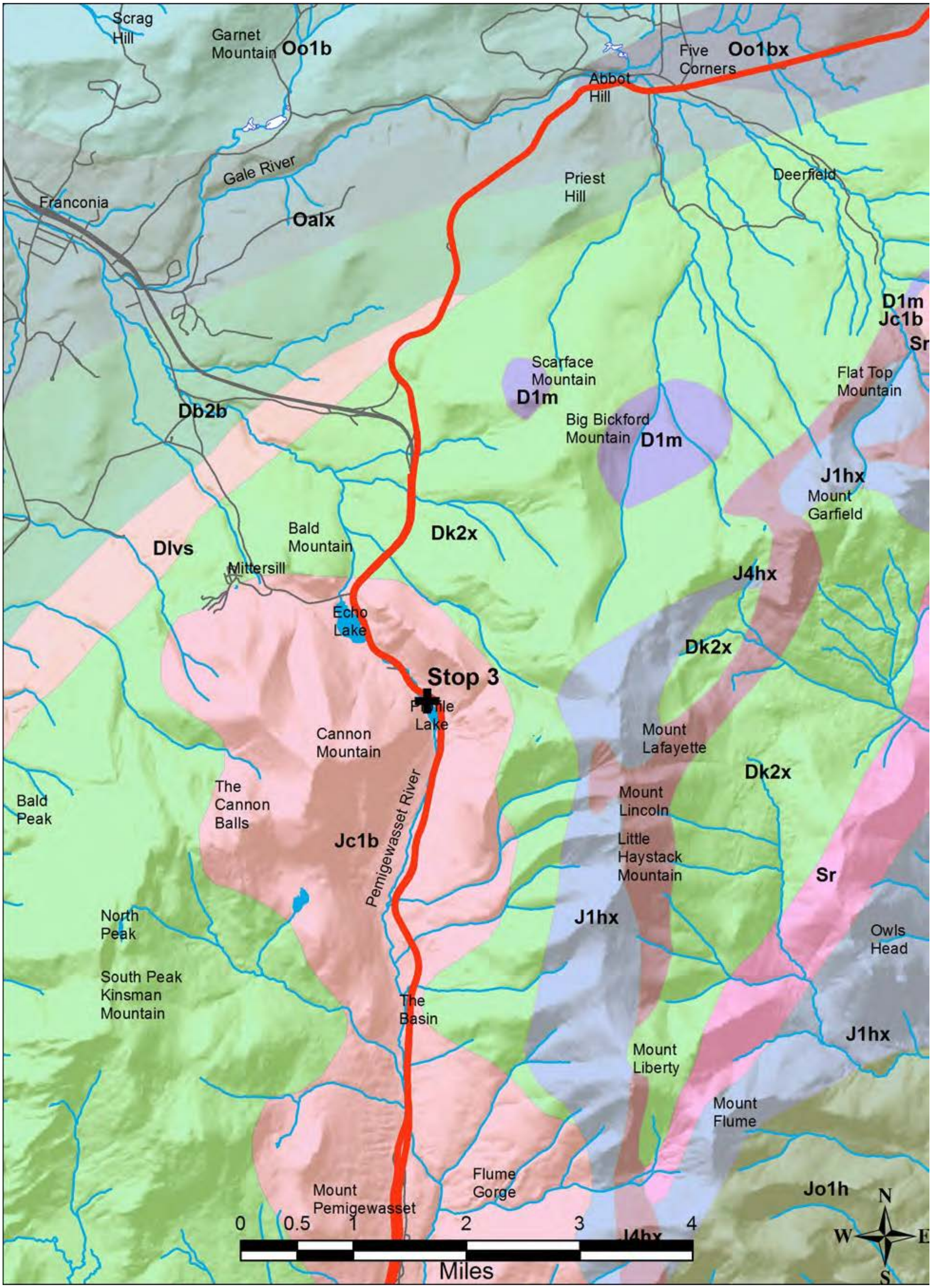

J1r
Mount Tremont
J4h
Jo1h
DSlr
Dk2x
Dk2x
J4hx
Birch Hill
Sugar Hill
Sabbaday Falls
Potash Mountain
Hedgehog Mountain
Mount Passaconaway
J7x
Swift River
Jo1h
The Fool Killer
North Peak
Middle Peak
South Peak
Mount Tripyramid
K4x
K9B
Sm
Scaur Peak
Mount Kancamagus
Jc1b
Stop 1
Jo1h
Mount Huntington
Flume Peak
The Scaur
Dk2x
East Peak
Jo1h
Mount Osceola
West Peak
Jo1h
Hancock Branch
Dc1m
Mount Tecumseh
Srl
Sru
Jc1b
0 0.5 1 2 3 4
Miles
N
S

Page 38: **Stop 1 to Drive-By Site B to Stop 2 (Pemigewasset Scenic Overlook to Loon Mountain Bridge to I-93 Barron Mountain) –** As we approach the center of Lincoln, we enter the "sea" of Silurian metasedimentary rocks of the Central Maine terrane. The rocks exposed along I-93 while heading south to Stop 2 are steeply dipping, high-grade schists of the Rangeley Formation. They are assigned to two members: Srl is the lower member and is typically a gray, thinly laminated pelitic rock with rare calc-silicate and coticules; and Sru, the upper (younger) member, is a rusty weathering metapelite and metasandstone with common calc-silicate pods and coticule. At the Barron Mountain rock cut, most of the rocks are coarse gray schists of Srl. In addition to the critical geologic engineering done to stabilize the cut, what mineral assemblage is obvious? A hand lens might be helpful. Note as we return north that the rocks just north of the North Woodstock are quite different from those seen so far… look for very coarse grained, dark gray porphyritic rocks of the Kinsman Granodiorite (Dk2x). The phenocrysts of alkali feldspar can reach nearly 10 cm and their cleavage often reflects sunlight easily as you drive-by… a 'sparkling' experience. These rocks of the Cardigan and Lincoln Mountain Plutons are also exposed through Kinsman Notch to the west and in an intrusive breccia at The Basin (see map on page 40 for location) should your travels allow such stops another time.

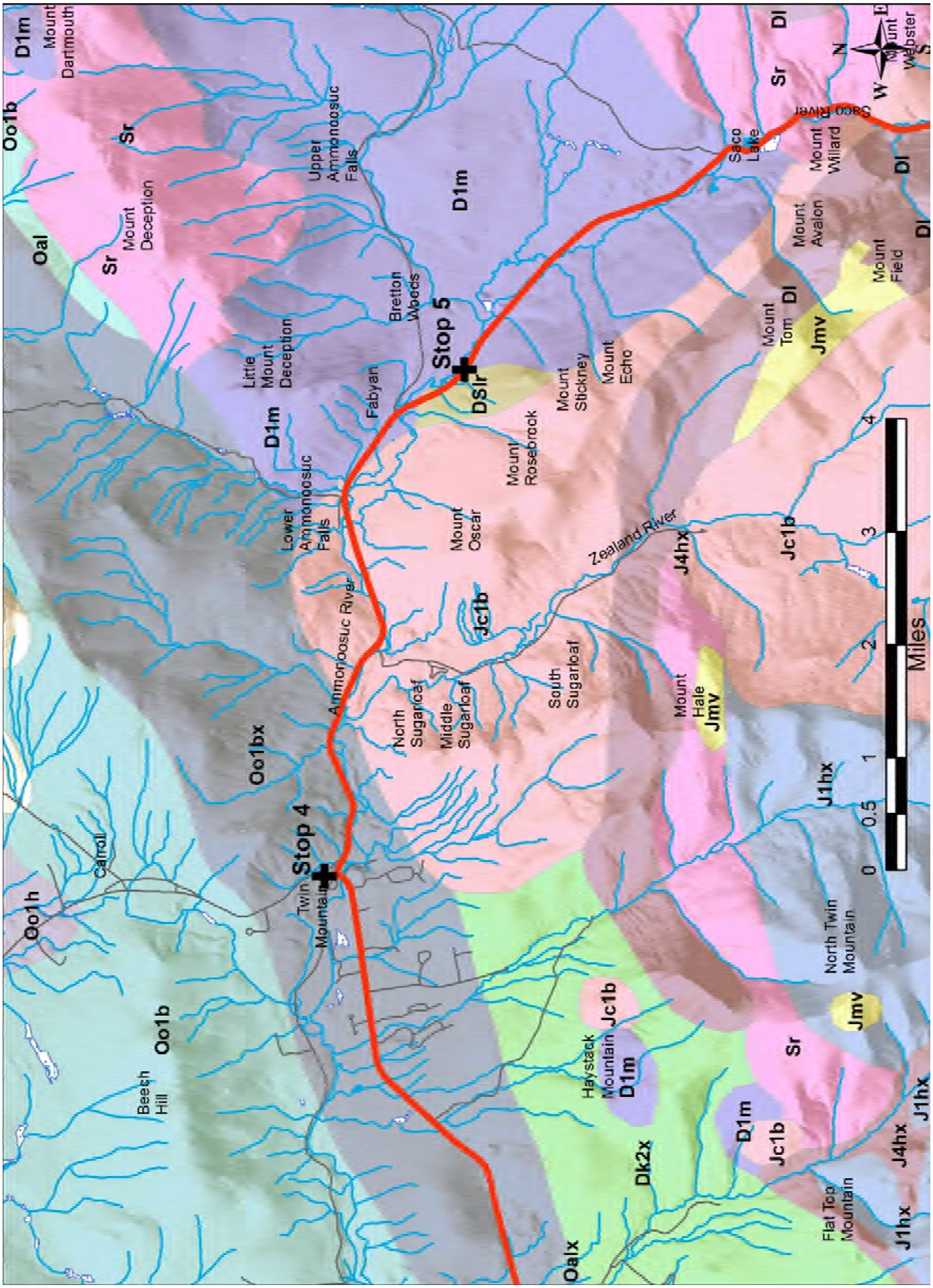

Dk2x
Sr
Mount Flume
Flume Gorge
Mount Pemigewasset
J1hx
J4hx
Jc1b
J1hx
Whaleback Mountain
Big Coolidge Mountain
Potash Knob
The Village of Loon Mountain
Jc1b
Site B
Dk2x
Jo1h
Black Mountain
Lincoln
East Branch Pemigewasset River
North Peak
South Peak
North Woodstock
Pemigewasset River
Sru
Russell Mountain
Srl
Fairview
Srl
Grandview Mountain
Srl
Sru
Dk2x
Mount Cilley
Dc1m
Hix Mountain
Woodstock
Stop 2
Barron Mountain
Dk2x
Bald Mountain
Sru
Wanosha Mountain
0 0.5 1 2 3 4
Miles
Sru
N
W
E
Jc1b

Page 40: **Stop 2 to Stop 3 (I-93 Barron Mountain Rock Cut to Old Man Historic Site) –** Both Kinsman and Conway intrusives are exposed along I-93 as we head north on our way to LUNCH. We pass through the central portions of two barely connected stocks of Conway granite – the Pemigewasset pluton and the Cannon Mountain pluton, the latter separated from the White Mountains composite batholith by a "screen" of Kinsman Granodiorite. Peaks to the west are underlain by Dk2x while those to the east are composed of "Mt. Lafayette" granite porphyry (J1hx) and porphyritic amphibole-bearing quartz syenite (J4hx), then a true screen of Dk2x and Sr, before entering the main batholith to the east. A path to the Old Man Plaza has numerous blocks of the now familiar Conway Granite along the way. Note also the landslide scars and landslide deposits at the south end of Profile Lake.

