65TH HIGHWAY GEOLOGY SYMPOSIUM

JULY 7-10, 2014 | HILTON HOTEL | LARAMIE, WYOMING

PROCEEDINGS

Hosted By:
The Wyoming Department of Transportation
On Cover – Picture of Lake Marie in Snowy Range
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Passed away peacefully, at 61, on May 16, 2014, at Hospice of the Valley - Eckstein Center, Scottsdale, AZ. He was born to the late Michael and Frances Priznar, July 26, 1952 in Albany, New York. Nick graduated from the Keveny Academy in Cohoes, NY in 1970, and received a degree in Geology from the University of Arizona in 1979. He married Gail (Martell) of Sierra Vista, AZ in 1978 in Tucson, AZ, where they lived before relocating to Phoenix in 1989. Nick served in the U.S. Marine Corp (30 DEC 1971-16 DEC 1975), stationed in Yuma, AZ where he was honorably discharged. He was an exploration geologist with the Phelps Dodge Corporation and worked for the Arizona Department of Water Resources (1986-88), before becoming a Geologist with the Arizona Department of Transportation in 1988. He worked these past 26 years to ensure that the state's roadways, highways and bridges were safe and solid for those traveling them. Nick enjoyed taking trips with his wife to visit family, and explore and hike different parts of the country. He had a lifelong interest in rocks, gemstones and minerals, and he loved hunting, fishing and gardening. He was an active member of the Highway Geological Symposium. We, who knew Nick Priznar, and the state of Arizona, have lost an immeasurable treasure too soon, but our lives have been enriched for having known him.
65th ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

Laramie, Wyoming
July 7 – July 10, 2014

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Jim Coffin  Chief Engineering Geologist, WYDOT
Mark Falk  Assistant Chief Engineering Geology, WYDOT
James Dahill  Geological Supervisor, WYDOT
Kirk Hood  Geological Supervisor, WYDOT
David Vanderveen  Geological Supervisor, WYDOT
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Julie Francis, WYDOT Archeologist
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<th>Organization</th>
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<th>Phone</th>
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<th>Email</th>
</tr>
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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, 64 consecutive annual meetings have been held in 33 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

<table>
<thead>
<tr>
<th>No.</th>
<th>Year</th>
<th>HGS Location</th>
<th>No.</th>
<th>Year</th>
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<tbody>
<tr>
<td>1st</td>
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<tr>
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<td>4th</td>
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<td>5th</td>
<td>1954</td>
<td>Columbus, OH</td>
<td>6th</td>
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<td>1956</td>
<td>Raleigh, NC</td>
<td>8th</td>
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<td>9th</td>
<td>1958</td>
<td>Charlottesville, VA</td>
<td>10th</td>
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<td>1960</td>
<td>Tallahassee, FL</td>
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<td>13th</td>
<td>1962</td>
<td>Phoenix, AZ</td>
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<td>15th</td>
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<td>Rolla, MO</td>
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<td>1965</td>
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<td>25th</td>
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<td>Redding, CA</td>
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<td>65th</td>
<td>2014</td>
<td>Laramie, WY</td>
<td>66th</td>
<td>2015</td>
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</tr>
</tbody>
</table>

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 interest. To cite a few examples: in Wyoming (1973), the group viewed landslides in the Big Horn Mountains; Florida's trip (1976) included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generation site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center. The Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central 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 Jerроме, 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.

The 2013 field trip of New Hampshire highlighted the topography and geologic remnants left by the Pleistocene glaciations that fully retreated approximately 12,000 years ago. The field trip included stops at various overlooks of glacially-carved valleys and ranges; the Old Man of The Mountain Memorial Plaza, which is a tribute to the famous cantilevered rock mass in the Franconia Notch that collapsed on May 3, 2003; lacustrine deposits and features of the Glacial Lake Ammonoosuc; views of the Presidential Range; bridges damaged during Tropical Storm Irene in August 2011; and the Willey Slide, location in the Crawford Notch where all members of the Willey family homestead were buried by a landslide in 1826.

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 34 persons have been granted Emeritus status. Fourteen 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 were dedicated to Earl Wright and Bill Lovell. The 2014 Proceedings of the 65th HGS held in Laramie, Wyoming are dedicated to Nicholas Michiel Priznar,
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*
George S. Meadors, Jr.*
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.*

<table>
<thead>
<tr>
<th>Name</th>
<th>Year</th>
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<tbody>
<tr>
<td>Hugh Chase*</td>
<td>1970</td>
</tr>
<tr>
<td>Tom Parrott*</td>
<td>1970</td>
</tr>
<tr>
<td>Paul Price*</td>
<td>1970</td>
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<tr>
<td>K.B. Woods*</td>
<td>1971</td>
</tr>
<tr>
<td>R.J. Edmondson*</td>
<td>1972</td>
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<td>C.S. Mullin*</td>
<td>1974</td>
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<td>A.C. Dodson*</td>
<td>1975</td>
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<td>Burrell Whitlow*</td>
<td>1978</td>
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<td>Bill Sherman</td>
<td>1980</td>
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<tr>
<td>Virgil Burgat*</td>
<td>1981</td>
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<tr>
<td>Henry Mathis</td>
<td>1982</td>
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<tr>
<td>David Royster*</td>
<td>1982</td>
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<tr>
<td>Terry West</td>
<td>1983</td>
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<tr>
<td>Dave Bingham</td>
<td>1984</td>
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<tr>
<td>Vernon Bump</td>
<td>1986</td>
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<tr>
<td>C.W. &quot;Bill&quot; Lovell*</td>
<td>1989</td>
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<td>Joseph A. Gutierrez</td>
<td>1990</td>
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<tr>
<td>Willard McCasland</td>
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<td>W.A. &quot;Bill&quot; Wisner</td>
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<td>David Mitchell</td>
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<tr>
<td>Harry Moore</td>
<td>1996</td>
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<td>Earl Wright*</td>
<td>1997</td>
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<td>Russell Glass</td>
<td>1998</td>
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<td>Harry Ludowise*</td>
<td>2000</td>
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<tr>
<td>Sam Thornton</td>
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<tr>
<td>Bob Henthorne</td>
<td>2004</td>
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<tr>
<td>Mike Hager</td>
<td>2005</td>
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<tr>
<td>Joseph A. Fischer</td>
<td>2007</td>
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<tr>
<td>Ken Ashton</td>
<td>2008</td>
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<tr>
<td>A. David Martin</td>
<td>2008</td>
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<tr>
<td>Michael Vierling</td>
<td>2009</td>
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<tr>
<td>Richard Cross</td>
<td>2009</td>
</tr>
<tr>
<td>John F. Szturo</td>
<td>2010</td>
</tr>
<tr>
<td>Christopher Ruppen</td>
<td>2012</td>
</tr>
<tr>
<td>Jeff Dean</td>
<td>2012</td>
</tr>
</tbody>
</table>

(*Deceased)
## HIGHWAY GEOLOGY SYMPOSIUM
### Previous, Present, and Future Symposium

#### Contact List

<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Name</th>
<th>Phone</th>
<th>Email</th>
</tr>
</thead>
<tbody>
<tr>
<td>2009</td>
<td>New York</td>
<td>Mike Vierling</td>
<td></td>
<td><a href="mailto:Rocdoc1959@gmail.com">Rocdoc1959@gmail.com</a></td>
</tr>
<tr>
<td>2010</td>
<td>Oklahoma</td>
<td>Jeff Dean</td>
<td></td>
<td><a href="mailto:jdean@odot.org">jdean@odot.org</a></td>
</tr>
<tr>
<td>2011</td>
<td>Kentucky</td>
<td>Henry Mathis</td>
<td>859-455-8530</td>
<td><a href="mailto:hmathis@iglou.com">hmathis@iglou.com</a></td>
</tr>
<tr>
<td>2012</td>
<td>California</td>
<td>Bill Webster</td>
<td>916-227-1041</td>
<td><a href="mailto:bill_webster@dot.ca.gov">bill_webster@dot.ca.gov</a></td>
</tr>
<tr>
<td>2013</td>
<td>New Hampshire</td>
<td>Krystle Pelham</td>
<td>603-271-1657</td>
<td><a href="mailto:kpelham@dot.state.nh.us">kpelham@dot.state.nh.us</a></td>
</tr>
<tr>
<td>2014</td>
<td>Wyoming</td>
<td>Jim Coffin</td>
<td>307-777-4205</td>
<td><a href="mailto:Jim.coffin@wyo.gov">Jim.coffin@wyo.gov</a></td>
</tr>
<tr>
<td>2015</td>
<td>Massachusetts</td>
<td>Peter Ingraham</td>
<td>603-668-0880</td>
<td><a href="mailto:pingraham@golder.com">pingraham@golder.com</a></td>
</tr>
</tbody>
</table>
65Th Annual
Highway Geology Symposium

Sponsors
The following companies have graciously contributed toward the sponsorship of the Symposium. The HGS relies on sponsor contributions for refreshment breaks, field trip lunches and other activities. We gratefully appreciate the contributions made by these generous sponsors.

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65th Annual Highway Geology Symposium

Agenda

Monday, July 7th

8:00 AM – 11:30 AM
  The Association of GeoHazard Professionals Committee Meeting / non-members welcome
  Location: Garden III

9:00 AM – 11:30 AM
  Transportation Research Board Mid-Year Geosynthetic Committee Meeting / non-members welcome
  Location: Garden I & II

11:00 AM – 5:00 PM
  Highway Geology Symposium Registration OPEN

12:30 PM – 5:00 PM
  Transportation Research Board: Technical Session “Geosynthetics in Roadway Design”
  Location: Salon A, B, and C

12:30 PM – 5:00 PM
  Poster Board Session – Open
  Location: Displayed throughout the Exhibitors Area

5:00 PM – 8:30 PM
  Highway Geology Symposium Exhibitor Area OPEN

5:15 PM – 6:30 PM
  HGS Steering Committee Meeting
  Location: Garden I & II

6:30 PM – 8:30 PM
  Ice Breaker Social
  Sponsored by: AMERITECH
  Location: Salon E (cash bars in exhibitor area)
Tuesday, July 8th

6:30 AM – 9:00 AM
   Breakfast
   Sponsored by: URETEK
   Location: Salon E

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

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

8:00 AM – 5:00 PM
   Poster Board Session – Open
   Location: Displayed throughout the Exhibitors Area

7:30 AM – 8:30 AM
   Welcome and Opening Remarks
   Jim Coffin, Wyoming Department of Transportation
   John Cox, WYDOT Director
   Martin Larsen, Wyoming Geologic Survey
   Location: Salon A through D

9:00 AM – 3:00 PM
   Highway Geology Symposium Guest Field Trip
   Transportation Sponsored by: Yeh and Associates
   Pick-up Location: West Conference Entrance of Hilton

   **Technical Sessions I**
   (moderator – George Machan)

8:30 AM – 8:55 AM
   Aggregate Freeze-Thaw Testing and D Cracking Field Performance 30 Years Later
   Author(s): Heather A. K. McLeod, Joshua Welge, Robert Henthorne

8:55 AM – 9:20 AM
   Falling Rocks-Harnessing the Schuylkill Expressway
   Author(s): Joseph Krupansky, Sarah McInnes

9:20 AM – 9:45 AM
   Mitigating Landslides in Wyoming Using Multiple Innovative Designs and Technologies
   Author(s): Ben Arndt, Kirk Hood, James Dahill
9:45 AM – 10:20 AM
**Morning Coffee Break**
Sponsored by: Gannett Fleming
Location: Intermingled throughout the Sponsors and Exhibitors

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**Technical Sessions II**
(moderator – Bob Henthorne)

10:20 AM – 10:45 AM
**Ground Anchor Selection, Design and Testing – Matching Ground Anchors to Their Desired Function and Testing Their Serviceability**
Author(s): Mark Telesnicki, Peter Ingraham

10:45 AM – 11:10 AM
**LiDAR-Based Rock Slope Assessment, State Route 8**
Author(s): Evan T. Lonstein, Andrew R. Blaisdell, Christopher L. Snow

11:10 AM – 11:35 AM
**Utilizing InSAR for Landslide Asset Management in Colorado**
Author(s): Ken Fergason, Bibhuti Panda, Danielle Williamson

11:35 AM – 12:00 PM
**Case Histories on Light Anchor Post System for Flexible Barriers**
Author(s): Giorgio Giacchetti, Ghislain Brunet, Alberto Grimod

12:00 PM – 1:00 PM
**Lunch**
Sponsored by: HI-TECH Rockfall Construction Inc.
Location: Salon E

(continued on next page)
Tuesday, July 8th (continued) 1:00 PM – 4:20 PM
Technical Sessions III & IV (after break)
(Note: Presentations in Italics are 20 minutes all others are 25 minutes)

**Location: Salon D**
Moderated by: John Szturo

1:00 PM – 1:25 PM
Geotech Tools – Geotechnical Solutions for Transportation Infrastructure - Implementation
Author(s): Vern Schaefer, Silas Nichols, Matt DeMarco, Ryan Berg

1:25 PM – 1:45 PM
*Assessing the Capabilities and Limitations of Terrestrial LiDAR, Terrestrial Photogrammetry, and Airborne LiDAR for Mapping Differential Slope Changes*
Author(s): Matthew Lato

1:45 PM – 2:10 PM
Using New and Old Technologies for an Economical Approach to Rock Stabilization on TN I-75
Author(s): Saieb Haddad, Daniel Journeaux, Frank Amend

2:10 PM – 2:30 PM
*Anchored Shear Piles for Landslide Mitigation: Design Approach and Instrumented Performance*
Author(s): Thomas Westover

2:30 PM – 3:00 PM
**Afternoon Refreshment Break**
Sponsored by: HNTB
Located: Intermingled throughout the Sponsors and Exhibitors
Moderated by: Phil Sirles

3:00 PM – 3:20 PM
*Using Time-Domain Reflectometry to Monitor Subsurface Void Propagation*
Author(s): Joseph Renner, Ed Lindgren, Mark Vessely

3:20 PM – 3:45 PM
Author(s): Terry West

**Location: Salon A, B, C**
Moderated by: Krystle Pelham

1:00 PM – 1:25 PM
Emergency Rockfall Mitigation BNSF Railway, Yakima Subdivision, Washington
Author(s): William CB Gates, Glen Gaz

1:25 PM – 1:45 PM
*Performance of Flexible Debris Flow Barriers in a Narrow Canyon*
Author(s): Simon Boone, William F. Kane

1:45 PM – 2:10 PM
Practical Consideration for the Design and Construction of Landslide Mitigation Using Horizontal Drains
Author(s): Timothy Pfieffer

2:10 PM – 2:30 PM
*Intelligent Compaction for Roadway Construction and Quality Assurance*
Author(s): Christopher Savan, Kam Ng, Khaled Ksaibati

2:30 PM – 3:00 PM
**Afternoon Refreshment Break**
Sponsored by: HNTB
Located: Intermingled throughout the Sponsors and Exhibitors
Moderated by: Peter Ingraham

3:00 PM – 3:20 PM
*Review of Bridge Approach Slab Systems in the United States*
Author(s): Seyed Yashar Yasrobi, Kam W. Ng, Thomas V. Edgar, Michael Menghini

3:20 PM – 3:45 PM
The Full Scale Mechanics of Slope Stabilization
Author(s): Steve Mumma

3:45 PM – 4:05 PM
*Evaluation and Stabilization of the Bret Landslide, Big Horn County, WY*
Author(s): Brett Arpin, Kirk Hood, Ben Arndt

3:45 PM – 4:05 PM
*Friction Losses in Tieback Anchors used for Landslide Stabilization*
Author(s): Benjamin Turner
Tuesday, July 8\textsuperscript{th} (continued)

4:05 PM–4:20 PM  
**Field Trip Preview Presentation by Mark Falk**  
Location: Salons A through D

4:30 PM - **Load buses for optional Dinner Show**  
Pick-up Location: West Conference Entrance of Hilton

5:00 PM - **Depart for Bit-O-Wyo Ranch for Dinner Show**  
Refreshments Sponsored by: Golder and Associates

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**Wednesday, July 9\textsuperscript{th}**

6:00 AM – 7:30 AM  
**Breakfast**  
Sponsored by: URETEK  
Location: Salon E

7:00 AM **Load buses for Field Trip**  
Pick-up Location: West Conference Entrance of Hilton

7:30 AM – 5:00 PM  
**Highway Geology Symposium Field Trip**  
Lunch Sponsored by: Geobrugg  
Afternoon Beverages sponsored by Golder and Associates  
(NO GLASS ALLOW INSIDE BUSES)

8:00 AM – 4:00 PM  
**FHWA Meeting**  
Location: Salon A

5:30 PM – 6:30 PM  
**Highway Geology Symposium Social Hour**  
Sponsored by: Olson Engineering Corp.  
Location: Grand Ballroom

6:30 PM – 9:30 PM  
**Highway Geology Symposium Banquet Dinner**  
Location: Grand Ballroom  
Keynote Speaker: John Waggener
Thursday, July 10th

6:30 AM – 9:00 AM

Breakfast
Sponsored by: URETEK
Location: Salon E

8:00 AM – 12:00 PM

Highway Geology Symposium Exhibitor Area OPEN

Technical Session V
(moderator – Steve Sweeney )
(Note: Presentations in Italics are 20 minutes all others are 25 minutes)

7:30 AM – 7:50 AM
Change we can Count On? Evaluating the Merits of Traditional and Modern Methods in Conduction Preliminary Assessment of Geohazards
Author(s): Edward Barefield

7:50 AM – 8:15 AM
Double Nickel Micropile Landslide Stabilization
Author(s): Kirk Hood, Jamie Martens, John Szturo

8:15 AM – 8:40 AM
Rock Bolting in Sensitive Environments
Author(s): Kyle A. Kershaw, Mark J. Vessely

8:40 AM – 9:05 AM
Author(s): N.I. Norrish, T.C. Badger, E.L. Smith

9:05 AM – 9:30 AM
Case Study in Using wicking geotextile to solve frost heave at the Pioneer Scenic Mountain Byway in Dillon Montana
Author(s): Mark Sikkema, James B. Carpita

9:30 AM – 9:55 AM
Improving Construction Safety along Highway US-34 Post September 2013 Flooding
Author(s): Dennis Sack, Phil Sirles, Scott Walerk

9:55 AM – 10:15 AM
Morning Coffee Break
Sponsored by: Access Limited
Location: Intermingled throughout the Sponsors and Exhibitors
Technical Session VI
(moderator – Mike Vierling)

10:15 AM – 10:40 AM
Geologic Causation and Geotechnical Mitigation Design for the US89 Landslide Disaster, Bitter Springs, Arizona
Author(s): William V. McCormick, Keith H. Dahlen

10:40 AM – 11:05 AM
Advances in Design and Construction of Large Diameter Drilled Shafts in Rock
Author(s): John P. Turner (paper will not be included on USB drive)

11:05 AM – 11:30 AM
Rockfall source detection & volume measurement from autonomous UAV-acquired Photogrammetry: A case study from a transportation corridor in northern Ontario, Canada
Author(s): Dave Gauthier, Jean Hutchinson, Matt Lato, David F. Wood, A.J. Morris

11:30 AM – 11:55 AM
Geotechnical Challenges in Laboratory Investigation of Bridge Abutment Scour
Author(s): Kam Hg, Ram Chakradhar, Robert Ettema, Edward K. Kempema

11:55 AM – 12:05 PM
Closing remarks/Adjournment
Jim Coffin

1:30 PM – 5:00 PM
FHWA Meeting
Location: Salon A
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Geosynthetics in Roadway Design

Laramie, Wyoming

Monday, July 7, 2014

Presided by

FHWA – Resource Center

Sponsored by:

AFS70 Committee on Geosynthetics
## Agenda

### Transportation Research Board Geosynthetics Committee

<table>
<thead>
<tr>
<th>Start Time</th>
<th>Topic</th>
<th>Spokesperson</th>
</tr>
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<tbody>
<tr>
<td>9:00 – 11:30 AM</td>
<td>Mid-Year Meeting</td>
<td>Barry Christopher</td>
</tr>
<tr>
<td>11:30 AM – 12:30 PM</td>
<td>Lunch</td>
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### Geosynthetics in Roadway Design Session

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<thead>
<tr>
<th>Start Time</th>
<th>Topic</th>
<th>Spokesperson</th>
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</thead>
<tbody>
<tr>
<td>12:30 – 1:00 PM</td>
<td>History of Geosynthetic Use on Forest Roads</td>
<td>Gordon Keller, Geotechnical Consultant (retired US Forest Service)</td>
</tr>
<tr>
<td>1:00 – 2:00 PM</td>
<td>Overview of GeoTechTools Geosynthetic Design Applications</td>
<td>Barry Christopher, Christopher Consultants</td>
</tr>
<tr>
<td>2:00 – 2:25 PM</td>
<td>Geotechnical Challenges and Experience with Geosynthetics at Wyoming DOT</td>
<td>L.J. Maillet and Jim Coffin, WYDOT</td>
</tr>
<tr>
<td>2:25 – 2:50 PM</td>
<td>Break</td>
<td></td>
</tr>
<tr>
<td>Start Time</td>
<td>Topic</td>
<td>Spokesperson</td>
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<tr>
<td>3:15 – 3:40 PM</td>
<td>Enhanced Lateral Drainage in Pavement Systems</td>
<td>Jorge Zornberg, University of Austin – Texas</td>
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<tr>
<td>3:40 – 4:05 PM</td>
<td>Deep Patch Repair with Geosynthetics</td>
<td>Brian Collins, Western Federal Lands Highway Division</td>
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<td>4:05 – 4:30 PM</td>
<td>Field Performance of Geosynthetics Used As Subgrade Stabilization</td>
<td>Eli Cuelho, Western Transportation Institute, Montana State University</td>
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<tr>
<td>4:30 – 5:00 PM</td>
<td>Panel Discussion</td>
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<td>5:00 PM</td>
<td>Adjourn</td>
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</table>
John Waggener was born and raised a 5th Generation Wyomingite in the I-80 town of Green River. He earned his BA and MA degrees in geography from the University of Wyoming. His love for Wyoming history led him toward a career as a faculty reference archivist at the American Heritage Center, the archive of the University of Wyoming, where he has been since 2001. His family traveled around the state a lot, and it was here where he developed his love for highway history and highway maps. His master’s thesis, *Putting Wyoming on the Map: the History of the Official Wyoming Highway Map*, covers the entire of the Wyoming map. His more recent historical research has varied widely from the history of the Wyoming license plate to the history of Interstate 80 between Laramie and Rawlins.

Opened on October 3, 1970, this 77-mile stretch of highway closed just four days later. Within one year of its opening, the road became known as the Snow Chi Minh Trail. It has become somewhat of a legendary stretch of highway because of poor winter-time road conditions due to the presence of snow and strong winds in this area on the north end of the Medicine Bow Mountains. The highway presented many challenges to the Wyoming Department of Transportation, and the road became a testing ground for various technologies such as snow fences, variable information signs, road and travel report systems, and snow plow and sander designs. Though technology has tamed conditions on this piece of I-80, it proves to be a difficult stretch of highway to maintain.