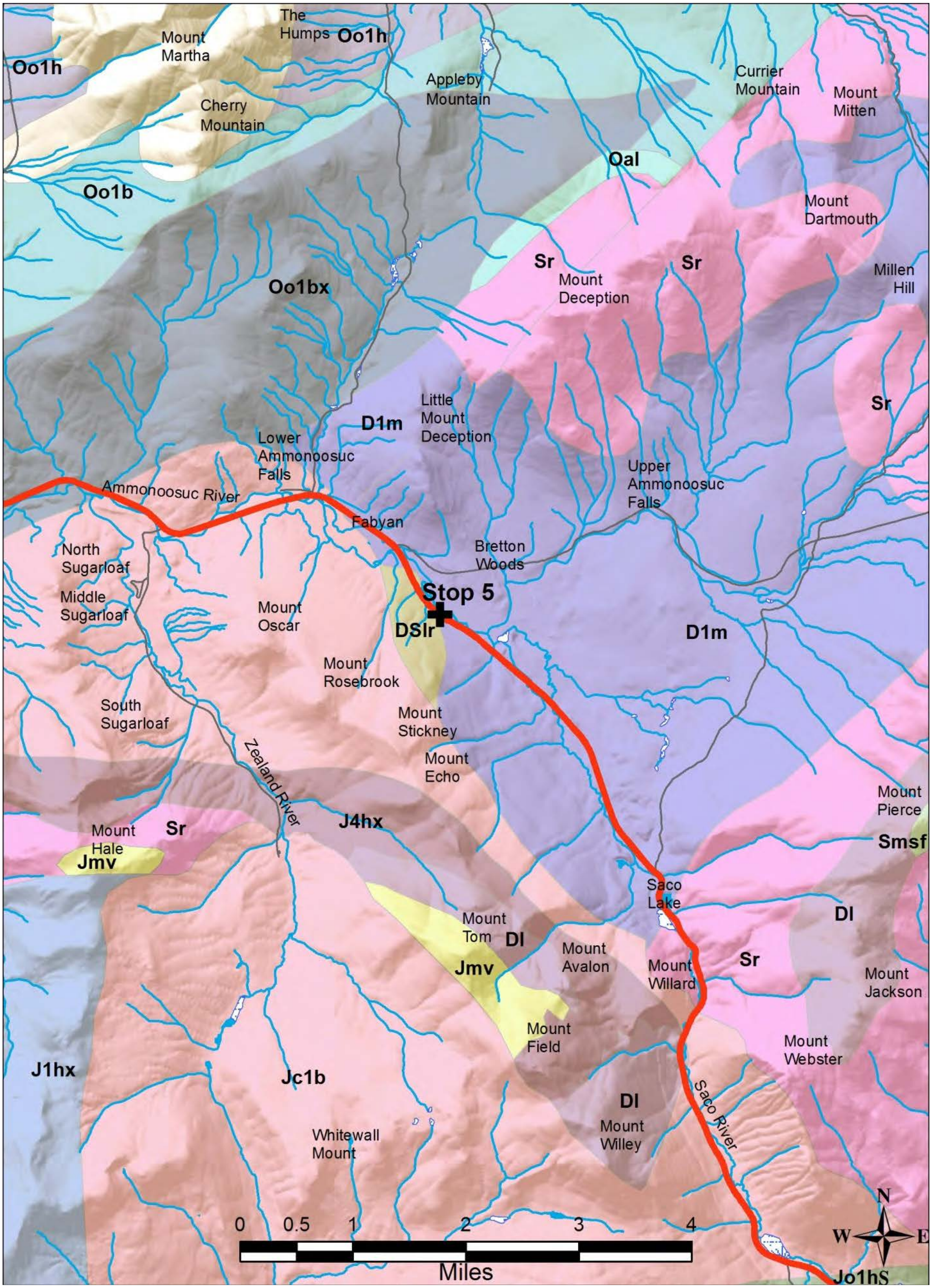
The fieldtrip will break for lunch after Stop 3. Lunch will be held at the Peabody Slope Base Lodge at Cannon Mountain.


Scrag Hill
Garnet Mountain
Oo1b
Five Corners
Oo1bx
Abbot Hill
Gale River
Franconia
Oalx
Priest Hill
Deerfield
D1m
Jc1b
Sr
Scarface Mountain
D1m
Flat Top Mountain
Big Bickford Mountain
D1m
Db2b
J1hx
Mount Garfield
Bald Mountain
Dk2x
Dlvs
Mittersill
J4hx
Echo Lake
Dk2x
Stop 3
Profile Lake
Cannon Mountain
Mount Lafayette
Dk2x
The Cannon Balls
Bald Peak
Mount Lincoln
Jc1b
Little Haystack Mountain
Pemigewasset River
Sr
J1hx
North Peak
Owls Head
South Peak Kinsman Mountain
The Basin
J1hx
Mount Liberty
Mount Flume
Flume Gorge
Mount Pemigewasset
Jo1h
0 0.5 1 2 3 4
Miles
J4hx
N
W
E
S

Page 42: **Stop 3 to Stop 4 (Old Man Historic Site to Carroll Visitors Center) -** Our travels to Stop 4 take us over poorly exposed rocks of the Ordovician Ammonoosuc Volcanics (Oalx, bimodal volanics) and of Ordovician Oliverian Plutonic Suite (Oo1bx). The latter are variably foliated biotite granite gneisses.

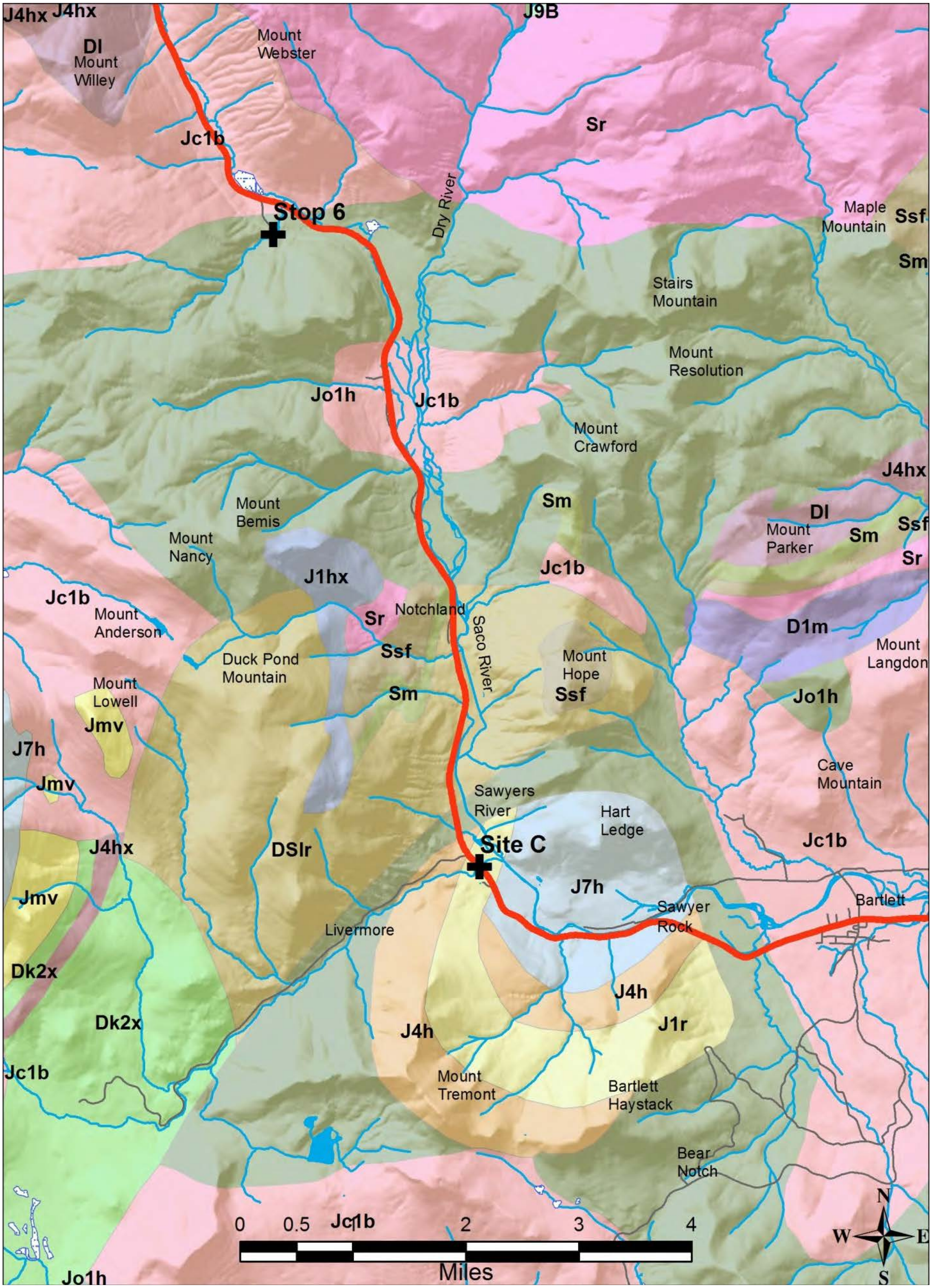

Stop 4
Stop 5
Twin Mountain
Carroll
Beech Hill
Fabyan
Bretton Woods
Saco Lake
Saco River
Zealand River
Ammonoosuc River
Upper Ammonoosuc Falls
Lower Ammonoosuc Falls
Mount Dartmouth
Mount Deception
Little Mount Deception
Mount Rosebrook
Mount Stickney
Mount Echo
Mount Oscar
Mount Tom
Mount Field
Mount Avalon
Mount Willard
Mount Webster
Mount Hale
North Sugarloaf
Middle Sugarloaf
South Sugarloaf
North Twin Mountain
Haystack Mountain
Flat Top Mountain
Oo1b
Oo1bx
Oo1h
Oal
Oalx
Sr
D1m
Dl
DSlr
Dk2x
Jc1b
J4hx
J1hx
Jmv
0
0.5
1
2
3
4
Miles
N
S
W

Page 44: **Stop 4 to Stop 5 (Carroll Visitors Center to Mt. Washington Scenic Overlook)** - From the Twin Mountain stop, we travel east across more Conway Granite before reaching the Mt. Washington Hotel Scenic Overlook, Stop 5 (underlain by two-mica granite of the Bretton Woods Pluton). Refer to Figure 3 on page 24 for an idealized cross-section of the area during Early Devonian time and visualize the overlying Littleton Formation eroded away to expose the pluton.at the present level of the land surface. Traveling southeast on US Route 302 we pass Saco Lake at the headwaters of the Saco River and through "the gate of the notch" into Crawford Notch. Along the way we cross a bit of Silurian Rangeley Formation and Conway Granite before reentering the main composite batholith as we approach Stop 6.


The Humps
Mount Martha
Oo1h
Cherry Mountain
Appleby Mountain
Curriher Mountain
Mount Mitten
Oal
Oo1b
Mount Dartmouth
Sr
Mount Deception
Millen Hill
Oo1bx
Sr
D1m
Little Mount Deception
Lower Ammonoosuc Falls
Ammonoosuc River
Upper Ammonoosuc Falls
Fabyan
North Sugarloaf
Bretton Woods
Middle Sugarloaf
Stop 5
Mount Oscar
DSlr
D1m
Mount Rosebrook
South Sugarloaf
Mount Stickney
Zealand River
Mount Echo
Mount Pierce
Mount Hale
J4hx
Smsf
Jmv
Saco Lake
Dl
Mount Tom
Jmv
Mount Avalon
Mount Willard
Mount Jackson
Mount Field
Mount Webster
J1hx
Jc1b
Mount Willey
Saco River
Whitewall Mount
0 0.5 1 2 3 4
Miles
N W E
Jo1hS

Page 46: **Stop 5 to Stop 6 (Mt. Washington Scenic Overlook to Willey House)** – South of Saco Lake, rock of the Littleton Formation occurs on the flanks of Mt. Willey as a screen between the outer ring dike and the main batholith. At the Willey House we have just crossed the contact between the Conway and Osceola granites. By Notchland we cross the largest roof pendant (~8 x 3 km northeast trending block) within the batholith that contains the stratigraphic section Rangeley (Sr), Smalls Falls (Ssf) and Madrid (Sm) Formations in addition to a mass of Dk2x.