![Snow Chi Minh Trail](image-url)
Tuesday, July 8th – morning sessions

1. 8:30 AM – 8:55 AM
   Aggregate Freeze-Thaw Testing and D Cracking Field Performance 30 Years Later
   Author(s): Heather A. K. McLeod, Joshua Welge, Robert Henthorne

2. 8:55 AM – 9:20 AM
   Falling Rocks-Harnessing the Schuylkill Expressway
   Author(s): Joseph Krupansky, Sarah McInnes

3. 9:20 AM – 9:45 AM
   Mitigating Landslides in Wyoming Using Multiple Innovative Designs and Technologies
   Author(s): Ben Arndt, Kirk Hood, James Dahill

4. 10:20 AM – 10:45 AM
   Ground Anchor Selection, Design and Testing – Matching Ground Anchors to Their Desired Function and Testing Their Serviceability
   Author(s): Mark Telesnicki, Peter Ingraham

5. 10:45 AM – 11:10 AM
   LiDAR-Based Rock Slope Assessment, State Route 8
   Author(s): Evan T. Lonstein, Andrew R. Blaisdell, Christopher L. Snow

6. 11:10 AM – 11:35 AM
   Utilizing InSAR for Landslide Asset Management in Colorado
   Author(s): Ken Fergason, Bibhuti Panda, Danielle Williamson

7. 11:35 AM – 12:00 PM
   Case Histories on Light Anchor Post System for Flexible Barriers
   Author(s): Giorgio Giacchetti, Ghislain Brunet, Alberto Grimod

Tuesday, July 8th – afternoon split sessions

8. 1:00 PM – 1:25 PM (Salon D)
   Geotech Tools – Geotechnical Solutions for Transportation Infrastructure - Implementation
   Author(s): Vern Schaefer, Silas Nichols, Matt DeMarco, Ryan Berg

9. 1:00 PM – 1:25 PM (Salon A, B, C)
   Emergency Rockfall Mitigation BNSF Railway, Yakima Subdivision, Washington
   Author(s): William CB Gates, Glen Gaz
10. 1:25 PM – 1:45 PM (Salon D)
Assessing the capabilities and limitation of terrestrial LiDAR, terrestrial Photogrammetry, and airborne LiDAR for mapping differential slope changes
Author(s): Matthew Lato

11. 1:25 PM – 1:45 PM (Salon A, B, C)
Performance of Flexible Debris Flow Barriers in a Narrow Canyon
Author(s): Simon Boone, William F. Kane

12. 1:45 PM – 2:10 PM (Salon D)
Using New and Old Technologies for an Economical Approach to Rock Stabilization on TN I-75
Author(s): Saieb Haddad, Daniel Journeaux, Frank Amend

13. 1:45 PM – 2:10 PM (Salon A, B, C)
Practical Consideration for the Design and Construction of Landslide Mitigation Using Horizontal Drains
Author(s): Timothy Pfeiffer

14. 2:10 PM – 2:30 PM (Salon D)
Anchored Shear Piles for Landslide Mitigation: Design Approach and Instrumented Performance
Author(s): Thomas Westover

15. 2:10 PM – 2:30 PM (Salon A, B, C)
Intelligent Compaction for Roadway Construction and Quality Assurance
Author(s): Christopher Savan, Kam Ng, Khaled Ksaibati

16. 3:00 PM – 3:20 PM (Salon D)
Using Time-Domain Reflectometry to Monitor Subsurface Void Propagation
Author(s): Joseph Renner, Ed Lindgren, Mark Vessely

17. 3:00 PM – 3:20 PM (Salon A, B, C)
Review of Bridge Approach Slab Systems in the United States
Author(s): Seyed Yashar Yaserobi, Kam W. Ng, Thomas V. Edgar, Michael Menghini

18. 3:20 PM – 3:45 PM (Salon D)
Author(s): Terry West

19. 3:20 PM – 3:45 PM (Salon A, B, C)
The Full Scale Mechanics of Slope Stabilization
Author(s): Steve Mumma

20. 3:45 PM – 4:05 PM (Salon D)
Evaluation and Stabilization of the Bret Landslide, Big Horn County, WY
Author(s): Brett Arpin, Kirk Hood, Ben Arndt

21. 3:45 PM – 4:05 PM (Salon A, B, C)
Friction Losses in Tieback Anchors used for Landslide Stabilization
Author(s): Benjamin Turner
Thursday, July 10th – morning sessions

22. 7:30 AM – 7:50 AM
   *Change we can Count On? Evaluating the Merits of Traditional and Modern Methods in Conduction Preliminary Assessment of Geohazards*
   Author(s): Edward Barefield

23. 7:50 AM – 8:15 AM
   *Double Nickel Micropile Landslide Stabilization*
   Author(s): Kirk Hood, Jamie Martens, John Szturo

24. 8:15 AM – 8:40 AM
   *Rock Bolting in Sensitive Environments*
   Author(s): Kyle A. Kershaw, Mark J. Vessely

25. 8:40 AM – 9:05 AM
   *Utilization of Displacement Monitoring to Modify Rock Slope Designs during Construction – I-90, Washington State*
   Author(s): N.I. Norrish, T.C. Badger, E.L. Smith

26. 9:05 AM – 9:30 AM
   *Case Study in Using wicking geotextile to solve frost heave at the Pioneer Scenic Mountain Byway in Dillon Montana*
   Author(s): Mark Sikkema, James B. Carpita

27. 9:30 AM – 9:55 AM
   *Improving Construction Safety along Highway US-34 Post September 2013 Flooding*
   Author(s): Dennis Sack, Phil Sirles, Scott Walerk

28. 10:15 AM – 10:40 AM
   *Geologic Causation and Geotechnical Mitigation Design for the US89 Landslide Disaster, Bitter Springs, Arizona*
   Author(s): William V. McCormick, Keith H. Dahlen

29. 10:40 AM – 11:05 AM
   *Advances in Design and Construction of Large Diameter Drilled Shafts in Rock*
   Author(s): John P. Turner (paper will not be included on USB drive)

30. 11:05 AM – 11:30 AM
   *Rockfall source detection & volume measurement from autonomous UAV-acquired Photogrammetry: A case study from a transportation corridor in northern Ontario, Canada*
   Author(s): Dave Gauthier, Jean Hutchinson, Matt Lato, David F. Wood, A.J. Morris

31. 11:30 AM – 11:55 AM
   *Geotechnical Challenges in Laboratory Investigation of Bridge Abutment Scour*
   Author(s): Kam Hg, Ram Chakradhar, Robert Ettema, Edward K. Kempem
Aggregate Freeze-Thaw Testing and D-Cracking Field Performance

30 Years Later

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Acknowledgements

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Disclaimer

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ABSTRACT

Premature deterioration of concrete pavement due to D-cracking has been a problem in Kansas since the 1930s. Kansas geology includes mineable limestone coarse aggregates with variable durability in the eastern portion of the state. Due to this variability and historically poor D-cracking field performance, the Kansas DOT initiated intensive identification and tracking of individual mined beds, as well as frequent durability testing during production in the 1980s. D-cracking field performance of concrete pavements containing limestone coarse aggregates was investigated in 2010-2012. Results of this investigation indicate that the rate of D-cracking decreased, but the minimum rate of D-cracking presence in concrete pavements is more than 30%.

In reaction to these results, KDOT implemented changes aimed at mitigating the risk of D-cracking. Implementation actions included increasing the number of freeze-thaw cycles for aggregate in concrete prisms from 300 to 660 cycles, freeze-thaw testing of all aggregate types (not just limestone) in concrete, focusing aggregate sampling at the point of concrete production, and including an “acceptable field-performance history” criterion for concrete aggregates. Ongoing research is being conducted to develop new methods to identify durable aggregates and faster testing techniques.
INTRODUCTION

D-Cracking
Deterioration of concrete pavements due to D-cracking has been a known problem since the 1930s. D-cracking is concrete pavement distress due to freezing and thawing of frost susceptible coarse aggregates containing water. The aggregate particles fracture internally, with cracks extending from the aggregate particles into the concrete paste and progress throughout the pavement slab (1,2,3). Typically, well-developed D-cracking appears on the surface of the concrete as closely-space, crescent-shaped and concentric hairline cracks located adjacent to and following roughly parallel to joints, cracks, or free edges (4). The cracks may be interconnected, giving the appearance similar to map cracking. Under magnification, D-cracked aggregates in concrete appear as internally fractured with cracks extending from the aggregate into the surrounding concrete paste.

Significance
Premature deterioration of concrete pavements due to D-cracking represents a significant cost to the State of Kansas. The cost of reconstructing a 2-lane concrete pavement can range from $0.8–1.15 million per lane-mile (5) and the cost of resurfacing/overlay (10-year life) can range from $300–500 thousand per lane-mile (6). Earlier than expected maintenance and restoration actions significantly increase costs to owners. Based on previous KDOT experience, the time from the identification of D-cracking to a necessary action (generally patching and an overlay) is approximately 4–6 years (7,8).

Causes and Efforts to Prevent D-Cracking
It is generally recognized that freeze-thaw deterioration of frost susceptible limestone coarse aggregate containing water is the cause of D-cracking (9, 10, 11, 12, 13, 14, 15, 16, 17, 18). With approximately 78 F/T cycles (drops below 32°F) and 33 “hard” F/T cycles (drops below 23°F) per year (19), D-cracking has been a concern in Kansas since the 1930s. Multiple studies investigating the causes and prevention of D-cracking have been completed across many states with freeze-thaw conditions. KDOT alone has conducted five major investigations over the past 80 years (17, 18, 19, 20, 21, 22, 23, 24). As a result of previous studies many DOTs in freeze-thaw climates, including Kansas, implemented durability requirements for aggregates used in concrete. Many acceptance and testing approaches have been taken, but generally aggregate sources are prequalified based on freeze-thaw testing of the aggregate and of concrete containing the aggregate.

Overview of Kansas Aggregate Sources
Kansas has a wide variety of landscapes with 11 different geologic regions (physiographic provinces) each characterized by unique features and geologic history. For the purpose of analyzing Kansas aggregates used in concrete, several generalizations can be made. Variable limestone deposits are the dominant aggregate source in eastern Kansas, and alluvial sand-gravel deposits are the source of the majority of aggregates in western Kansas. Large sand deposits composed mostly of quartz are present in south central Kansas along the Arkansas and Kansas River valleys.
Eastern Kansas and North Central Kansas geologic history is dominated by the advancement and retreat of multiple shallow inland seas. These waters produced layered sedimentary deposits consisting largely of alternating layers of hard and soft rocks, mainly limestones, shales, and sandstones. The largest underlying strata in Eastern Kansas consist of the Pennsylvanian limestones (calcite, calcium carbonate, \( \text{CaCO}_3 \)) and shales (hardened, compacted clay or silt). These deposits generally exist as alternating, thinly-layered beds that are highly variable with location, generally dipping (sloping) to the west and northwest. The shales are soft, easily erodible and often occur as bedding planes between limestone deposits. Sandstone is also common in eastern and north central Kansas, and is often interbedded with limestone and shale. The Pennsylvanian limestones are the main source of minable aggregate for roughly the eastern third of Kansas.

Other mineral deposits are found in select regions of Kansas, including gypsum and salt (halite) in central and south central Kansas, and loess (fine-grained deposits consisting mainly of silt) and glacial erratics, such as quartzite, in the glaciated region of northeastern Kansas. These materials are not considered as feasible sources of aggregate for building materials.

**Historical D-Cracking Field Performance in Kansas**

Five D-cracking studies have been conducted on Kansas highways, in 1944–45, 1951–52, 1964–65, 1980, and 2010-2012. The first three studies were conducted on approximately 1200 miles, or nearly all of the concrete pavement on the state highway system. The fourth and fifth studies included 279 miles and 2177 lane miles of concrete pavement, representing approximately 39% and 69% of the bare concrete pavement on the state highway system, respectively.

The first studies used rating systems reporting a rating of “good” if 0–12% of the pavement panels (1 in 8 panels) exhibited D-cracking. Any rating above 0 exhibited some level of D-cracking. It is important to note that for all four surveys, a pavement rating of “good” did not mean an absence of D-cracking. Rather, it simply meant that the level of D-cracking was considered acceptable at the time. They all concluded that all limestones in Kansas are susceptible to D-cracking and that once D-cracking begins it cannot be stopped. These first surveys also each indicated improvement in the D-cracking field performance due to changes implemented in aggregate size and sources.

<table>
<thead>
<tr>
<th>Rating</th>
<th>Percentage of Panels D-cracked</th>
<th>Survey Year</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1944–45</td>
</tr>
<tr>
<td>Good (0–3)</td>
<td>0–12%</td>
<td>54%</td>
</tr>
<tr>
<td>Fair (3.1–6)</td>
<td>13–50%</td>
<td>8%</td>
</tr>
<tr>
<td>Poor (6.1–10)</td>
<td>Greater than 50%</td>
<td>38%</td>
</tr>
</tbody>
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*Rated as good or "otherwise"

Table 1. Results from Previous Kansas D-Cracking Studies
In 1979 FHWA required that KDOT to stop using D-cracking aggregate in federally funded projects. As a result, another study was initiated and new aggregate durability specifications were implemented in the 1980’s with the goal of achieving the initial 20-year design life. The structural design life for KDOT concrete pavements is 20 years (4), with two rehabilitation actions to achieve a total life cycle of 40 years.