If you're still following along, find where US Route 302 turns east south of Notchland. Here we cross one of more interesting of the smaller ring complexes at Hart Ledge. It consists of partial ring dikes and stock of syenite, quartz syenite, sodium and iron-rich amphibole syenites and granites of Jurassic age (but younger than the Osceola Granite that it cuts). From here it's a straight shot back to the conference center.


J4hx
J4hx
Dl
Mount Willey
Mount Webster
J9B
Sr
Jc1b
Stop 6
Dry River
Maple Mountain
Ssf
Sm
Stairs Mountain
Mount Resolution
Jo1h
Jc1b
Mount Crawford
J4hx
Sm
Dl
Mount Parker
Sm
Ssf
Sr
Mount Bemis
Mount Nancy
J1hx
Jc1b
Jc1b
Mount Anderson
Sr
Notchland
Saco River
D1m
Mount Langdon
Duck Pond Mountain
Ssf
Mount Hope
Ssf
Sm
Mount Lowell
Jo1h
Jmv
J7h
Jmv
Cave Mountain
Sawyers River
Hart Ledge
J4hx
DSlr
Site C
Jc1b
J7h
Jmv
Bartlett
Sawyer Rock
Livermore
Dk2x
J4h
Dk2x
J4h
J1r
Jc1b
Mount Tremont
Bartlett Haystack
Bear Notch
0
0.5
1
2
3
4
Jc1b
Miles
Jo1h
N
W
E
S

See previous map on Page 46: **Stop 6 to Drive-By Site C to Hotel (Willey House to US Route 302 Sawyer River Bridge to North Conway)** - The fieldtrip concludes after Drive-By Site C.

Field Trip Stop and Drive-By Site Descriptions

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Drive-By Site A

Soil Nail Walls along Kancamagus Highway in Albany, NH

Three soil nail walls were constructed from 1996 to 1998 along the Kancamagus Highway in Albany, New Hampshire. The Kancamagus Highway, designated a National Scenic Byway, is a 35-mile long stretch of NH Route 112 that cuts through the White Mountain National Forest. The walls reach a height of up to 22 feet and range in length from 425 to 600 feet. The soil nail walls replaced deteriorating wood crib walls that supported steep hillside cuts. Subsurface conditions encountered within the wall excavations included fill materials from the original crib walls and a natural glacial ice contact deposit consisting of gravelly sands with cobbles and boulders. The sites also had high groundwater levels.

The walls were constructed in a top to bottom sequence, with the excavation and wall construction occurring in 5 to 7 foot lifts. A shotcrete facing with a geocomposite drain system placed behind the shotcrete was utilized for each wall. The shotcrete facing ranged in thickness from 6.75 to 11.5 inches and was reinforced with either 0.5 inch diameter reinforcing bars or steel wire mesh. The geocomposite drain extended the full height of the wall on a 5 foot center to center spacing (Figure 1). A temporary dewatering system was required during the construction of each wall to lower the high groundwater levels.

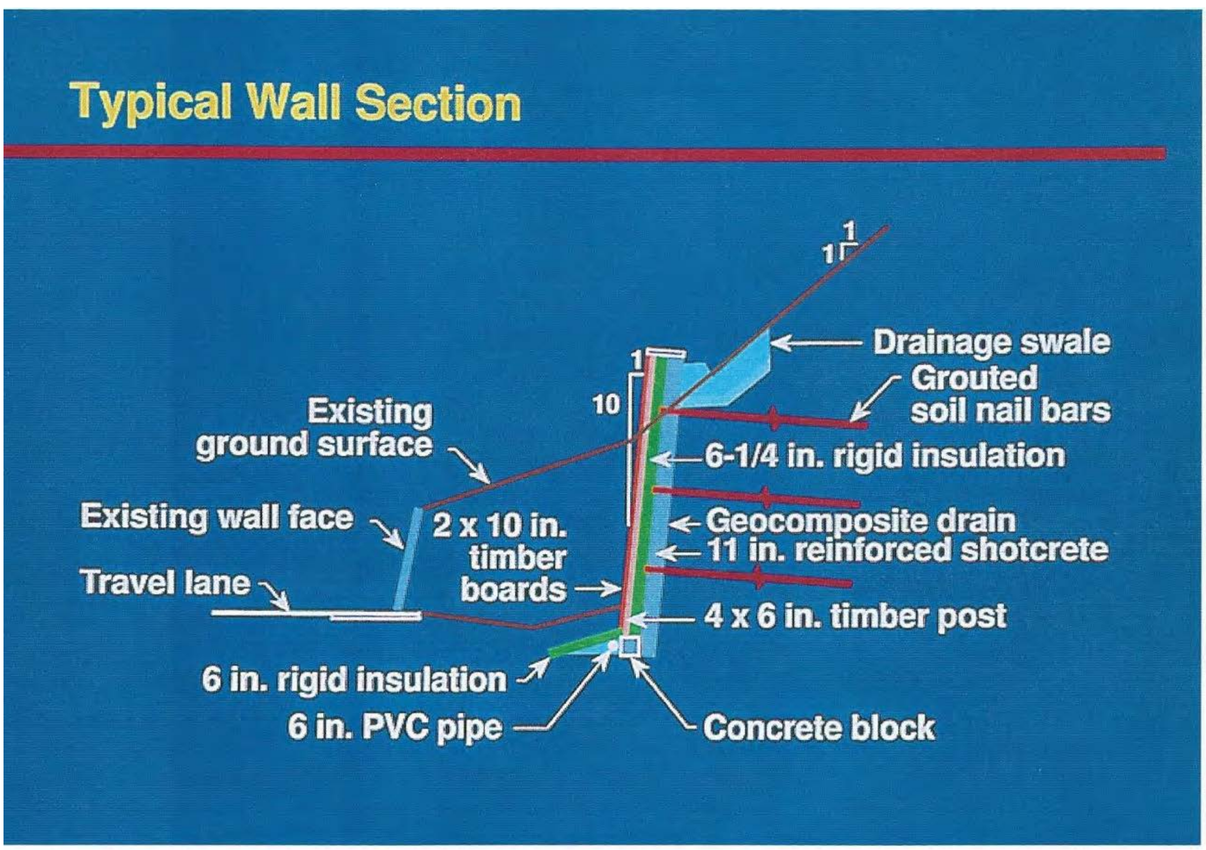

Figure 1 - *Typical soil nail wall section (Haley & Aldrich, 1997)*

The soil nails consisted of epoxy coated 1.25 inch diameter threaded reinforcing bars, with a minimum of 1.5 inches of grout cover. The soil nail lengths ranged from 20 to 50 feet and were generally placed on a 5 foot vertical and horizontal spacing. The design load of the nails was typically 40 kips. The soil nails were installed prior to placement of the shotcrete on the first wall and after placement of the shotcrete through blockouts on the remaining two walls (Figure 2).

To minimize the freeze-thaw effects behind the shotcrete, two layers of rigid extruded polystyrene insulation with a total thickness of 6.25 inches were placed over the shotcrete facing (Figure 3). In the past, cracking and displacement of the nails caused by ground freezing and thawing have precluded the use of these types of walls in cold climates (Haley & Aldrich, 1997).

A rough timber facing, which consisted of pressure treated 2 inch by 10 inch horizontal boards and 4 inch by 6 inch vertical posts, were attached to the wall over the polystyrene insulation (Figure 4). The timber facing provided an architectural wood face finish, which was requested by the US Forest Service (Figure 5) for aesthetic reasons. It was anticipated that the planking would have a shorter service life than the wall itself, requiring replacement of the facing in future maintenance. It is not known exactly how long the facing will last, but its replacement is expected to be relatively easy.

Geotechnical instrumentation was used to monitor conditions during construction of the walls and to verify whether the rigid insulation prevented ground freezing behind the wall. Slope inclinometer casing was installed prior to beginning the excavation on all three walls and was used to monitor lateral ground movements during and after construction. Vibrating wire piezometers were used to verify that the temporary dewatering systems had lowered the groundwater levels to an acceptable level prior to beginning the wall excavation. In addition, the piezometers were used to verify the long term performance of the horizontal drains installed on one of the walls.

Vibrating thermistors installed on the first wall behind the shotcrete verified that the rigid insulation successfully prevented ground freezing behind the wall. Vibrating wire load cells and strain gages were placed on three in-line soil nails on the third wall to determine if the actual nail loads were consistent with the predicted design loads.

One wall had permanent horizontal drains installed 50 feet into the hillside to lower the groundwater level and to reduce long term groundwater pressures on the wall. The horizontal drains consisted of two inch diameter slotted PVC pipes installed with a five degree upward angle. The addition of horizontal drains to reduce groundwater pressure, allowed the NHDOT to decrease the length of the nails to as short as 20 feet and reduce the thickness of the shotcrete.

Geotechnical design services were provided by Haley and Aldrich, Inc. of Bedford, New Hampshire for the first two walls and by the NHDOT Geotechnical Section for the third wall. Construction oversight was provided by the NHDOT Bureau of Construction. All three walls were constructed by Busby Construction of Atkinson, NH, in three separate contracts.

This soil nail wall design has provided a relatively maintenance-free, economical and environmentally friendly solution for locations with challenging site and subsurface conditions in a cold climate.

Figure 2. - *Drilling Soil Nail (Cleary, 1996)*

Figure 3.-*Placing Insulation (Cleary, 1996)*

Figure 4. - *Attaching timber facing (Cleary, 1996)*

Figure 5. - *Soil Nail Wall with timber facing (Cleary, 1996)*

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STOP 1: Pemigewasset Scenic Overlook

This stop lies immediately west of the height-of-land (870 m; 2,855 ft.) on the Kancamagus Highway (NH 112). It provides an opportunity to view scenery typical of the interior of the White Mountain region. The high peaks on the left are the East Peak of Mt. Osceola (1,267 m; 4,156 ft.) and Mt. Osceola (1,323 m; 4,340 ft.). The long Scar Ridge leads off to the right. To the immediate right of the Stop is Mt. Huntington (1,128 m; 3,700 ft.), and to the northwest and beyond are the higher peaks of Mt. Whaleback (1,093 m; 3,586 ft.) and Mts. Flume and Liberty (1,319 m; 4,328 ft. and 1,359 m; 4,459 ft., respectively). These peaks lie at the southerly end of the Franconia Ridge that forms the easterly side of Franconia Notch. Through the valleys below flow the East Branch of the Pemigewasset River and its tributaries.

Much of the region visible is underlain by the Conway and Mt. Osceola Granites. They dominate the Early Jurassic White Mountain composite batholith (see Eusden, et al., 2013, p. 78). They are the same age (~180 Ma) and differ primarily by the presence of one or two feldspars and hydrous or anyhydrous mafic minerals, respectively. They are roughly centered within overlapping ring dikes composed of finer-grained, porphyritic granites and/or syenites that may also preserve volcanic tuffs, breccias, and flows. These components testify to a long history of violent volcanic activity in the middle Mesozoic. Younger analogs include the Tertiary San Juan volcanic field of southwestern Colorado, the Pleistocene Yellowstone volcanics, and perhaps the recent Mt. Toba, Krakatoa, Mt. Mazama (Crater Lake), Mt. Pinatubo, and Mt. St. Helens activity in which parts or the entire volcanic edifice was removed during eruption.