The study surveyed pavements constructed between 1961 and 1974 (6–19 years old), and correlated quarry source with pavement performance. The 1980 survey results showed that 57% of all the concrete pavements (including those that did not contain limestone) were either overlaid or exhibited D-cracking before 20 years had elapsed. Because D-cracking was the predominant source of early deterioration at that time, it could be assumed that many of the overlays were required due to D-cracking.

Since the 1980 study, the requirements for an “acceptable condition” rating for pavements have increased significantly. It was found during the 2010-2012 study, that the condition of pavements labeled “good” in previous studies was currently considered unacceptable. Any D-cracking present was considered unacceptable in the 2010-2012 study. Also, the “fair” pavements according to the original rating systems do not currently exist in the state system because KDOT has taken corrective action before any concrete pavement is allowed to get to such a poor condition.

2012 D-CRACKING STUDY RESULTS

The 2010-2012 D-Cracking study provided a 20 to 30 year follow-up to the KDOT implementation (from 1981–1987) of the aggregate durability specifications in the 1980s. The objective was to study the D-cracking field performance of Portland Cement Concrete Pavements (PCCP) built in Kansas and whether they were achieving the intended 20-year design life. Field survey methods used are described previously (19). One hundred thirty-three PCCP projects were field performance evaluated, representing 73% of the current concrete state highway system in Kansas. The study was conducted on projects that contained limestone, were at least 10 years old, met specified criteria for length and location, and included pavements that had been overlaid. Over 230 quarries in Kansas have been evaluated for Class 1 status since 1980. Aggregates from 52 quarries were used to construct the projects surveyed in the 2010-2012 study. The rate of success was determined and the results compared with the aggregate source parameters. Results indicated that nearly one-third of the PCCP built in Kansas between 1981 and 2000 are exhibiting D-cracking before 20 years service. The specified testing did not fully predict failure, but it did reduce the rate of failure. Of utmost interest was the observation that a limited number of quarries were linked with a high percentage of the failures.

For the 2010-2012 study, results were presented on a project basis because each project represented a “decision” made; (i.e., source of coarse aggregate selection) for that project, regardless of the length and scope of the project. A rating of “yes” indicated that D-cracking was observed on the project and a rating of “no” indicated that D-cracking was not observed on the project. A project had a “Success” result if there was no D-cracking and the project had reached the 20-year design life or if there was D-cracking observed after the project had already reached the 20 years design life. A project had a “Fail” result if there was D-cracking observed before
the project reached the 20 year design life. If a project was less than 20 years old and D-cracking had not yet been observed, then the result was “Inconclusive” because the project had not yet reached the 20-year design life. Only after a non-D-cracking project reached 20 years old could it be considered a “Success” as there would still be time for D-cracking to become evident before 20 years.

Overall Analysis of Field Performance
Overall, there was a 31% failure rate, 37% success rate, and 32% rate of indeterminate projects (no D-Cracking and not yet 20 years old). The data indicated that D-cracking was observed on 54 of the 131 projects or 41%. There were 41 of the 131 projects, or 31%, that were less than 20 years old when D-cracking was observed and were termed “FAIL.” Thirteen projects, or 10%, were 20 years old or more when D-cracking was observed and were termed “PASS” As well as 35 projects, or 26%, that were 20 years or older and exhibited no D-cracking in 2010 and were also termed “PASS.” Forty-two projects, or 32%, exhibited no D-cracking and were less than 20 years old. These projects (exhibiting no D-cracking at an age less than 20 years) were termed “INCONCLUSIVE” because a “pass” cannot be determined until the pavement reaches the design life of 20 years. Fig. 1 illustrates the distribution of overall field performance.

![Pie chart showing distribution of passing, failing, and inconclusive results in the 2010-2012 study.](image)

Figure 1. Number of passing, failing, and inconclusive results in the 2010-2012 study.

The failure rate for the 2009–2010 Study represents the D-cracking failure rate for concrete pavements in Kansas. If the failure rate of the inconclusive (blue) projects turns out to be the same as the current failure rate of 31%, then the final failure rate for the full data set will be 41%.

It is clear that the rate of failure has increased with time. In addition to material variability, multiple outside influences were considered. The distribution of successes and failures over time is displayed in Fig. 2. The rate of failures during the 1980s (before the Comprehensive Highway
Program, CHP) is generally lower than during the 1990s (during the CHP). If the final results for the inconclusive projects during CHP turn out to be all “successes” (best possible result), the rate of success during the CHP would still be worse than for the pre-1990 era. In addition, between 1986 and 1989 quarry monitors were phased-out of being stationed in the quarries. The 1990 KDOT Specification Book dictates acceptance at the point of usage (at the project), where prior to that acceptance was at the quarry. The number of annual failures increased after the quarry monitors were removed from the quarries and acceptance was changed to the project.

It is important to note that field performance can be affected by the material itself (e.g. not F/T durable), by production issues (e.g., contamination, the addition of unacceptable material to crushers or stockpiles, etc.), or a combination of these factors. It is the general view of the 2010-2012 contributing authors that the new changes to inspection and monitoring may have some limited impact on field performance, but it is not likely to reduce the rate of D-cracking to an acceptable level.

**Figure 2. Survey results for survey projects by year of construction and legislative dates for CHP and CTP**

**Quarry Analysis**
Fifty-two quarries supplied material for the 132 projects. Twenty-four quarries provided aggregate to the 42 criteria projects that exhibited D-cracking before 20 years’ service. Considering the quarries with the highest numbers of D-cracking projects, 6 of the 24 quarries (25%) supplied material to 23 of the 42 (55%) failing projects. Nine quarries provided aggregate to 29 projects that failed. Therefore 55% of the failures came from six quarries and 69% of the failures came from nine quarries. The overall results are shown in Fig. 2 with the projects...
containing material from these 9 quarries. These quarries account for approximately 69% of the premature failures, shown in Fig. 1, which is significantly higher than the 14% failure rate for the remaining 44 quarries with acceptable field performance.

Fig. 3 illustrates the results for these same nine quarries over time, including an average failure rate during the 1990s of 81%.

Figure 3. Survey results for survey projects containing material from the 9 quarries with unacceptable field performance, by year of construction and legislative dates for Comprehensive Highway Program (CHP) and Comprehensive Transportation Program (CTP).

D-Cracking in Less than 10 years
On at least four projects surveyed during the 2010-2012 study, crews observed D-cracking on pavements that were less than 10 years old. The ages of the observed pavements were 6, 7, 8, and 9 years old.

Limited Success of the 1980s Specification
KDOT’s 1980’s specifications improved the overall D-cracking performance. On a mileage basis, the Annual Pavement Survey results indicated that the minimum D-cracking failure rate was about 45% in 1980 and 24% in 2010 for projects under 20 years old. On a mileage basis (and not considering projects under 10 years old in 2009-2010), the new specifications decreased the failure rate from approximately 45% to 24%.

Under the 1980 specifications, 95% of the concrete pavements were expected to last 20 years before D-cracking. The 2010-2012 survey data showed that 95% of the concrete pavements in fact only lasted 11 years. With this data and supporting outside forensic agreement with the
results, KDOT decided that with a 24% failure rate and 11-year life for concrete pavement, further improvement was necessary.

Exhibited life can be defined as the minimum (shortest) life of all the “successful” (acceptable) projects for a given acceptable failure rate. In the 2010-2012 study, the observed failure rate was 31%, the youngest age that a project failed under the failure category was 9 years after construction, and the oldest age that a pavement failed was 19 years after construction. The minimum age of the remaining “successful” (acceptable) projects was 20 years (the design life for PCCP in Kansas). This 20-year age represented the exhibited life for a failure rate of 31%. Reversing the analysis, if the failure rate was chosen to be 5% (instead of specifying the design life), then out of the 132 projects included in the study, the 7 projects (5%) with the youngest (minimum) age at failure would be considered “failing” projects and the minimum age of all the remaining “successful” projects was 11 years. Therefore, the current exhibited life for a 5% failure rate (on a project basis) was 11 years.

FREEZE-THAW TESTING OF AGGREGATE IN CONCRETE – THE KTMR-22 PROCEDURE

KDOT instituted prequalification of calcareous stone sources for concrete paving aggregate by testing with KTMR-22, a concrete freeze-thaw test method based on a method developed by the state of Iowa for a similar purpose. KTMR-22 is a modification of the ASTM C666 Method B procedure with the exception that instead of a 14-day lime water cure, the specimens are stored in a 100% relative humidity room for 67 days, then a 50% relative humidity room for 21 days, followed by soaking in 70°F water for 24 hours, and finally tempered in ≤40°F water for 24 hours before taking initial readings and starting the freeze-thaw cycling.

The initial specification requirement in the 1980s was a Durability Factor (DF) of ≥95 and percent expansion (%E) of ≤0.025 at 300 cycles of freezing and thawing. Later modifications of KDOT specifications resulted in the addition of what was thought to be a higher performing class of calcareous stone. The initial specification limits became labeled “Class I” stone and a second classification or “Class II” stone was introduced with higher testing limits of DF ≥97 and percent expansion ≤0.015 at 300 cycles. Concrete paving mixes using Class II stone were allowed to have up to 20% aggregate retained on the ¾-in. sieve while Class I were limited to a maximum of 5%.

THE 2013 AGGREGATE SPECIFICATION REVISION

In response to the findings of KDOT’s 2010-2012 D-Cracking Study, KDOT changed the specification for concrete paving aggregates in January 2013. The changes are summarized below.

Increased the Number of Freeze-Thaw Cycles from 300 to 660
Since KDOT’s specifications for OGCA were already a minimum Durability Factor of 95, the option of tightening the specification requirement was unrealistic. However, the evidence suggested the current method and specifications were not adequately predicting field
performance. An alternative option was to increase the number of freeze-thaw cycles for the procedure and material specification. This option allowed minimal disruption to continuous testing using existing equipment and methodology. Weather data analyzed during the 2010-2012 D-Cracking Study showed that Kansas averages 33 hard freeze-thaw cycles per year. The new limit was based on a simple calculation of 33 average annual freeze thaw cycles multiplied by the KDOT expected design life of 20 years.

Replaced “Durability Factor” with “Relative Dynamic Modulus of Elasticity” (RDME)
Durability Factor, or DF, is a term defined in the ASTM C666 procedure. Extending the KTMR-22 testing to 660 cycles resulted in a higher frequency of samples that not only do not meet our materials specifications, but also in samples that perform so poorly that the testing must be terminated before 660 cycles to prevent damage to testing equipment due to disintegrating specimens. Following ASTM’s procedure for calculating DF on these failed specimens leads to the reporting of questionable results, and is discussed later.

Changed reference to “On Grade Concrete Aggregate” (OGCA)
Aggregate specifications for On Grade Concrete Aggregate (OGCA) was differentiated from specifications for Aggregate for Concrete Not Placed on Grade. D-cracking generally occurs in on-grade concrete, applications where the concrete has a drying gradient with one surface exposed to drying and another never fully drying. This method of distinction clarified which products in addition to mainline paving require a higher level of freeze-thaw durability, including curbs, gutters, and sidewalks.

Consolidated Specification to One Paving Class for All OGCA
By extending the testing to more than twice the original number of freeze-thaw cycles, it was not considered necessary to distinguish between multiple classes of concrete aggregates. Any aggregate meeting the new specification (RDME ≥95 and %E ≤0.025 at 660 cycles) is performing at a higher level than the original specification at 300 cycles, so all on-grade concrete mixes are allowed up to 20% retained on the ¾-in sieve.

Prequalification of OGCA Now Required for All Coarse Aggregate Types
Non-calcareous stone sources have generally been considered to be freeze-thaw resistant. However, during the 2010-2012 study, some river gravel deposits in western Kansas and Eastern Colorado failed extended KTMR-22 testing and field sites containing those materials indicated likely D-cracking. As a result, all aggregate sources currently must meet the same prequalification requirements, regardless of aggregate type.

Attempted Use of Acid Insoluble Residue Testing (KTMR-28) for Pre-Screening Sand-Gravel Sources
KDOT’s KTMR-28 acid insolubility test is based on ASTM D3042. During initial extended KTMR-22 freeze-thaw testing, at least one gravel source exhibited poor freeze-thaw durability. This source also exhibited a lower KTMR-28 results as compared to other typical gravel sources. As a result, the revised OGCA specification initially required a 95% minimum acid insolubility result for sand-gravel sources as a screening test prior to freeze-thaw testing. The goal was to expedite the prequalification process, saving on manpower, materials, and freezer space.
For research purposes, all sand-gravel samples continue to be freeze-thaw tested per KTMR-22 for the purpose of collecting data to aid future review of this new requirement. Initial results indicated that seven screening tests on sand-gravel sources contradicted the KTMR-28 requirement. Five samples failed KTMR-28 yet passed KTMR-22 and two samples failed KTMR-22 yet passed KTMR-28. To date, the only source that has failed both the KTMR-22 and KTMR-28 requirements is the first sample that inspired the initial specification limits. As a result, the KTMR-28 requirement for sand-gravel sources has since been rescinded.

Focus on Production Sampling at Concrete Production Sites
The prequalification process for OGCA requires two samples from current aggregate production, consisting of “approved beds” for calcareous sources, be tested according to KTMR-22 and meet the OGCA specification requirements. Approved beds are beds that the producer has been actively working to supply aggregate for KDOT concrete paving projects and that are currently meeting specification requirements. If the beds did not meet previous specification requirements or if the source is a new producer for KDOT, the beds are approved by Ledge Sampling and Evaluation by KDOT Geology.

Continued prequalification for all sources is based on acceptable test results of active production aggregate samples. Previously, the required sampling frequency from each source was one sample for every 20,000 tons produced or a minimum of 3 per year from any producing source, and one sample from the concrete production site at each project using 5,000 tons or more. The revised requirements for production sampling frequency of each source include one sample per year at the source, once every 5,000 tons from any ready-mix concrete plants supplying to KDOT projects, and once every 20,000 tons from any contractor batch plants supplying to KDOT projects. In addition to acceptable test results, the revised OGCA specification requires sources to demonstrate acceptable field performance when used in on-grade concrete placed on KDOT projects.

IMPACT OF OGCA SPECIFICATION CHANGES
One of the aggregate sources that had been approved for use in on-grade concrete prior to the January 2013 specification revision was identified through the 2010-2012 Study to be a source that demonstrated poor field performance. Samples of the previously approved beds from that quarry were tested in order to determine their compliance with the new OGCA specification. The results of that test are shown in Figure 4.
The results for this sample at the old specification limit of 300 cycles were RDME = 96, %E = 0.003 and the results at the new specification limit of 660 cycles were RDME = 77, %E = 0.037. Although the only true measurement of the success of the OGCA specification changes will be future field performance of on-grade concrete constructed with prequalified coarse aggregate sources, test results for this particular source have indicated that the new specification limits are at least a step in the right direction.

CHALLENGES DUE TO CHANGING TESTING AND MATERIAL SPECIFICATIONS

DF vs. RDME
The KTMR test procedure, very briefly, consists of casting sample aggregate into three prisms using a standard concrete mix, curing and tempering the specimens, collecting initial measurements, and then subjecting them to cycles of freeze-thaw. Once a week, each prism is removed and subsequent measurements are collected. These measurements are mass, length, and fundamental transverse frequency. The calculations used in the test procedure are the same as those described by ASTM C666:

Relative Dynamic Modulus of Elasticity
\[ P_c = \left( \frac{n_1^2}{n_2^2} \right) \times 100 \]
where:
- \( P_c \) = relative dynamic modulus of elasticity, after \( c \) cycles of freezing and thawing, percent,
- \( n \) = fundamental transverse frequency at 0 cycles of freezing and thawing, and
- \( n_1 \) = fundamental transverse frequency after \( c \) cycles of freezing and thawing.

Durability Factor
\[ DF = \frac{PN}{M} \]
where:
- DF = durability factor of the test specimen,
P = relative dynamic modulus of elasticity at N cycles, \%
N = number of cycles at which P reaches the specified minimum value for discontinuing the test
    or the specified number of cycles at which the exposure is to be terminated, whichever is
    less, and
M = specified number of cycles at which the exposure is to be terminated.

KDOT has defined the test parameters as follows:
N = lesser of 660 or cycles where either P ≤ 60 or %E ≥ 0.100
M = 660 cycles

One of the first obstacles to performing this calculation correctly is that ASTM C666 provides no
guidance for interpreting the collected data. The procedure requires recording measurements at
intervals not to exceed 36 cycles. Therefore, the data will be represented by points on a plot of
RDME or %E vs. number of cycles. Areas where guidance is lacking are:
• If the test is completed at X number of cycles with a result that is within the criteria for
testing termination; (i.e., not showing excessively low RDME or excessively high %E),
is the result a straight line interpolation between the points that surround M?
• Should all the points be fitted with a line or polynomial that is then used to interpolate the
result at M? The method KDOT has used since the January 2013 revision to the OGCA
specification is to fit the data points with a polynomial (typically third order or higher)
and use the equation of the polynomial to calculate the RDME or %E at 660 cycles.
• An additional obstacle is that there is no direction for correcting %E results if testing is
terminated early.

When a third order or higher polynomial is used to graph results for a sample that performs
significantly poorly during the test, it becomes apparent that the ASTM C666 calculation for DF
assumes a linear relationship between RDME and number of cycles which has significant effect
on the final result. This is illustrated in Figure 5.
Figure 7 contains data points from an aggregate sample that was tested in KDOT’s laboratory. The test data points, when connected using a third order polynomial, indicate the RDME at 660 cycles to be less than zero. While the validity of reporting a zero value versus a negative value for the result of this test could be argued, it is clear that reporting a result above zero at an M of 660 cycles would be incorrect. However, ASTM C666 would have the results of this particular test be reported as DF = 28, even though the point of 28 at 660 cycles clearly falls far from the plotted curve.

The extended freeze-thaw testing that KDOT has performed has shown similar evidence on many failing samples. The end result, regardless of method used, will not vary the prequalification of the source because this only becomes an issue when the sample has significantly failed the specification requirements and is terminated prior to 660 cycles. However, KDOT’s concerns are surrounding the issues of proper terminology in test reporting and accurate data collection for historical and analytical purposes. Therefore, KDOT’s OGCA specification references RDME instead of DF.

**Relationship Between RDME and %E**

So far, KDOT has performed over two hundred and thirty KTMR-22 procedures at the new specification limit of 660 cycles of freeze-thaw. Figure 6 shows each test that has completed at the time of this publication plotting the average %E vs. the average RDME for each set of three specimens. The graph suggests there is sufficient correlation between increasing percent expansion and decreasing RDME and less than 8 percent of our samples have fallen in the quadrants of the graph where the sample is meeting one part of the specification requirements yet failing the other with respect to percent expansion and RDME (quadrants II and IV).
Even with a small percentage of the testing population falling into quadrants II and IV, there is still concern regarding how KDOT will react to such test results, especially when they are seen from sources that have some established history of meeting the KTMR-22 requirements. One recent production sample from a dolomite source that is prequalified to supply OGCA in Kansas has demonstrated substantially high RDME values, yet did not meet the specification requirement for percent expansion. The results from this sample are shown in Figure 7.

![RDME and %E Plots for Recent Dolomite Sample](image)

**Figure 7 – RDME and %E Plots for Recent Dolomite Sample**

One of the three prisms from this sample was removed and examined after measurements were collected at 661 cycles. The examination was performed by Randy Billinger, P.G., KDOT Research Geologist. Mr. Billinger’s findings are summarized below:

*I have looked at prism B and cannot definitively determine why the prism is failing expansion. The prism looks good. There is no external damage except for one pop out. We have cut the prism in many places and have looked at 12 internal surfaces (7 polished and 5 unpolished). I do not see any micro or macro cracking in the paste. I do not see any paste issues. The paste-aggregate bond is moderately tight to tight.*

*The air void system is good. Total air is 7.3%, spacing factor is 0.11mm and specific surface is 31.71 mm²/mm³ (this number should be between 25 to 45 mm²/mm³).*

*The aggregate does not appear to be cracking. There are some pieces that have what appear to be cracks, but I think most of these are part of the aggregate itself and were in the aggregate when it went in the mix. To investigate this, I took pieces of aggregate from this source that I have in the lab and polished about 40 pieces. Some of the aggregates show the same type of cracking as seen in the aggregate in the prism. The aggregate is very crystalline and I think the cracking seen is mostly within the rock as it sits in the outcrop. However, there may be some cracking being caused by freeze-thaw, I just cannot prove and state that at this time. Of the aggregates that show some type of crack, I don’t see any cracks exiting the aggregate and cracking into the paste.*
As stated, the aggregate is crystalline and many of the pieces have numerous voids/mineral lined vugs in them. Vugs are cavities (in this case small cavities with visible crystals in them). The porous nature of many of the aggregates may have something to do with the unusual expansion numbers. However, I just don’t see positive evidence of freeze-thaw damage in the aggregate.

I am not seeing any ASR or ACR at this time.

The remaining two specimens are still being tested at the time of this publication and have undergone well over 1100 cycles of freeze-thaw. They still show little to no surface evidence of deterioration and have relatively high RDME with relatively high percent expansion. These prisms will be removed once they begin to show visible deterioration on their surfaces or at a time that the space they are occupying in the freezer is critically needed for ongoing production testing. At that time they will also be examined by Mr. Billinger and hopefully provide better evidence of the cause behind their unusual behavior.

Until the testing on this sample is terminated and, more likely, until KDOT sees similar results from additional samples in the future, it will be difficult to predict the changes that will be made to the OGCA specification. This sample may end up being an outlier that can never be fully explained. However, in the meantime, samples that exhibit non-typical test results may need to be examined for physical evidence of freeze-thaw related distress prior to publishing test results that could adversely affect a source’s prequalification. Also, if additional evidence suggests that percent expansion is less accurate at predicting true freeze-thaw durability of the aggregate, the specification limit for percent expansion may either need to be revised or removed.

**Unusual RDME Results**

One particular source has on multiple occasions exhibited unusual behavior. Two samples from this source have both showed similar results where the early measurements, taken after fewer than 50 freeze-thaw cycles, resulted in RDME results at or just below the specification limit; however, after more than 1500 cycles of freeze-thaw, the overall decrease in RDME after the initial freeze-thaw exposure was 4 or less. Results of the samples are shown in Fig. 8 and 9.
Again, more than two hundred and thirty samples have been freeze-thaw tested to greater than 660 cycles. Of those, only four samples have shown an RDME result that was less than 4 points from the RDME calculated after the first round of freeze-thaw cycles. Those results are summarized in Table 2.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Initial Exp</th>
<th>Initial RDME</th>
<th>Initial Cycles</th>
<th>Result Exp</th>
<th>Result RDME</th>
<th>Difference Initial - 660</th>
<th>Final RDME</th>
<th>Final Cycles</th>
<th>Difference Initial – Final</th>
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<tr>
<td>2775</td>
<td>0.000</td>
<td>96</td>
<td>128</td>
<td>0.015</td>
<td>93</td>
<td>3</td>
<td>88</td>
<td>1048</td>
<td>8</td>
</tr>
<tr>
<td>2978</td>
<td>0.009</td>
<td>95</td>
<td>16</td>
<td>0.021</td>
<td>94</td>
<td>1</td>
<td>93</td>
<td>824</td>
<td>2</td>
</tr>
<tr>
<td>3037</td>
<td>0.011</td>
<td>95</td>
<td>40</td>
<td>0.024</td>
<td>94</td>
<td>1</td>
<td>93</td>
<td>1503</td>
<td>4</td>
</tr>
<tr>
<td>3039</td>
<td>0.013</td>
<td>94</td>
<td>48</td>
<td>0.016</td>
<td>95</td>
<td>-1</td>
<td>93</td>
<td>1511</td>
<td>1</td>
</tr>
</tbody>
</table>

It should be noted that the %E for all four samples in Table 2 met the OGCA specification requirement. The samples in Table 2 represent less than 2% of the population of test results generated to date. It is intriguing that one source has supplied 50% of that small population and that both of those samples exhibited a drop in RDME of 4 or less over a span of 1463 cycles.