All this terrain was at least twice buried by Pleistocene continental glaciation. The stoss and lee topography created by ice moving from northwest to southeast across the region is evident in steeper southeast-facing and gentler northwest-facing slopes. All these slopes are geologically young, having been exposed just since the departure of the Late Wisconsinan Ice Sheet 12,000 to 15,000 calendar years ago.

Of obvious interest are the numerous landslide tracks that scar the mountainsides. All those visible are debris avalanches that originated on 30^{o} and steeper slopes from super-saturation of collapsed moraine and bouldery-cobbly weathering detritus. Slides were initiated by heavy precipitation and/or melting events, with most occurring prior to human settlement of the region. These tracks have been subject to repeated but lower-volume avalanching as saturation events mobilize residual debris in their tracks and along their margins. The surfaces of their debris fans grade downslope from bouldery-cobbley debris to chaotically interbedded deposits of coarse sandy-silty gravel and minor amounts of stratified silty sand. The fans display stratigraphy typical of debris flows with the largest clasts on and near their surfaces. This stratigraphy is created by dispersive stresses within the debris streams during their rapid downslope movement.

Recent movements include those created by Tropical Storm Irene in 2011. This extraordinary storm furnished the region with 6 to 8 inches of rain in an 8 to 10-hour period. This deluge remobilized practically all of these tracks and created numerous new tracks in many previously unaffected drainages. We may be able to observe one of these new tracks from the highway later in the trip on the northwesterly side of Mt. Eisenhower (1,451 m; 4,760 ft.) near Bretton Woods.

Drive-By Site B

Loon Mountain Bridge over East Branch Pemigewasset River in Lincoln, NH

Map showing the track of Tropical Storm Irene on August 28, 2011, along the border of New Hampshire and Vermont.

On August 28, 2011, Tropical Storm Irene (downgraded from hurricane status as it entered the New England area) tracked northward, centered over the Connecticut River Valley, which is the border between New Hampshire and Vermont. Being on the easterly side of this fast moving storm, NH was spared the worst effects of the storm. Nonetheless, the state still experienced high winds, heavy rain and flash floods. As the storm passed over the higher elevations of the state, the water was rung out of the clouds with up to 7 inches falling in a 12 hour period in the White Mountain National Forest region. Damage to the region was extensive, including a number of roads and bridges being washed out. For a short period of time travel east to west in the region was not possible because of many closed roads. Fortunately, nobody was injured in the flooding.


Loon Mountain Bridge the day after the tropical storm event with 30 feet of the north embankment soil eroded away.

A major bridge impacted by Tropical Storm Irene was the Loon Mountain bridge crossing over the East Branch Pemigewasset River in Lincoln, NH. The 1960's era bridge was a three span structure that originally crossed a small lake created by a dam 1,000 feet down river. Thus, the bridge was designed for calm water conditions and had shallow spread footings on soil. A flood in 1973 destroyed the dam, and the impoundment drained. The bridge then experienced severe scour problems at the piers from the river, so they were retrofitted with sheet piles to deepen them.

Unfortunately, the stub abutments were never retrofitted. When Tropical Storm Irene hit, the river became a torrent, eroding about 30 feet of the northern river bank, leaving the north bridge abutment with virtually no support. The bridge stood for three days before collapsing. The span was removed, and a temporary one lane bridge was installed. Because of its history of problems, it was determined that a full bridge replacement would be pursued at a cost of 6 million dollars.

August 31, 2011, the north abutment falls.

STOP 2: Barron Mountain Rock Cut

Rock Slide

On November 7, 1972, during the construction of Interstate 93 in Woodstock, NH, a rockslide consisting of 17,000 +/- cubic yards of rock buried a portion of the Interstate 93 northbound barrel. The site, which overlooks the Pemigewasset River, is located at the base of Barron Mountain between I-93 Exits 30 and 31. Immediately after the slide, all rock excavation in the area was ceased and an extensive redesign of the roadway was undertaken to include changes to the highway alignment, reconfiguration of the rock slope, construction of a massive concrete retaining wall, relocation of a segment of NH Route 175 along with construction of three new bridge structures over the Pemigewasset River, drilling of 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).

Site Conditions and General Geology

The existing northbound rock slope reaches a maximum height of 130 feet and is 600± feet in length. A rock bench, 90 feet above ditch elevation, extends along the rock slope except on the north end where the scar from the slide is located (Figure 1). The southbound barrel, constructed 30 feet below the northbound barrel, notches into bedrock in the median and rests on fill down slope. A concrete wall, built along the river, supports the toe of the steep embankment slope.

Slightly foliated gneissic rock composes the southern portion of the rock cut, grading into strongly foliated quartz-mica schist in the northern section. A large andesite dike intrudes the country rock in the middle of the cut with smaller basalt and pegmatite intrusions scattered throughout the rock slope. Several sets of joints crisscross the rock, some parallel to and others transverse to the foliation. Many of the joints in the schistose zone are filled with mylonite gouge with the presence of slickenside along some of the surfaces. Numerous fractures and offset features indicate a complex history of past tectonic events.

The 1972 rockslide occurred along a highly fractured mylonite zone, which ranged in thickness from 1/2 inch to 11 feet (Figure 2). This weakened zone of low strength material along the failure surface was oriented nearly parallel to the roadway alignment and dipped into the road at approximately 38 degrees (Fowler, 1976 and 1977). Additional mylonite zones with similar orientation were encountered south of the slide area during the early stages of the reconstruction requiring a second redesign of the rock slope. Water flowing along unfavorably oriented mylonite zones was most likely a major factor in triggering the slide.

Rock Reinforcement Installed to Stabilize Slope after Slide

Both active and passive reinforcements were installed at the Barron Mountain site (Figure 3). The passive reinforcement consists of 70 tendons, generally 50 to 60 feet in length (instrumented tendons were longer), installed with no anchorage assembly in three rows on a 10’ by 10’ grid pattern along the toe of the southern half of the northbound rock slope. An additional 30 tendons were installed on 8 foot centers in the upper portion of the rock slope. 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 installed to secure existing blocks, to tie together the rock mass, to preserve the full gravity effect of the rock bench,, and to prevent minor rock falls from reaching the highway (Haley & Aldrich, 1974). 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. A total of 200+ rock bolts were installed on the northbound and southbound rock slopes at the Barron Mountain site.

Figure 1 - *Barron Mountain Rock Cut (Lane, 2004)*

Figure 2 - *Mylonite zone along failure surface prior to slide (Fowler, 1972)*

Field Instrumentation

Field instrumentation was installed to monitor the rock mass behavior and to collect data on the performance of the rock reinforcement (Haley & Aldrich, 1974). The instrumentation included three 150 foot deep six position rectilinear extensometers on rock tendons, three 80 foot deep four position rectilinear extensometers on rock tendons, two 50 foot deep two position mechanical extensometers on rock tendons, ten sets of temperature measurements on rock tendons using thermistors, and four sets of load cells on rock bolts. In addition, twelve vertical holes were drilled 50 feet into rock to serve as observation wells for monitoring water levels and to listen for subaudible rock noise.

Initially all the instruments were read weekly and the observation wells checked monthly. The data collected from the instruments was continuously plotted in an attempt to identify potential movement within the rock mass and monitor changes in stress levels in the rock reinforcement elements. After 18 months, instrument readings were reduced to four times a year. As time passed, instruments began to fail, readout wires were damaged by rock fall, wires were chewed by mice, and metal readout boxes corroded. Every spring and fall, detailed visual observations were conducted to include examination of cracks in the rock, water seepage, staining and condition of exposed portions of the rock reinforcement and grout. Although, no obvious trends in the instrument readings developed, loosening of bearing plates at several of the non-instrumented rock bolts and heavy seepage of water from a few tendons were observed. Continuous plots of the instrument readings were maintained until 1985, when the last of the instruments stopped working. Although visual inspections of the rock slope and the reinforcement were conducted periodically, there was no method for determining the actual condition of the existing rock reinforcement. Over time, corrosion of the metal reinforcing elements, particularly the unprotected segments of the rock bolts and the long-term reliability of the resin grouted pre-stressed bolts were becoming more of a concern.

NHDOT Two Phased Research Study (2003-2004)

The estimated design life of unprotected rock reinforcement systems is approximately 50 years based on service life and metal loss equations. The New Hampshire Department of Transportation (NHDOT) has been concerned with the longevity of the rock reinforcement system at the Barron Mountain rock cut given that more than half the anticipated design life has passed. To address this concern, the NHDOT undertook a research study to assess the existing condition of the rock reinforcement. The condition assessment followed the recommended practice from NCHRP 24-13 (NCHRP, 2002) and was performed in two phases implemented in the summer 2003 and fall 2004. McMahon and Mann Consulting Engineers, P.C. conducted both phases of the research.

The first phase (Fishman, 2004) involved the measuring of the corrosiveness of the surrounding environment and performing nondestructive testing (NDT) on selected reinforcement elements. Samples of weathered rock and groundwater were tested for pH, resistivity, moisture conditions, and sulfate and chloride ion concentrations. A rate loss model was used to determine potential metal loss from corrosion and to estimate the remaining service life of the reinforcement. The study utilized four NDT methods, recommended in NCHRP 24-13 (NCHRP, 2002), to assess the condition of the reinforcement elements. Two were electrochemical: half-cell potential measurements and measurement of polarization current; and two involved wave propagation techniques: the impact echo test and an ultrasonic probe. The electrochemical tests identify the presence of corrosion or the vulnerability of the reinforcement steel to corrosion. The wave propagation techniques assess the severity of the corrosion, diagnose the loss of pre-stress and the lack of grout cover, determine if the cross section had been compromised, and identify locations of potential bending or deformation in the metal bars.

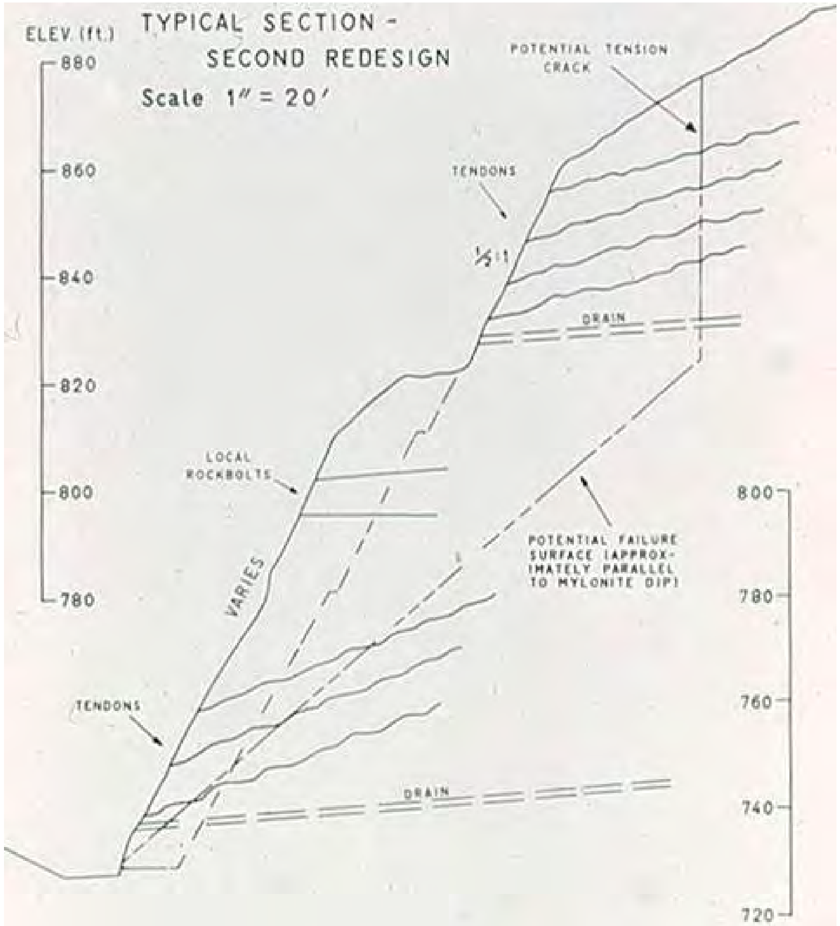

Figure 3 - *Typical section with rock reinforcement (Fowler, 1976)*

The second phase (Fishman, 2005), which was a pooled fund study, used destructive testing to verify the results from Phase I. The techniques included lift-off testing of selected rock reinforcement and the physical, chemical, and metallurgical testing of steel and grout samples retrieved from exhumed reinforcement (Figure 4). The grout condition was evaluated by observing the coverage of the exhumed reinforcement, by the consistency of the grout, and by the physical properties of the grout mix. Bulk specific gravity and absorption were used to determine the effectiveness of the grout as a barrier against moisture and to manage the intrusion of elements that could cause corrosion. Examination of the exhumed metal elements consisted of visual observations of corrosion, measuring the pit depths and the loss of section. Samples of the exhumed metal reinforcements were subjected to tension tests to measure the percentage of elongation and to determine the corresponding stress-strain curves. Metallurgical tests included a spectrographic analysis to assess the metal composition, and a metallographic examination to observe the microstructure of the thread bar material. Destructive testing verified that the electrochemical tests correctly identified the presence of corrosion. The lift-off tests and direct measurements confirmed the echo test results. Measurements on exhumed rock reinforcement verified that the greatest loss of section was within the free length behind the anchorage assembly.