At the time of this publication, samples 3037 and 3038 have been the only two samples from that source that have been tested beyond 660 cycles. Both sets of specimens have been retained for analysis by KDOT's Research Geologist but, at the date of this publication, the examination has not yet been completed. Future samples from the same source will be closely monitored for a similar trend.

It will again be difficult to predict how this information will affect future testing or specification limits. Analysis of the 3037 and 3039 specimens will play a critical role in the future for determining the prequalification status of this source. If the specimens show no signs of aggregate related distress and future samples from this source exhibit similar performance, it is
possible that KDOT may have to consider additional specification criteria such as waiving the 660 cycle requirements if additional, slightly more relaxed, criteria are met at a significantly higher number of freeze-thaw cycles.

**Length of Test Procedure**

The 90 day curing period combined with extending the freeze-thaw cycles to 660 has resulted in a test procedure that takes about six months to perform, once the concrete prisms are molded. Any backlog of samples or other delays in schedule only exacerbate this issue. Many concrete paving projects are completed in six months, which will limit the recourse KDOT will have if a sample collected during production of concrete on a project fails.

There is ongoing research being conducted as a joint effort between KDOT and Kansas State University to find ways to accelerate the schedule of the KTMR-22 test without negating all of KDOT’s historic test results. However, this research is still relatively young and it will be some time before KDOT is in a position to adopt any significant changes to the KTMR-22 procedure.

**Conflicting Test Results**

Since the revised OGCA specification places more emphasis on sampling production aggregate at the concrete plant site and requires different sampling frequencies for the aggregate source, contractor plants, and ready-mix plants, it is likely that at some point in the future a source will have multiple samples being tested at or around the same time. Should a series of KTMR-22 test results demonstrate variable results where some pass and some fail the specification requirements, it will present a challenge for KDOT to determine a best course of action in terms of maintaining or revoking that source’s prequalification status.

**Conclusions**

KDOT has a history of D-cracking pavements and significant efforts, including five extensive studies into the phenomenon of D-Cracking, have been made to mitigate the problem. Past changes in quarry production observation and QA/QC programs appear to have had some effect on the quality of pavements produced; however KDOT recognized that with the desire for longer lasting pavements, modifications to past testing of aggregate freeze-thaw durability are required to assess aggregate sources and achieve a longer exhibited pavement lives. As a result of the 2010-2012 study, KDOT has implemented significant changes in aggregate source testing requirements to further mitigate the risk of D-cracking. The current specifications require extended freeze-thaw testing with the specified number of testing cycles far exceeding other known DOT requirements. D-cracking is still a problem in Kansas today; however, modified testing and aggregate approval based on acceptable long-term freeze-thaw behavior is a step in the right direction toward extending anticipated pavement life.

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Falling Rocks – Harnessing the Schuylkill Expressway

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ABSTRACT

As Tropical Storm Lee passed over Southeastern Pennsylvania on September 8, 2011, heavy rains triggered a significant rockslide along the Schuylkill Expressway (Interstate 76) merely 6 miles west of Philadelphia, Pennsylvania, temporarily closing the eastbound traffic lanes into the city.

As part of an on-going effort to remediate this hazard prone area of Philadelphia’s arterial system, PennDOT geotechnical engineers and consultants teamed up to evaluate the nature of the failures occurring along the exposed rock cut and developed a mitigation program which included rock scaling, selective tree removal, rock bolting, reinforced steel wire mesh, shotcrete shoring, and slope drain installation.

As southeastern Pennsylvania is not well known for significant rock cuts, the dangers associated with rockfalls are not a common natural hazard threatening the region. As such, the engineers, geologists, and contractors were tasked with the challenges of designing and constructing the region’s first mechanically stabilized rock slope on a stage that averages 130,000 vehicles a day. Several logistical challenges were encountered during construction, including maintenance and protection of traffic, a very limited work zone, time constraints, and “falling rocks”.

The total construction cost for the project was $2.8 million with a duration of four months from notice to proceed through completion of construction. This project illustrated PennDOT’s ability to quickly respond to a slope failure by teaming with consultants and contractors to provide a safe and efficient design and expedited construction schedule.
INTRODUCTION

Since the last continental glaciers began to retreat approximately 18,000 years ago, the Schuylkill River has carved a valley across the Piedmont Plateau as it flows southeast out of the Appalachian Mountains toward the City of Philadelphia. Northwest of Philadelphia the river travels south cross-cutting the less resistant Cambrian and Ordovician age limestone and dolomite formations before it abruptly meanders east at Conshohocken where it crosses the Cream Valley Fault and the unconformable contact with highly resistant Precambrian age metamorphic gneiss. As the river followed the path of least resistance along the existing fault line, valley down cutting exposed the more durable gneiss along its southern bank.

The Philadelphia and Reading Railroad originally established in 1833 (3) was the first major mode of transport and industry to take advantage of the Schuylkill River corridor as a direct land route from prospering suburbs into the city. With the ascent of the automobile as the predominant mode of transportation, construction of the existing Schuylkill Expressway began in 1949 (2), utilizing much of the already established railroad route along the narrow rock cut passages approaching the city. Several sections of the route between Conshohocken and Philadelphia required blasting of new rock cuts or expansion of existing railroad cuts through prominent gneiss and schist bluffs along the south and west banks of the river. Today the Schuylkill Expressway carries approximately 128,000 to 174,000 vehicles per day between Philadelphia and Montgomery Counties (1), serving as the primary corridor into Philadelphia from the west and is the busiest road in the commonwealth of Pennsylvania.

Between August 27 and September 9, 2011, the Philadelphia area received more than 12.5 inches of precipitation (4), as Hurricane Irene (August 27-29) and Tropical Storm Lee (September 5-9) punished the mid-Atlantic region. As Tropical Storm Lee continued to dump rain over an already oversaturated region, runoff triggered a significant rockslide along the eastbound Schuylkill Expressway (Interstate 76) in Lower Merion Township, Montgomery County, temporarily closing traffic lanes into the city. The slide occurred along a commonly congested half-mile stretch of road between PA State Route 23 and River Road. The rock cut adjacent to this section of Interstate 76 is 15 to 70 feet high, near vertical and between 2 and 12 feet from the paved edge of the roadway.

The Pennsylvania Department of Transportation’s (PennDOT) Engineering District 6 recognized that this half-mile section of Interstate 76 was prone to rockslides/falls due to the nature of the local geology and erosional forces at work. Following a preliminary geologic assessment, PennDOT Engineering District 6-0, assisted by URS geotechnical consultants of Fort Washington, PA, prepared preliminary design drawings for a comprehensive rock anchor and slope stabilization treatment program and advertised a bid package in June 2013 for construction of the District’s first mechanically stabilized rock slope. At the onset of construction activities, PennDOT geotechnical engineers requested Gannett Fleming, Inc. to provide consultation services consisting of quality control of construction operations and design review through completion of construction.

There were several factors that posed challenges to this project. This was the first rock slope remediation project in District 6, as such; both the Geotechnical and Construction Units
were inexperienced with this type of work. Due to the high volume of traffic, all construction activities were required to take place while maintaining both eastbound travel lanes open to traffic. In addition, the existing shoulder was 8 to 12 feet wide, which considerably limited available space to stage equipment and perform routine operations. These obstacles necessitated significant teamwork and creativity regarding traffic control, scheduling, and planning including limited night and weekend work. These challenges were met by the entire Construction team (PennDOT, Consultant, and Contractor staff) with efficiency and flexibility that allowed the project to be completed on time even given a significant increase in scope of work which arose after mobilization and commencement of construction activities.

SITE GEOLOGY

This section of the Schuylkill River and adjacent Expressway parallels a high-angle fault (Cream Valley Fault) trending northeast-southwest and an unconformable contact that separates carbonate bedrock formations of the Piedmont Lowland Section and metamorphic units of the Piedmont Upland Section of the Piedmont physiographic province in southeast Pennsylvania (5,6). The topography is characterized by flat upper terrace surfaces cut by shallow valleys of low to medium relief generally shaped by the meandering of the Schuylkill River and its tributaries over many centuries, and more recently by urban development. Specifically, valley down-cutting and stress relief along with urban expansion for transportation and industry combined to expose the 70-foot, near-vertical rock slope consisting of meta-sedimentary and meta-igneous Precambrian Baltimore gneisses (6) along the project corridor.

The oldest rocks in the Piedmont including the Baltimore gneisses were formed over a long and vigorous tectonic history. These rock units initially experienced periods of thrusting and metamorphism associated the development of a magmatic arc during the Grenville Orogeny approximately 1,100 Ma. Subsequent episodes of tectonic activity throughout the Paleozoic Era including formation of the North American continent further metamorphosed, deformed, and faulted (Cream Valley and Rosemont Faults) these assemblages. Eventually, these rocks have become exposed at the surface through differential uplift and erosion (Schuylkill River).

The Baltimore gneiss exposed along the road cut consists of interfingered units of meta-sedimentary gneiss and meta-igneous gneiss. The strike of the foliation trends northeast-southwest (nearly parallel to the Expressway) and dips approximately 75° to 80° northwest (toward the Expressway). The meta-sedimentary rock units display stress relief displacement sheet-jointing that follows the foliation planes and the meta-igneous units display a blocky non-uniform joint pattern. The stress relief displacement jointing is the result of elastic rebound, relaxation, and dilation of the rock mass due to the changes in stress induced by valley down cutting and slope excavation as the mass moves toward a new state of equilibrium. As relaxation progresses over time, inner rock layers will experience similar elastic movements and a strain gradient may be developed resulting in strength loss to the rock mass and continued instability in the form of rockfalls (8).
The jointing conditions along with differential weathering of the rock assemblages have fostered slope instabilities leading to translational slides and toppling failures. In addition, it is evident that environmental processes such as freeze-thaw and root wedging along with anthropogenic involvement including the road cut and diversion of drainage patterns may have initiated these failures.

EXISTING CONDITIONS

Through the project area, the Schuylkill Expressway consists of two 12-foot-wide travel lanes in each direction with a 5 foot high center concrete median barrier separating east and westbound traffic. The work zone was bounded by the limits of the eastbound outside shoulder which varies from just 8 feet wide at the west end of the project (near the SR 23 onramp) to 12 feet at the east end of the project. The shoulder is bordered by a 32 inch high single-face concrete barrier separating an elevated catchment area. The catchment area consists of structural fill soils piled behind the adjacent barrier and varies from approximately 2 to 12 feet from the paved edge of the roadway to the rock face. A PennDOT traffic camera pole and associated facilities are positioned within the catchment area near the center of the project area.

The rock slope varies from approximately 15 feet high on the east and west ends of the project area to a maximum height of approximately 70 feet near the center. The brow of the rock slope transitions to an expansive soil covered and forested upslope region rising to the south at an inclination of about 3H:1V. The majority of the existing rock slope had been overgrown by...
thick vegetation consisting of small to large shrubs, briars, vines, and dispersed trees. The brow/crest of the rock slope and upslope region are completely forested sustaining larger and denser vegetation including sizable trees varying in caliper from 2 inches to more than 24 inches, maintaining sizable root systems capable of prying into the fractured rock.

Figure 2 – Existing Conditions Cross-Section (Eastbound Schuylkill Expressway)

In addition, 24-inch to 48-inch diameter corrugated metal pipe (CMP) slope drains extend the full height of the slope at four locations within the work area. The slope drains are connected to concrete lined swales that capture stormwater runoff and near surface groundwater from the upslope area and drain to a storm water drain system beneath the roadway. The western most slope drain appears to maintain continuous flow throughout the year.

SITE RECONNAISSANCE AND PRELIMINARY DESIGN

PennDOT and URS geotechnical engineers performed an initial reconnaissance to assess the existing geologic conditions of the site. Due to the high volume of traffic, narrow shoulder space, and vegetation covering the rock cut, the initial geologic assessment was limited. The
scope of work was developed based on window mapping of the exposed rock face, which could only be accessed by walking the top and bottom of the slope, and areas that could be reached by a small (40 ft) boom-lift. The primary failure mechanism, characterized by stress displacement jointing along the foliation plane was readily identified. Large sections of detached bedrock in the form of elongated sheets and overhanging boulders were observed. During a subsequent site reconnaissance, PennDOT geotechnical engineers noted a broken headwall and concrete inlet apron maintaining one of the CMP slope drains allowing collected stormwater into the fracture network behind the rock face. The malfunctioning CMP slope drain was observed to be within the area of the initial rockfall and was likely a significant contributor to triggering the event.

Based on the findings of the geologic assessment, a comprehensive rock anchor and slope stabilization treatment program was recommended to remediate any loose material that was immediately threatening the roadway and mechanically stabilize the rock slope from future failures. The comprehensive treatment program was estimated to cost $2.3 million and included:

- Rock face scaling including clearing of vegetation (1,240 hrs);
- Rock anchor bolt installation, consisting of 10-ft and 15-ft, 1-inch diameter 150 KSI galvanized post-tensioned anchor bars (6,770 lf);
- Installation of anchored mesh (35 SY);
- Installation of shotcrete buttresses (24 CY); and
- Installation of horizontal drains (400 lf)

(1) Numbers in parentheses denote the estimated item quantities included in the bid package.

The entire 2,000 linear feet of the exposed rock slope was selected to receive treatment. The bid package consisted of a photomosaic plan detailing the conceptual layout of pattern and selective anchor bolt locations, specific locations to receive anchored mesh and shotcrete, design drawings, details and specifications for the rock anchors, anchored mesh and shotcrete buttresses, specifications for miscellaneous road construction, and required traffic control plans.

Because the majority of the rock slope was obstructed by heavy vegetation or inaccessible during the geologic assessment, the final design and layout of the stabilization items was to be completed concurrently with the rock face scaling/clearing operations and revised to adapt to existing conditions.

CONSTRUCTION

The bid package was advertised through PennDOT’s ECMS system for a period of six weeks. The successful bidder was general contractor Buckley and Company, Inc. (Buckley), of Philadelphia, Pennsylvania. Buckley teamed with specialty subcontractors Ameritech Slope Constructors (Ameritech), of Ashville, North Carolina, to perform the rock slope scaling work, and Terra Structures, a division of The H&K Group (Terra), of Skippack, Pennsylvania to perform the drilling and installation of the rock anchors and stabilization measures. Construction lasted three months, beginning on September 9, 2013 with completion on December 20, 2013. It was at this point, the onset of construction activities, that PennDOT requested Gannett Fleming,
Inc. to provide consulting services consisting of design review and quality control of all field operations through the completion of construction.

An experienced geologist from Gannett Fleming familiar with the site conditions and objectives of the stabilization program, provided full-time oversight and quality control efforts of all construction activities to support PennDOT. In addition, PennDOT provided a full time geotechnical engineer for guidance and full-time construction inspectors to measure and approve payment items and quantities.

**Maintenance and Protection of Traffic**

Maintaining the two eastbound lanes of traffic through the work zone during the execution of the rock slope work was particularly challenging for the construction team. As part of the project, a contractor-designed traffic control plan was developed to maintain traffic through the work zone and protect the traveling public as well as construction personnel within the work zone. To expedite the construction schedule and provide additional working space when necessary, night time single-lane closures and 15-minute (maximum) PA State police escorted traffic stoppages were permitted between the hours of 10 PM and 5 AM.

In addition to the traffic control pattern layout, signage, barriers, and standard devices, the construction team was tasked with developing a rockfall catchment system that would protect the roadway, buried utilities, and the travelling public from falling rocks and bouncing debris during the rock slope scaling operations. Due to the narrow limits and progressions of the active work zone, the anticipated size and kinetic energy of the tumbling rocks, Buckley designed and fabricated two mobile rock-shield trailers fabricated from single drop crushed car haulers. The mobile rock-shield trailers incorporated collapsible steel plates along the right side (rock slope side) that could be lowered to the catchment/ground surface to protect buried utilities, the concrete barrier wall, and paved shoulder from falling rock as well as allow easy loading of the accumulated rock debris. In addition, the trailer included a nylon woven mesh netting system that could be raised via hand crank and pulley to a height of 5-feet above the top of the trailer to capture any smaller debris that might escape the reach of the trailer.

**FIGURE 3 – Rock-Shield Trailer**
Rock Slope Scaling

Due to the spatial limitations of the work zone Ameritech utilized rapelling techniques to access the slope face. Each crew consisted of two scalers working the rock face and one foreman on the ground supervising safety and equipment. Typical scaling arrangements referred to as “hangs” consisted of two crews working side by side from the top of the rock face down to the base. In general, between two and five hangs could be completed in a typical 8-hour work shift. The crews used scaling bars to remove the majority of the loose rock, and 22 to 70 ton high-pressure air cushions (MatJacks) to dislodge and remove larger disconnected rock masses and boulders. Ameritech only used the MatJacks where larger rock masses were already detached from the intact rock face and it was considered potentially dangerous to disturb blocks by drilling. In general, the physical size of the rock masses scaled from the slope was considerably larger than anticipated during the preliminary design.
Several localized areas of significantly large rock masses or areas considerably higher on the slope were selected to be scaled during overnight hours to allow a single lane closure with police escorted 15-minute traffic stops as added protection against falling debris. Near the completion of scaling activities, an extremely large unanticipated detached rock mass was exposed after loose material was removed from the slope face. This mass was considered directly hazardous to the work zone and traveling public. The large mass was separated from the intact face by approximately 6-inches and could be rocked under the leverage of a scaling bar. Ameritech scaled the rock mass using two 70-ton MatJacks during an emergency traffic stoppage. The scaled rock mass was captured by the rock-shield trailer; however, the undercarriage of the trailer was crushed by the impact force of the falling rock. All of the falling debris was retained by the rock-shield trailer and traffic was immediately released.

Following the removal of the wrecked trailer, the remaining scaling operations were performed at night using a combination of wooden crane mats and a 12-inch thick stone (recycled concrete) pad as protection for the single face concrete barrier and paved shoulder. In addition, the remaining rock-shield trailer was moved into the closed travel lane to protect traffic.

Based on the amount of rock scaled from the slope and the extent of mechanical weathering observed once the vegetation was removed from the rock face, PennDOT and Gannett Fleming engineers and geologist revised the original design to incorporate a combination of fully grouted dowels and post-tensioned anchors, larger treatment areas to receive anchored mesh, additional locations to receive shotcrete shoring, and removal of selected trees along the brow of the slope. The final layout of the revised treatment scheme was presented in a new photomosaic plan package displaying the recently scaled rock slope.
The increased quantities, most notably the anchored mesh (from 35 SY to 1,000 SY) and shotcrete (24 CY to 50 CY) added approximately $500,000 to the total project cost. Although an increase in scope, cost, and schedule is not desirable during construction, the decision to implement the final recommendations and protect the traveling public was made easier as the Schuylkill Expressway is the busiest road in Pennsylvania (1).

**Rock Anchors**

The final rock anchor scheme consisted of installing 368, 10-foot fully grouted rock dowels across areas designated for pattern bolting, anchored mesh, and shotcrete shoring, and 59, 15 to 20-foot post-tensioned anchors, totaling more than 4,600 linear feet of anchor length. All anchors/dowels installed consisted of 1-inch diameter, galvanized, continuous thread, 150 ksi steel bars.

The intent of the rock dowels was to prevent ongoing slope relaxation while providing overall reinforcement to the existing rock face by increasing shear resistance along planes of weakness and interlocking fractured rock masses. Rock dowels were generally spaced over a 10 foot x 10 foot staggered grid pattern in the areas designated for pattern bolting and an 8 foot x 8 foot staggered pattern in the areas to receive anchored mesh and shotcrete. Post-tensioned anchors were installed through selected key blocks and larger rock masses to prevent specific block movement and provide additional stability in areas of concern.
Rock anchor drilling was performed using a remote controlled TEI HEM550 excavator mounted rock drill with combined air-water mist regulator and 3-inch button rock bit. In general, anchor locations less than 25 feet above the ground surface were accessed with a Caterpillar 321D LCR (compact radius) excavator that could remain within the limits of the work zone. Anchor locations at higher elevations were accessed with a Caterpillar 345C with a 65-foot triple-joint demolition boom and drilled during night-time hours while a lane closure was in effect. Each drill hole was battered at a downward angle of 15° from horizontal.

Terra provided between one and four full-time crews during the course of the slope stabilization work to accommodate the construction schedule. As Terra’s crew grew comfortable with the rock conditions and equipment arrangement, they averaged approximately 250 linear feet of drilling per day.

Terra selected a non-shrink neat cement flowable grout mix consisting of bagged Portland Type I cement and water with a 28-day design strength of 5,000 psi. The grout was mixed on-site with a mobile, trailer-mounted batch plant and pumped to the back of the drill hole through a full-length plastic grout tube. The contractor molded a set of six 2-inch test cubes each day grouting operations were performed to verify the grout compressive strength. Each specimen tested met or exceeded the required average 7-day design strength of 2,800 psi. Terra revisited each anchor location a minimum of 3 days after grouting to hand pack the remaining annulus space between the open hole and anchor bar with Portland cement and form a bearing plate leveling pad if necessary.

Per the designed criteria, the cement grouted post-tensioned rock anchors were installed maintaining a 10-foot bond length for 15-foot anchors and 15-foot bond length for 20-foot...
anchors to achieve the 40 kip design load. Completed anchor assemblies consisted of an 8-inch x 8-inch galvanized steel bearing plate coupled with a hexagonal nut with a hardened washer, and beveled washers as necessary to apply uniform load transfer from the anchor bar to the bearing plate. The initial two post-tensioned anchors fully installed were proof tested to 133% of the design load to confirm the contractor’s installation methods and materials followed by 5% of the remaining production post-tensioned anchors to verify the anchorage capacity. The tensioned anchors were locked off at 10 kips to allow for additional loading as the rock relaxes. All proof testing and lock-off procedures were performed in the presence of a field engineer/geologist. Each anchor tested was considered acceptable based on movement and creep rate criteria defined in the specifications.

**Shotcrete**

In order to stabilize the slope and allow anchors to be installed safely, shotcrete was applied over specific zones of significantly weak and/or deteriorating rock noted during scaling. In addition, shotcrete buttresses were constructed in areas beneath sizeable rock overhangs to provide additional support and prevent specific blocks from falling.

A total of 50 cubic yards of shotcrete was applied over six unique areas along the rock face. The selected shotcrete mix provided from a local ready-mix concrete plant was designed to achieve a minimum compressive strength of 4,000 psi in seven days and 5,000 psi in 28 days. Terra provided core samples of the shotcrete taken from a 30-inch x 30-inch x 4-inch test panel shot during the initial shotcrete application to verify the grout compressive strength. Each tested core met or exceeded the required 7-day design strength of 3,000 psi.

Prior to applying any shotcrete, Terra fitted 6-inch wide, vertical geocomposite strip drains spaced at three foot intervals flush with the rock surface. Strip drains were constructed to daylight below the treatment area. Each shotcrete lift was supported by 4-inch x 4-inch welded wire fabric fastened to #5 and #8 rebar dowels embedded into intact rock.

![FIGURE 8 – Shotcrete Application](image-url)
Following the minimum 7-day curing period and verification testing, rock dowels and post-tensioned anchors were drilled and installed through the shotcrete surface, bearing on the finished face.

FIGURE 9 – Night Drilling Arrangement (through finished shotcrete face)

Anchored Mesh

Localized areas that experienced a large amount of scaling and/or lacked areas of intact rock were selected for secured drapery treatment (anchored wire mesh). The objective of the anchored mesh was to provide additional resistance against initiation of potential rockfalls and retain debris within the mesh system.

Terra selected Maccaferri’s Double Twist Woven Wire Mesh System consisting of steel geocomposite double twist hexagonal wire rock mesh and associated hardware. More than 1,000 square yards of anchored mesh were installed over seven treatment areas along the slope. The mesh was secured to the fully grouted rock dowels typically spaced over an 8 foot x 8 foot staggered pattern. In addition, secondary pins consisting of ¾-inch galvanized continuous thread bars coupled to a clawed steel plate were installed in specific areas that required additional reinforcement to firmly secure the mesh to the terrain. The top mesh boundary was reinforced with wire rope, fastened to outer eye bolts securely embedded in rock and held in position by the uppermost dowels and secondary pins. During the final weeks of construction, Terra crews worked day and night shifts to complete the anchored mesh installation and bring the project to completion on schedule.
Site Drainage

To improve near surface groundwater flow and promote slope drainage a total of twelve 20-foot perforated horizontal drains were installed at an upward inclination of 5° in areas where groundwater and excess stormwater were consistently noted exiting the rock slope. The horizontal drains were positioned in the field to intersect projected water bearing fractures. Each horizontal drain maintained some degree of flowing water following installation.