The tendons are in better condition compared to the rock bolts. In spite of the apparently high porosity of the cement grout, it appears to have protected the steel from significant corrosion to date. Many of the rock bolts have suffered a loss of pre-stress and some corrosion is evident. Thus, with respect to impacts on service-life, the rock bolts at this site are more vulnerable than the tendon reinforcements. Compared to loss of service from corrosion, results from the condition assessment revealed that loss of pre-stress is the bigger concern relative to remaining service-life. The condition assessment also revealed locations of increased corrosion activity. Thus, a sound technical basis was established for planning future maintenance and rehabilitation activities at the site, ultimately resulting in a cost savings to the NHDOT.

The Barron Mountain rock cut was a unique site for determining the effectiveness of these techniques because of the age of the reinforcement, the environmental conditions, the variety of installation procedures, and the use of different types of grout. The loss of measured cross section of the unprotected portion of the rock reinforcement was consistent with the predictions from the mathematical models for the service life of unprotected steel and with the observations from the NDT.

Removal of Unstable Block and NDT Testing Along I-93 Southbound Median Rock Cut

During 2005, an unstable block was removed from the existing median rock slope along the southbound barrel. The block (estimated dimensions 6 ft. by 7 ft. by 8.5 ft.) had become detached from the surrounding rock forming an open tension crack, 1 to 2 inches in width. Although the block had moved, it was still held in place by two 1-inch diameter resin grouted rock bolts. Rock bolts in the median cut were of similar design and installed with the same procedures as those along the northbound barrel. The bearing plates for rock bolts securing the block had buckled, indicating a high level of distress and a significant increase in load. Portions of the rock bolts that had secured the block were recovered for testing when the block was removed. One of the steel rock bolt bars was severely corroded where it had crossed an open joint. Measurements showed the bar had lost approximately 25% of its cross section (Figure 5).

Figure 4 - *Exhuming rock bolt by over coring (Lane, 2005)*

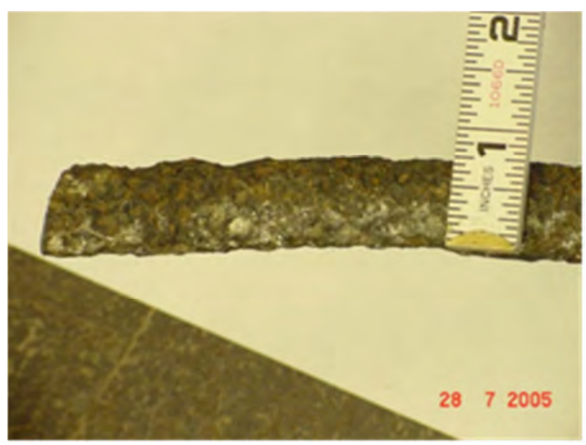

Figure 5 - *Approximately 25% loss of cross section of rock bolt recovered from the southbound median (Lane, 2005)*

McMahon & Mann Consulting Engineers returned to the Barron Mountain rock cut in 2007 to evaluate thirty-six existing rock bolts installed along the median rock cut of the southbound barrel. The work was conducted under the National Cooperative Highway Research Program (NCHRP 24-28), LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforcing Systems in Geotechnical Applications (NCHRP, 2011). The evaluation consisted of conducting nondestructive tests (NDT) on three clusters of rock bolts scattered throughout the existing cut and testing of soil/rock infilling material. The NDT measurements included half-cell potential; corrosion rate; wave dispersion from sonic echo tests; arrival times from sonic echo tests; and arrival times from ultrasonic tests. Interpretations of the impact tests indicated approximately 30% of the bolts had experienced loss of pre-stress and the grout quality was questionable in 80% of the cases. These results were consistent with the findings from the two phased research study (2003-2004) completed on the existing rock bolts installed along the northbound rock cut.

Retrofit and Remediation of Rock Reinforcement at Barron Mountain (2009)

Although visual inspections of the rock slope and reinforcement were conducted annually there was no method for determining the actual condition of the existing rock reinforcement. More than half of the generally accepted 50-year service life had passed, and results of two phased research study conducted under a pooled fund study (TPF-5(096)) indicated that approximately 30% of the rock bolts may have suffered a loss of pre-stress. The research provided an effective method for identifying areas of possible corrosion, assessing the overall condition of the reinforcements and estimating remaining service life. As a result, in the summer of 2009 a remediation contract was advertised that was completed by Pacific Blasting and Demolition, Ltd. Over 200 resin grouted rock bolts were tested and approximately 32% of the rock bolts tested exhibited a loss of pre-stress and required replacement.

Figure 6 – *Rock bolt installation being performed in 2009 by Pacific Blasting and Demolition.*

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STOP 3: Old Man Historic Site

Franconia Notch and the Old Man of the Mountain

Franconia Notch is one of the best developed and famous glacial troughs in northeastern North America and the White Mountain region. Its fame arises not only from its impressive scenery but from the former presence on the cliffs of Cannon Mountain of the singularly famous Old Man of the Mountain natural rock "Profile". On May 3, 2003, the Profile collapsed and fell about 825 ft. (250 m) onto the talus slope above the Interstate 93 "Parkway" in the Notch. The collapse resulted in the loss of a famous geologic landmark, the official emblem of the State of New Hampshire, and a sublime "old friend" to many who had visited its viewing site in the 198 years since it was discovered in 1805. On this 10th anniversary of its collapse, this stop will visit the Old Man Memorial Plaza & Geological Exhibit at its former viewing site where the geology and rock mechanics of its creation and failure can be reviewed and the implications of its collapse considered. Eusden et al. (2013) contains a general summary of these matters. Detailed discussion, along with the Profile's nearly 200-year "human history", appears in Fowler (2005).

Formation of the Notch

Franconia Notch (Figure 1) is a classic example of a glacial trough created by several lengthy episodes of erosion by thick continental ice sheets. Because their "plastic" ice masses sought the path of least resistance through the mountain range as they approached from the northwest, they first converged into its deeper pre-existing passes and then flowed through them continuously as they thickened and then waned during each episode. The last two of these ice sheets (the Illinoian and Late Wisconsinan) were sufficiently thick to bury the region's highest peaks as shown by deposits of basal till on the summit of Mt. Washington at 6,288 ft. (1,900 m).

Most workers here agree that the last pulse of continental glaciation (Late Wisconsinan) arrived here about 25,000 calendar years ago, reached its maximum thickness about 18,000 years ago, started to recede from the highlands about 15,000 years ago, and had fully retreated from lowlands north of the highlands by about 12,000 years ago. Surficial geologic mapping shows this deglaciation process occurred in two ways. First the higher peaks were exposed as nunataks above the ice sheet as it downwasted around them, while the rest of the ice sheet gradually retreated northwestward around the mountain highlands.

Franconia Notch is joined in the region by Crawford and Pinkham Notches as the best developed examples of its glacial troughs. Together, they represent the deepest of the pre-glacial passes in the mountain front. The glacially-sculpted U-shaped cross-sections of these Notches are distinctive and best developed in Crawford Notch, which we will visit at Stop 5. This cross-section in Franconia Notch has been substantially modified by post-glacial talus infilling, particularly along its western slopes.

Figure 1 – *View of Franconia Notch from the south.*

The Old Man of the Mountain

The Old Man of the Mountain (Figure 2) was a delicately cantilevered, 7,200-ton rock mass created naturally by the combination of two actively persistent weathering processes that formed the Cannon Cliff after the departure of the last ice sheet around 12,000 calendar years ago. The first is the intense kaolinization weathering of potash-rich feldspars along the joint systems in its Early Jurassic Conway Granite pluton. The second is the easy mechanical excision of consequently loosened blocks by intense cycles of freeze-thaw wedging (presently 30-60 cycles per year). The effectiveness of these processes is evident from the enormous talus slope accumulated at the base of the Cannon Cliff, the most extensive in the region.

Figure 2 - *Former Old Man of The Mountain from Profile Lake, Franconia Notch, N.H.*

Three systems of joints in the rock mass were exploited by these processes. The first and second are subvertical and subhorizontal joints formed during the cooling of granite pluton (Figure 3), while the third is a combination of more closely-spaced, near-surface subhorizontal and subvertical joints formed during surface dilation as the weight of the last ice sheet was removed.

This dilation opened easy paths for deep penetration of large volumes of rain, meltwater, and wind-driven cloudwater into the rock mass. This penetration in turn encouraged the rapid progress of kaolinization along the joints and a dramatic simultaneous increase in porosity and cleft water pressures within the intersecting joint systems (Figure 4). This ample supply of cleft water made it relatively easy for the active diurnal and seasonal freeze-thaw cycling to mechanically excise joint-bounded blocks destabilized by the weathering. In these ways, the formation of the strikingly human Profile was a serendipitous consequence of just which of these destabilized blocks was excised and in just what order so that the Profile and the delicate cantilevering needed to perch it on the cliff was created and temporarily (from a geologic point of view) preserved (Figure 5).

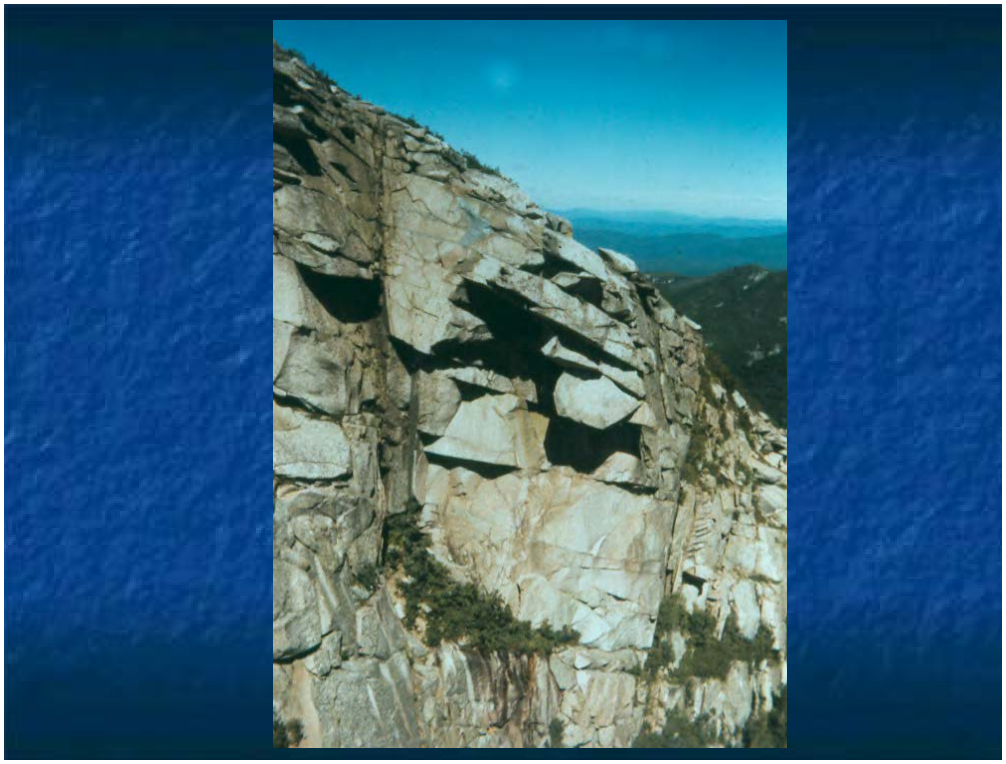


Figure 3 - *Joint distribution and rock mass bounding joints in the pre-collapse rock mass. Chin block (now missing) creates lowest shadow.*

Figure 4 - *Intersecting weathered joints in the pre-collapse rock mass. Chin block on the right behind the vertically hanging rope.*

Figure 5 - *Delicately cantilevered pre-collapse rock mass. The triangular chin block is to the right of the vertically hanging rope.*

Figure 6 - *Post-collapse residual rock mass after Profile toppled forward (left). Note broken steel turnbuckles on residual Forehead (topmost) slab.*

On May 3, 2003, as had been predicted by earlier studies (Fowler, 1982, 2005), the structural base upon which the cantilevering relied failed and the frontal portion of the rock mass that included the Profile's particular blocks toppled forward and off the cliff. This event created a significant social and tourist-economy loss for the State. In response, numerous proposals were put forward to physically replace the Profile in front of the cliff using various constructed combinations of rock and/or lighter-weight artificial materials tensionally anchored to the residual rock mass. Such proposals rely on the assumption that this residual mass is in sufficiently sound structural condition to accept this sort of foundation (Figure 6), but post-collapse evaluations show this assumption is not appropriate.

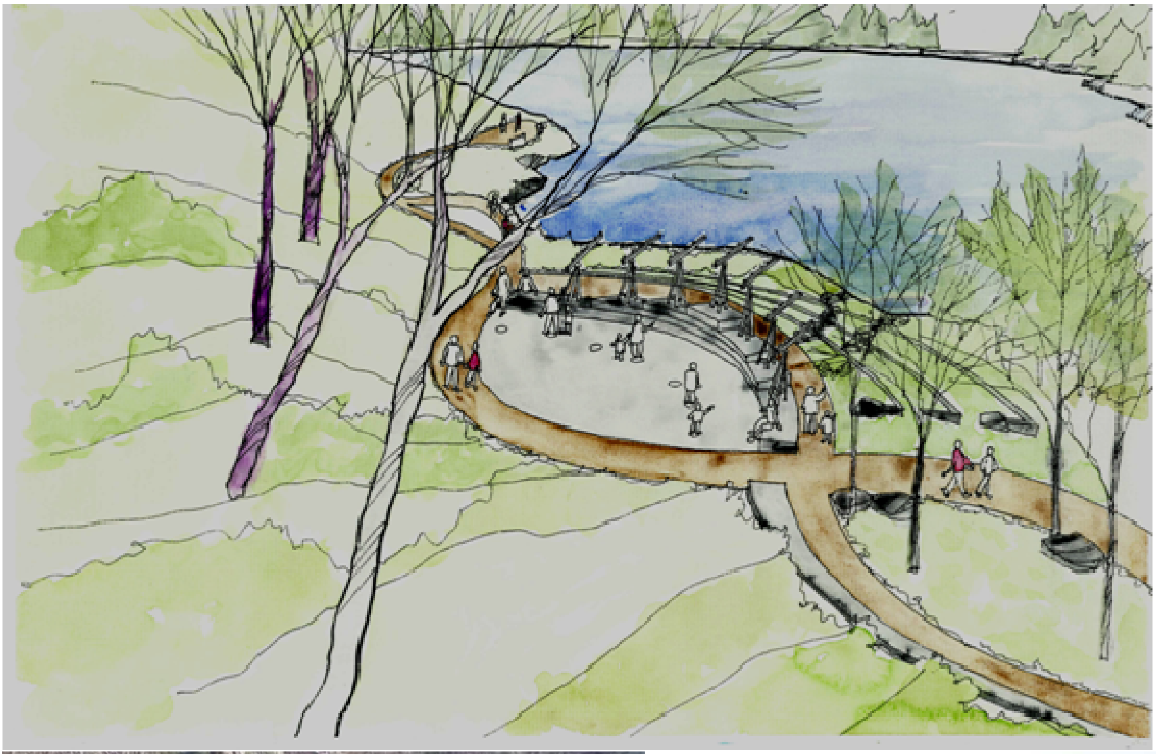
These evaluations strongly suggest the bulk structural integrity of the residual mass has been, and continues to be, seriously compromised by the same intense processes that formed and then destroyed the Profile. They show that deep penetrative weathering and buildup of cleft water pressures along its joints, combined with intense freeze-thaw excision, continue unabated reducing its bulk strength, resistance to various modes of block displacement, and its capability to serve as a foundation resource for tensional anchorage.

Immediate post-collapse and recent continuing observations (Fowler, 2005, 2009) show these deteriorated conditions continue to develop deeply within the residual rock mass. From the residual cliff inward, significant deterioration is observed to depths of at least 20 ft. (6 m), while observations on the mountainside behind and above show these conditions to depths of more than 50 ft. (15 m), especially in the vicinity of the compound subvertical bounding joint systems at the margins of the residual mass (Figure 3). Increasing cleft water pressures are particularly problematic in this rock mass because there is no feasibly reliable way to reduce or eliminate them by sealing or draining, as shown by many attempts to do so on and above the original Profile over the many years before its collapse.

Figure 3 shows these seriously deteriorated internal and bounding joint combinations. When the cantilevered blocks that comprised the Profile collapsed, the rearward portion of the similarly deteriorated rock mass behind did not fall simultaneously. This permitted the residual mass to remain tenuously perched on the cliff. However, partial or even complete collapse of this mass could now easily occur as a combination of sliding and toppling within and along these weathered joint systems.

In any case, these geotechnical circumstances lead to the conclusion that successful long-term active and/or passive reinforcement of this dynamically impermanent rock mass will be very difficult and probably impossible to achieve. This leads informed geotechnical and public policy observers to generally agree that investment in any project proposed to be founded upon it will represent imprudent professional and political risk.

Old Man Memorial Plaza



The Old Man Memorial Plaza recreates the remarkable experience of seeing the Profile high above on the cliff. The Plaza and its exhibits have been constructed by the Old Man of the Mountain Legacy Fund in cooperation with the NH Geological Survey using entirely private donations and the proceeds from the sale of personalized pavers on the Plaza floor. Pavers are of three sizes that are custom-engraved with personal messages; many are still available. Information about the Plaza, its exhibits, history and geology of the Old Man, purchase of pavers, and Fund activities is available at: http://www.oldmanofthemountainlegacyfund.org/.

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LUNCH: Cannon Mountain

Lunch will be served at the Peabody Slope Base Lodge of the famous Cannon Mountain Ski Area owned and operated by the State of New Hampshire, Division of Parks & Recreation. The ski area was constructed in the early 1930's by the Civilian Conservation Corps and others as a way to provide jobs during the Depression and to tap into the newly-popular automobile-based recreation industry then emerging. Cannon was one of the first ski areas of its size in North America and was the first in North America to feature transportation of tourists and skiers to its highest point, the summit of Cannon Mountain (4,100 ft.), via the aerial tramway. The Tramway is celebrating its 75th year of operation this year. In 1938 when it opened, it immediately became a sensation in the skiing world, drawing much appreciated attention to northern New Hampshire's recreation opportunities, along with providing hearty skiers with up to 2,300 feet of elevation drop. Many of the earliest competitive alpine skiing events were held here at Cannon because of its steep and highly technical terrain. Many championship skiers have learned and perfected their technique on its challenging often windblown and icy slopes, the latest of these being Bode Miller of Olympic and World Cup fame.

STOP 4: Carroll Visitors Center

Introduction

This stop is located in Twin Mountain village at the former site of the Twin Mountain House (1868-1960). It was one of the "grand hotels" that catered to White Mountain tourists from the 1800s through the mid-1900s (Figure 1). Most of those hotels have been lost to fire or demolition. (Later today we'll see one of the great survivors – the Mount Washington Hotel at Bretton Woods.) Note the information kiosk at the visitor center, which provides interesting details about this part of New Hampshire.

Figure 1 - *View looking north at the Twin Mountain House in the 1800s (part of a stereoscopic view published by Kilburn Brothers, Littleton, NH). Note the cross section of an esker ridge to left of hotel and the Ammonoosuc River in foreground.*

We are overlooking the Ammonoosuc River valley just south of here (Figure 2). This is one of several rivers that flow generally west out of the White Mountains and empty into the Connecticut River on the New Hampshire-Vermont border. Many Quaternary scientists have investigated the Ammonoosuc River basin, starting with Louis Agassiz in the mid-1800s

(Thompson, 1999). The appeal of the region to 19th century geologists was fueled by easy railroad access and popularity of the mountain scenery with tourists and artists. There were numerous controversies over the extent and types of glaciers that affected the White Mountains, and the manner in which they developed and subsequently retreated. Details of the glacial chronology and its relation to climate change continue to be studied today.

Figure 2 - *Google Earth view looking south across Twin Mountain village, showing the stop at Carroll visitor center at the junction of Routes 3 and 302. The gravel pit at bottom-center edge of photo is the esker location shown below in Figure 3.*

The glacial features in the Ammonoosuc valley tell us much about the retreat of the last glacial ice sheet from the White Mountains. As the climate warmed toward the end of the Ice Age, the Laurentide Ice Sheet began to melt. This caused the glacier to become thinner, while at the same time its southern margin retreated back toward Quebec. Some of the rock debris carried by the glacier was simply released from the melting ice and not carried any farther. This material is called "till", and it forms a widespread blanket over New Hampshire.

Just north and west of here, there are clusters of bouldery till ridges (moraines) that were heaped up at the margin of the glacier when it was briefly reenergized during its overall retreat. These deposits include the Beech Hill Moraines marked on Figure 4. Recent research has shown that they formed during a brief interval of cold climate called the "Older Dryas event" that occurred about 14,000 years ago (Thompson et al., 2009).

Other glacial sediments were transported by meltwater streams that originated within or upon the ice sheet. The coarser material (gravel) usually was left closest to the glacier, either as subglacial tunnel fillings (eskers) or as stream deposits laid down a short distance beyond the ice margin (outwash). Sand, silt, and clay tended to be carried greater distances and sometimes came to rest in temporary glacial lakes. Both eskers and glacial lake deposits occur here in the Ammonoosuc River valley and will be described below. Glacial sand and gravel deposits are very important to the New England economy. They offer well-drained building sites that are easier to excavate than till or bedrock. They are also important sources of construction aggregate, and many of them are high-yield aquifers.

Note: The following sections are modified from Thompson et al. (1999, 2002).

The Ammonoosuc Valley Esker System

Meltwater flow within the Laurentide Ice Sheet carved tunnels at the base of the glacier. The subglacial drainage often followed valleys, but the water was confined under pressure and could actually flow uphill over topographic barriers beneath the ice. Today we can tell where some of those tunnels were located because they became choked with sand and gravel. After the ice melted away, the tunnel-filling sediments were left behind as ridges called *eskers*.

Figure 3 - *West side of esker ridge, just north of Twin Mountain. W.B. Thompson photo.*

The Ammonoosuc esker system originates just north of Twin Mountain village (Figure 3) and follows the valley eastward. It merges with another esker entering from the north and continues to Crawford Notch. The esker ridge is discontinuous, and parts of it are concealed in the forest. A convenient place to visit the esker is the Eisenhower Wayside Park, located on U.S. Route 302 between Bretton Woods and Crawford Notch. A cross-section of the ridge is seen from the parking lot, and you can hike up a short path to the top and follow the ridge crest back into the woods.

Glacial Lake Ammonoosuc

In New England, much of the meltwater from the ice sheet poured into temporary lakes that formed in front of the glacier as it retreated northward. Many of those lakes resulted from the ice damming valleys that sloped toward the glacier margin. At any given time, the water level in each ice-dammed lake was controlled by the elevation of the lowest gap in the surrounding hills through which the lake water could escape. As the ice sheet retreated from a river basin, the lake level in that valley would drop whenever a new and lower outlet was uncovered by melting of the ice. Eventually the lake would completely empty and disappear when the ice sheet left the basin.

We are not certain who was first to recognize the former existence of glacial lakes in this part of the White Mountains. It may have been Warren Upham, who worked with State Geologist Charles Hitchcock on the monumental geological survey of New Hampshire in the 1870s. Upham's observations led him to propose that a lake had existed in the upper Ammonoosuc River valley, in the vicinity of today's Bretton Woods community (Upham, 1878). This was the water body that James Goldthwait (1916) named "Lake Ammonoosuc". It resulted from damming of the river basin by the last ice sheet when it receded west and north from the valley. As the ice margin withdrew, numerous lake levels developed through the process described above (Thompson et al., 1999).

In 1930, Richard Lougee assisted Goldthwait in the gravel inventory funded by the New Hampshire Highway Department. Lougee was assigned to map several 15-minute quadrangles in the White Mountains. He identified many of the glacial lakes that were dammed by the margin of the ice sheet as it withdrew from this area and temporarily blocked the normal stream drainage. Lougee also realized that the earliest stage of Lake Ammonoosuc (the Crawford stage) drained eastward through Crawford Notch.

Following the Crawford stage, glacial retreat enabled Lake Ammonoosuc to drain southwest into the Gale River valley in Franconia. The water escaped through five progressively lower outlets called "spillways". The lake levels corresponding to these spillways are known as the Gale River stages of Lake Ammonoosuc (G1-G5 in Figure 4).

The widest and deepest phase of Lake Ammonoosuc may have been Gale River 2 stage. The log for a well located just west of Twin Mountain village shows a contact between thick glaciolacustrine clay and the underlying till at an elevation of 401 m (Flanagan, 1996).

Comparison of this lake-bottom elevation with the nearby G2 spillway elevation of 445 m indicates a local water depth of at least 44 m.

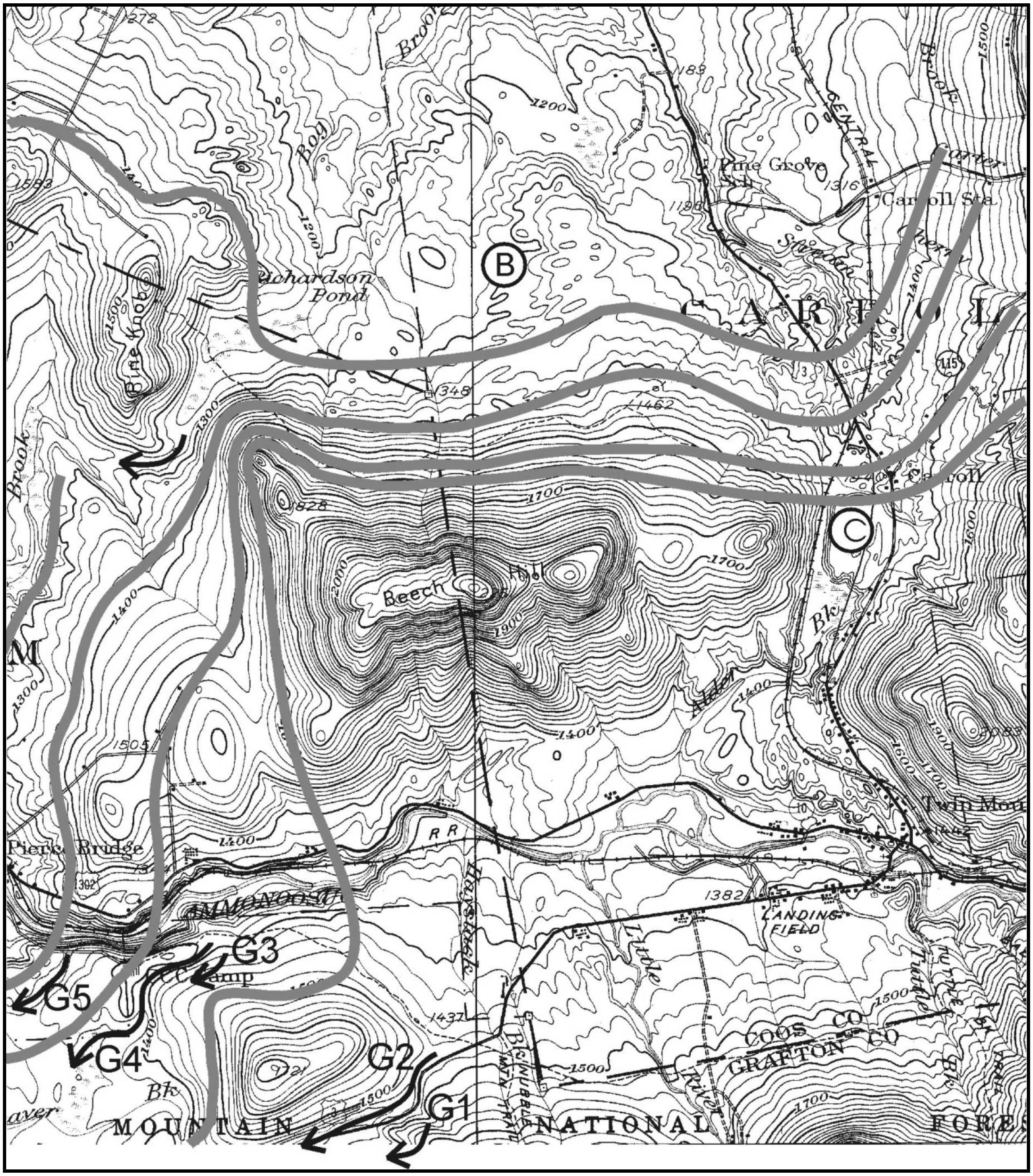


Figure 4 - *Map from Thompson et al. (1999) showing outlets for the Gale River stages of glacial Lake Ammonoosuc (arrows G1-G5). The thick gray lines mark successive positions of the glacier margin. B = Beech Hill Moraines. C = Carroll Delta.*

Figure 4 also shows recessional positions of the glacier margin that correlate in time with the Gale River and later stages of Lake Ammonoosuc. These ice margins were inferred from the orientation of nearby moraines, together with ice blockages of the valley that would have been required to hold the lake at elevations corresponding to the known deltas and spillways.

Lougee (1940) published the elegant block diagram reproduced in Figure 6. It shows the ice margin lying against the northwest flanks of Beech Hill and Cherry Mountain when the Carroll Delta was built into Lake Ammonoosuc. This delta is located a short distance north of our stop, on US Route 3. The elevation of its upper surface indicates that it was deposited during the Gale River 2 stage of Lake Ammonoosuc. The Carroll Delta and associated esker deposits have been important sources of sand and gravel for many years. The Twin Mountain Sand & Gravel pit (now owned by Pike Industries) has been worked at least since the mid-1900's (Figure 5).

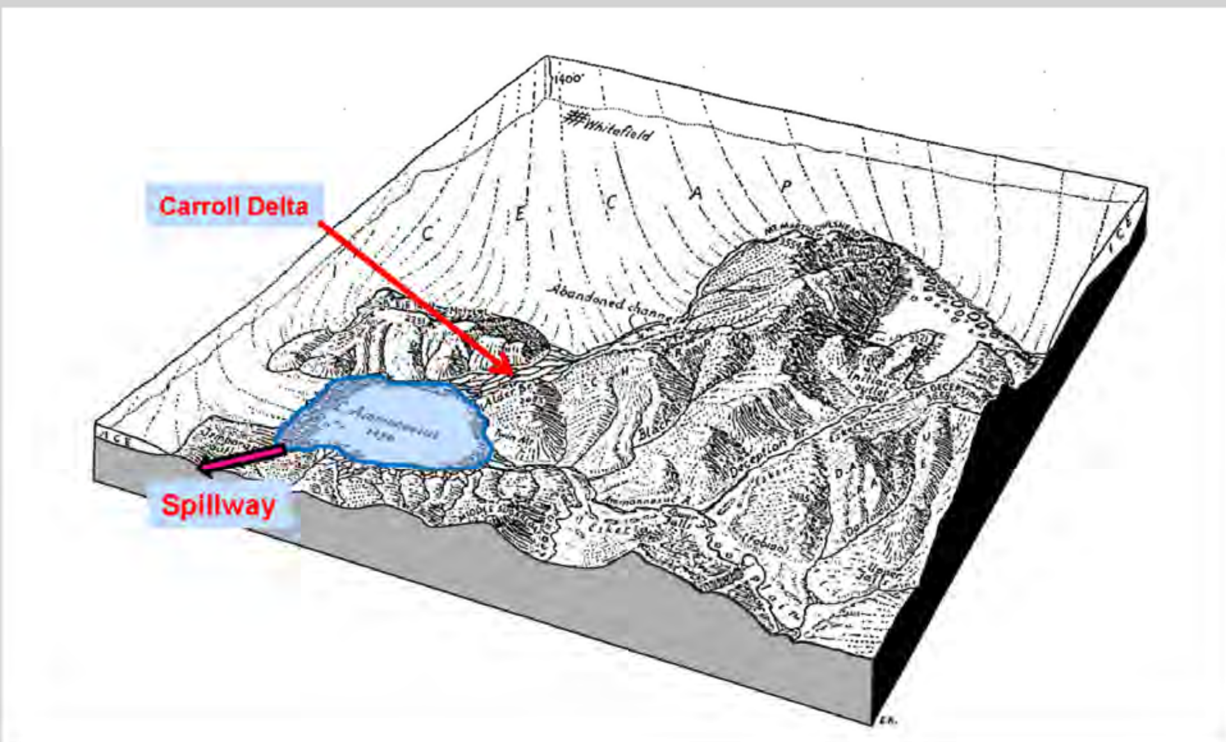
Figure 5 - *View looking west at an exposure of the Carroll Delta in the Twin Mountain Sand & Gravel pit. The former lake level is marked by the contact between the nearly horizontal fluvial beds (top) and the sloping foreset beds below. The glacier margin stood at the north edge of the delta, and the delta built southward (R to L) into Lake Ammonoosuc. W.B. Thompson photo.*

Today we can still see the spillway channels eroded by water draining from glacial lakes in the White Mountains. Most of them have flat floors cut into glacial till, and are now occupied by swampy wetlands. The channels are most clearly visible when leaves are off the trees. A good example is the G2 spillway (Figure 4) that drained the overflow of the Gale River 2 stage of

Lake Ammonoosuc. This channel crosses US Route 3 in several places southwest of Twin Mountain village.

Water entered Lake Ammonoosuc not only from the melting glacier but also from the early Ammonoosuc River and smaller streams draining the surrounding mountains, as shown by Lougee's diagram. The position of the lake shifted westward – and its surface elevation dropped – as the ice margin retreated down the valley toward Bethlehem. Eventually it drained completely and the upper reach of the Ammonoosuc River joined with the lower part of the river that flows southwest from Littleton. Following the disappearance of Lake Ammonoosuc, flood plain and alluvial fan deposits have accumulated on the old lake floor.

Figure 6.
Glacial Lake Ammonoosuc - Gale River 2 stage
(after Lougee, 1940)



Cog Railway

The steam engine and passenger car displayed next to the Carroll visitor center (Figure 7) were recently retired from the nearby Mount Washington Cog Railway. This famous tourist attraction is the world's oldest mountain-climbing cog railway, having been completed in 1869. The New Hampshire legislature approved plans to build the railway, though they thought it was an impossible task. One legislator suggested that it "should not only be given a charter up Mount Washington but also to the moon"!

Similar early cog railways were built on Pikes Peak, Mount Rigi (Switzerland) and other mountain locations. They are called "rack railways" because they have a cog wheel on the engine that meshes with a center rack rail on the track. The siding switches are thus quite complicated! Mt. Washington's "Cog" operation has included several generations and designs of steam engines, much to the delight of both tourists and rail fans. Each engine has a name, and they are listed along with their current status in a Wikipedia article about the Cog: http://en.wikipedia.org/wiki/Mount_Washington_Cog_Railway.

In the early 1900s, regular train service carried passengers right to the base of Mount Washington, where they could simply cross the station platform and board the Cog to the summit! Starting in 2008, new diesel-fueled Cog engines began to replace the coal-burning steam locomotives. A single daily steam train still climbs the mountain, with most of the other old locomotives kept in reserve.

Figure 7 – *Cog steam engine and passenger car on display next to the Carroll visitor center.*

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STOP 5: Mt. Washington Scenic Overlook

If the weather is clear on the day of the field trip, this stop will provide us with a spectacular view of the entire Presidential Range. The summit of Mt. Washington (6,288 ft.; with its buildings, towers, and the Mt. Washington Cog Railway ascending its slopes) dominates the view with Mts. Clay (5,533 ft.), Jefferson (5,712 ft.), and Adams (5,774 ft.) to its left. To its right are the peaked summit of Mt. Monroe (5,372 ft.), the subdued ridge of Mt. Franklin (5,001 ft.), the dome of Mt. Eisenhower (4,760 ft.), and the long ridge of Mt. Pierce (4,312), named for the only NH "native son" to serve as a U.S. President: Franklin Pierce. This view is one of the most-often photographed scenes in the White Mountains, especially if graced with some snow and early-Autumn foliage below timberline.

The peaks to the left of Mt. Monroe, except for Mt. Clay, are underlain by high-grade and thus highly erosion-resistant metasedimentary members of the Devonian Littleton Formation. Those to its right are underlain by the lower-grade and less-resistant metasedimentary members of the Silurian Rangeley Formation (see Eusden, et al., 2013). As indicated at Stop 3, Mt. Washington was thinly covered by the last ice sheet (Late Wisconsinan), suggesting that here at Stop 5, there was about 4,500 feet of ice overhead at the Last Glacial Maximum.

Any stop at this location needs to address the spectacular Mount Washington Hotel in the foreground, one of the last of the "grand hotels" in the White Mountains. Opened in 1902 and operated more or less continuously since, the Hotel and its elegant 'turn-of-the century" ambiance have played host to many famous (and some infamous) guests, along with the Bretton Woods International Monetary Conference in 1944 that established the World Bank and set the stage for the economic recovery that followed World War II. The Hotel and its surrounding Resort are today operated by Omni Hotels & Resorts. It was recently host to the record 1,150 attendees at the Annual Meeting of the Northeastern Section of the Geological Society of America.

As we proceed south from Stop 5 to Stop 6 (Willey House Site), we will pass through the initial spillway of ancestral Glacial Lake Ammonoosuc (described in Stop 4). This spectacular spillway is the narrow cleft we will pass through about ½ mile south of the Appalachian Mountain Club's Highland Center at the height of land on US Route 302. The cleft has been substantially modified since immediate post-glacial time by roadway and railway construction through it since the early 19th Century, but it is clear an enormous volume of meltwater was funneled toward and through it when the ice sheet had begun to move north of the area. Evidence of this can be seen from the right-hand sides of the buses, just after passing through the cleft, in the form of the huge but now wholly abandoned plunge pool located immediately south of and below the cleft.

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STOP 6: Willey House

Crawford Notch and the Willey Slide

As indicated at Stop 3, Crawford Notch is one of the deepest best developed glacial troughs in the region (Figure 1). As with Franconia Notch, its deeply developed glacially eroded cross-section is distinctive, being the result of several episodes of continental glaciation passing through and eroding its flanks.

Figure 1 - *Crawford Notch looking south from Mt. Willard (873 m; 2,865 ft.). The debris fan of the Willey Slide lies above and to the right of the south end of the pond visible to the left of the highway in the center of the photograph (elev. ~ 400 m; 1,320 ft.). The highway curves to the left and then over a combination of this debris fan and several others from similar avalanche tracks to its north.*

The Willey Slide occurred during a torrential rain event in 1826 and was responsible for the deaths of all the members of the unfortunate Willey Family who had homesteaded on the floor of the Notch at the eventual location of its debris fan. The story of this tragic event was made famous at the time by Nathaniel Hawthorne in his short story entitled "The Ambitious Guest"

(1835), but it has been revisited since by numerous authors. Interested readers can consult Google for this extensive listing of publications. Here we discuss the general rheology of the slide and its deposits, but a bit of the story is needed to properly set the stage.

Upon hearing the noise of the approaching debris avalanche above their house, the family decided to evacuate and seek refuge at locations beyond what they anticipated would be the track of the avalanche. They did so but were quickly buried, along with their farm hands and livestock, by two debris streams that separated around "a large boulder" located just above the house (Hawthorne, 1835). Had they stayed in the house, they would have been spared by this division of debris streams and tragedy would have been averted. All of their remains were subsequently recovered except those of one child who still "rests in peace" somewhere beneath the surface.

Recent STATEMAP surficial geologic mapping of the immediate area for the U.S. and N.H. Geological Surveys (Fowler, 2012) has identified the specific features of this avalanche track and helps better establish what actually happened during the slide. The debris avalanche followed a single track downslope from its initiation point about 1 km (1/2 mi.) and about 500 m (1,650 ft.) above on the slopes of Mt. Willard. At a point approximately 150 m (500 ft.) above the house location, a series of subtle bedrock promontories separated the fast-moving debris flow into the two streams that moved past the house to the north and south. This subtle "divide" occurs on the slope well above and behind the "large boulder" assumed to have spared the house. Work in the area shows this track system has since been frequently (and recently) remobilized by comparatively small-volume debris flows with detritus following the same tracks as the 1826 slide. A short walk up and behind the observation platform at the "large boulder" shows these two debris flow channels and their abutting levees. The slopes laterally below the platform show the mixed bouldery-cobbly and till-based debris that comprised the original and later slides.

Publicity about the "tragedy of the Willey Slide", along with the more or less contemporaneous Owl's Head Slide to the north in Jefferson, NH and the region's scenery, aroused great public interest. This curiosity encouraged regional railroad firms, along with local hotel and livery services, to develop excursions and tours of these and other sites during the mid to late 19th Century. This was the beginning of the now robust automobile-based tourist economy in the White Mountains.

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Drive-By Site C

US Route 302 Bridge over the Sawyer River in Harts Location, NH

Figure 1 - *US Route 302 bridge the day after the tropical storm event. Note the large dip in guardrail from undermined abutment.*

Another bridge impacted by Tropical Storm Irene in 2011 (described in Drive-By Site B for the Loon Mountain Bridge) was the US Route 302 Bridge over the Sawyer River in Harts Location, NH. The bridge, built in 1990 to replace an older structure, was a modern, single span bridge with a 95 foot wide opening. The bridge superstructure consisted of a concrete slab on steel girders. The bridge substructure consisted of full height concrete abutments with spread footings founded on alluvial sands and gravel. There was no history of severe flooding or problems with the bridge. Just downstream is a single span railroad bridge with granite block abutments, constructed over 140 years ago.

Figure 2 - *A close up of the scour that occurred behind the north abutment with the guardrail suspended in the air, the utilities cut, and approach slab fallen into the hole.*

Normally, the 9.1 mile long Sawyer River has small flows in a cobble and boulder lined channel that is about 15 feet wide and runs only a few feet deep. On the day Tropical Storm Irene hit, the river nearly overtopped the bridge. The main force of the water was directed at the north abutment because of a bend in the river about 300 feet upstream. The Sawyer River collects runoff from the White Mountains to the west and south, which have significant height, so runoff concentrations were rapid and fast moving. As a result of the flash flooding, the bridge suffered severe scouring effects. Both of the abutments were undermined by the rush of water, and a large quantity of material behind the north abutment was lost. The approach slab fell, buried utilities were cut, and the road was severely damaged. The abutments settled 6 to 18 inches from scour holes formed below them. The abutments and footings cracked from the settlement, the girders twisted and the deck cracked. The highway bridge was a total loss with no way to salvage any part of it. The railroad bridge just downstream suffered some minor scouring at its south abutment, but was spared any major damage because it was shielded by the highway bridge. The scouring at the railroad bridge was quickly and easily repaired by filling the scour hole with concrete.

Figure 3 - *Temporary bridge being launched east of damaged bridge.*

As a consequence of the damage to the bridge and along other portions of the highway, US Route 302 was closed for over two weeks. A temporary bridge was installed to allow the highway to reopen. The replacement US Route 302 Bridge is currently under construction through a design-build contract with Alvin J. Coleman & Sons of Conway, NH for a cost of 2.4 million dollars. The bridge design was performed by GM2 Associates of Concord, NH. The replacement bridge consists of a slab on steel girder deck, but the span has been increased to 135 feet to provide a larger hydraulic opening for the Sawyer River. The foundation depths have been extended 5 feet deeper than the previous bridge, but they still have a spread footing configuration. Very large rip rap stones, with a minimum dimension of 3.4 feet, were specified at the abutments as a scour countermeasure.

A deep foundation configuration was considered for the bridge in the pre-bid, conceptual design phase by the NHDOT, but subsurface conditions at the site were found to be very problematic. Test borings were drilled as deep as 121 feet without hitting glacial till or bedrock at the site – an unusual condition in NH. Only alluvial and glacial drift materials were encountered in the test borings, and these deposits consisted of sand, gravel, cobbles and boulders. Representative soil densities were difficult to obtain in the test borings that employed Standard Penetration Tests because of the cobbles and boulders, which were nested in several layers as much as 30 feet thick!

Figure 4 – *New, deeper abutments (14 feet) are also protected by large rip rap stones.*

Driven piles were not possible at the site. Deep predicted scour depths made slender pile foundations, such as drilled micro-piles, unworkable because of potential unsupported pile lengths. Larger diameter drilled shafts were also impractical because of the numerous boulders present. After considering all alternatives and design standard requirements, a spread footing configuration was deemed the most economical solution by the NHDOT. The deeper spread footings protected by very large rip rap in conjunction with the wider bridge span were the recommended approach to provide cost effective scour resistance for the bridge.

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Acknowledgements

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