In addition to installing horizontal drains to facilitate drainage of water flow within the rock slope, Buckley repaired the broken concrete apron above the malfunctioning CMP slope drain noted during the initial site reconnaissance, allowing the upslope drainage system to function properly.

Due to construction impacts on the traveling public and potential hazards associated with the onset of winter, the project was under a strict construction schedule. The contractor and subcontractors faced daily challenges associated with working in a high volume traffic area with a limited work zone, multiple pieces of equipment, and a compressed time schedule. Inclement weather interspersed throughout the timeline, including significantly low temperatures during the latter months of the project, resulted in less-than-ideal working conditions that hampered drilling and grouting efforts. Effectively managing these conditions was a major contributor to reaching the timely conclusion of the project.

CONCLUSIONS

The construction team encountered several design and construction challenges associated with the ever-changing field conditions throughout the course of the rock slope stabilization project. Meeting these challenges was facilitated through constant communication between PennDOT, Gannett Fleming, and the Contractors, along with detailed attention to the geologic conditions as construction progressed. Several factors contributed to the successful implementation of the stabilization elements:

1. Understanding the geologic and environmental dynamics influencing the rock slope movements.
2. Revision of the original design scheme to accommodate unforeseen slope conditions.
3. Flexibility of the contractors to adapt construction methods to the irregular site conditions.
4. Experienced geotechnical engineers and geologists familiar with the site conditions and objectives of the stabilization program on site full-time providing direct oversight and quality control efforts.

To date, several months after completion of the stabilization work, the site appears to be performing well with no reports of rock debris on the road surface or obvious visual indicators of active movement, even after the region experienced record snowfalls and low temperatures throughout the winter months of 2014.
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Mitigating Landslides in Wyoming Using Multiple Innovative Designs and Technologies

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The Wyoming Department of Transportation (WYDOT) recently completed two successful large-scale landslide mitigation projects. Yeh and Associates (YA) was the consultant selected to provide the professional design services and design support.

The first landslide was located in the Snake River Canyon south of Jackson, Wyoming along Highway 89. The landslide area affected the end of a 20 foot high MSE wall which transitioned into an embankment fill that extended for more than 750 feet in length along the roadway. The failing wall and embankment were located approximately 500 feet above the Snake River. During the spring of 2011, the slide area became extremely active necessitating an emergency response. Two mitigation options were selected which included supporting the MSE wall on 5-inch diameter micropiles and using lightweight expanded polystyrene fill (EPS) in the failing embankment area. Construction was completed in the Fall 2011.

The second slide known as the Narrows Landslide, located along State Highway 220 south of Casper, Wyoming, had been active since 1965. The slide extended for more than 500 feet in length along the roadway above the North Platte River. WYDOT and YA collaborated on an innovative drilled shaft mitigation design that became known as the Coupled Shear Pile (CSP) system. This system uses the frame action of two drilled shafts that are connected at the top by a rigid pile cap. The coupled system transfers the maximum moments generated by the moving slide mass through the rigid pile cap rather than having the maximum moment at the slide plane as occurs with a single row of drilled shafts. Using this coupled system, the size of the drilled shafts was reduced to 48 inches in diameter which is significantly smaller than single row drilled shaft stabilization systems assuming both moment and shear are considered in the design. Additionally the project specified an oscillating drilled shaft construction method that was successfully completed ahead of schedule in the summer of 2013.
INTRODUCTION

The following is a description of two large-scale landslides mitigation projects that the Wyoming Department of Transportation (WYDOT) recently completed construction with design and construction support provided by Yeh and Associates, Inc (YA).

The first landslide known as the Blue Trails Landslide (not to be confused with an earlier nearby Blue Trails Project constructed in 1990’s) is located in the Snake River Canyon south of Jackson, Wyoming along Highway 89. The landslide area affected the end of a 20 foot high MSE wall which transitioned into an embankment fill that extended for more than 750 feet in length along the roadway above the Snake River.

The second landslide known as the Narrows Landslide, located along State Highway 220 south of Casper, Wyoming, had been active since 1965. The slide extended for more than 500 feet in length along the roadway above the North Platte River. WYDOT and YA collaborated on an innovative drilled shaft mitigation design known as the Coupled Shear Pile (CSP) system. Both landslide locations are depicted in Figure 1.
BLUE TRAILS LANDSLIDE

Background

Between 2006 and 2011 WYDOT observed progressive movement of a roadway section in the Snake River Canyon along Highway 89 which consisted of embankment failure and pavement distress and an actively failing Mechanically Stabilized Earth wall (MSE) for approximately 750 feet along the fill side of the roadway. Figure 2 depicts failing section looking upstream along the Snake River. Figures 3 and 4 depict movement of MSE and embankment in 2010.

Figure 2. Photograph depicts project site in 2010. Red line depicts approximate headscarp along roadway with red arrow depicting general movement direction.
Figure 3. Roadway cracking during summer 2010.

Figure 4. MSE wall distress in October 2010.
Mitigation Design

WYDOT was provided with multiple mitigation options for the failing embankment section which included replacing a section of the existing highway embankment with lightweight expanded polystyrene fill (EPS), soldier pile/tieback wall with lagging, and a ground anchor wall. After review and discussions of the mitigation options and alternatives with WYDOT Geology, the design team proceeded forward with the lightweight EPS fill option in the embankment section. Figure 5 depicts a conceptual view of the EPS system.

Based on field observations, most of the MSE wall was performing satisfactory with the exception of approximately a 150 foot section which moved vertically downward approximately 3-feet and translated horizontally approximately 1-foot during the spring and summer of 2011. The proposed repair option consisted of removing the failed section of MSE wall and rebuilding the failed section on a deep foundation system. The depth to bedrock varied under the MSE wall; however, generally it was relatively shallow and dropped off steeply near the face of the retaining wall. Because of the nature of the bedrock, micropiles were selected to support an MSE wall. As the design progressed, it became necessary to design the micropile footing with ground anchor tiebacks consisting of a steel bar with the same size and strength of the bar in the composite micropile system. The ground anchor was necessary for seismic stability and lateral earth pressures. Figure 6 depicts the generalized concept of supporting the MSE on micropile foundations.
Figure 6. Depiction of Micropile Design to Support Failing MSE Wall.

Construction

Record snows during the winter of 2010 and 2011 contributed to increased embankment and roadway embankment failure. Figure 7 depicts the failed roadway embankment section in June 2011 along the transition between the MSE wall and embankment fill. Figure 8 depicts the excavation of the MSE wall and embankment prior to placement of the micropile/tieback foundation and EPS fill. Figures 9, 10, and 11 depict construction of the tiebacks, EPS and final condition respectively.

Figure 7. Failed roadway section in June 2011. Top of MSE is at extreme left.
Figure 8. Excavated section prior to micropile and EPS construction.

Figure 9. Drilling tiebacks for micropile system.
Summary

The Blue Trails project was awarded in the Summer of 2011 and was completed by November 2011. The total length of the project was 390 feet at a bid award cost of $2.3 Million or approximately $5,900/LF lumping both micropile foundations and EPS within the total project.

Overall, the wall and embankment systems are performing as they were designed. It was imperative to have the geotechnical/geo-structural designer involved in both the design and the construction process since a multitude of construction issues arose during the construction. Communication between the Owner, Contractor, Subcontractors and Designer of Record were imperative for the successful completion of this project.
NARROWS LANDSLIDE

Background

The Narrows landslide located on US 220, approximately 15 miles south of Casper, Wyoming has been a maintenance and safety issue since the mid 1960’s. This section of roadway has experienced vertical drops in the pavement of up to one foot per year between 1966 and 2012. Due to the continual patching, the total asphalt depth was in excess of 18 feet thick along many sections of the 500 foot long roadway section. The headscarp of the landslide is located approximately 180 feet east of the highway centerline, and the toe is in the North Platte River, 200 feet west of the highway centerline. Shear movement, as recorded in slope instrumentation was occurring 40-45 feet below the highway centerline elevation. Thirty-eight test holes were drilled within the limits of the landslide during various geotechnical investigations between 1966 and 2007. Four slope inclinometers installed in 1976 sheared in slope movement by 1977. Groundwater monitoring wells installed in 2002 and 2007 indicated dry conditions or very low water table despite continued landslide movement. Figure 12 depicts aerial view of landslide, Figure 13 depicts landslide looking upstream along North Platte River, and Figure 14 depicts generalized cross section of landslide prior to mitigation.
Figure 13. Project location looking upstream along the North Platte River.

Figure 14. Figure depicts generalized cross section looking downstream along North Platte River. Red arc depicts lower landslide and brown arc depicts upper slide surface.
Mitigation Design

The designs presented to WYDOT consisted of evaluating and submitting nine separate landslide mitigation options for review to either ultimately reject or accept. One of the main criteria was a planned future build out of the roadway from two lanes to four lanes within the landslide section. The mitigation option would need to be designed to accommodate embankment fills and retaining walls. Mitigation options evaluated, analyzed, and submitted consisted of such designs as: retaining wall systems, lightweight fill replacement systems using expanded polystyrene similar to Blue Trails, ground improvement systems such as jet grouting, conventional single drilled shaft systems, ground anchor tieback systems, micropile systems, and finally a unique concept using two parallel drilled shafts connected through a pile cap which became known as the coupled shear pile system (CSP).

Supporting documentation for the CSP system was based on an article titled “Slope Stabilizing Piles and Pile-Groups: Parametric Study and Design Insights”. The publication describes many aspects of comparing single pile systems with groups of piles for stabilizing landslides using various analytical techniques and parametrically evaluating concepts with 3D finite-element (FE) modeling. The publication suggested that single piles may be inadequate for stabilizing deep landslides since a single row of elements may not provide sufficient bending resistance (Figure 15a). The publication proposed using two parallel rows of shafts or piles that are connected through a rigid pile cap to stabilize the landslide. The rigid pile cap utilizes the frame action of the system to reduce the overall moments and shear forces on the overall system (Figure 15b).

Figure 15a. Generalized depiction of moment and shear diagrams for a single shaft or pile.
Ultimately the coupled shear pile system was selected based on many variables which included: roadway construction detour requirements, protected waterway systems, planned future build out of the roadway, and addition of future retaining walls to be constructed on the landslide mitigation system.

**Single Shaft vs Coupled Shafts**

Analysis comparing single row shaft elements and coupled row (rigid frame) were conducted by the design team. Analysis methods used consisted of using the following modeling programs:

- two dimension limit equilibrium,
- lateral and group pile,
- finite element,
- structural

What became evident from the initial modeling effort was that all the software modeling programs needed to be compared against each other, as only a single modeling effort might lead to great uncertainty in the final mitigation design. Specifically, if only limit equilibrium models were used it is possible to model a single horizontal shear resisting force at the shear plane to increase the factor of safety as shown in Figure 16. This type of analysis only addresses shear of the shaft at the failure surface but does not fully evaluate deformation or deflection of the single pile element at the top.

For the couple shaft system, the modeling scenario was to apply a horizontal pressure along the length of the shaft system to provide a full height rigidity resistance in the limit equilibrium software as shown in Figure 17.
The limit equilibrium methods were also compared to finite element (deformation) programs. Figure 18 depicts the modeled existing conditions with relative displacement vectors to approximate the deformation and subsequent failure of the landslide. Figure 19 depicts a finite element model depicting only a horizontal shear force at the base of single element pile which is comparable to Figure 16. Displacement vectors are relatively unchanged from the non-mitigated (existing) conditions assuming the
effect of lateral resistance at the failure plane. Figure 20 illustrates finite element model depicting a horizontal distributed pressure loading condition provided by the coupled shear pile system. Notice that the deformation arrows only occur below the mitigation elements.

Figure 18. Finite element modeling of landslide prior to mitigation. Red arrows represent displacement vectors for visual effect.

Figure 19. Finite element modeling of landslide modeling single pile element. Red arrows represent displacement vectors for visual effect. Yellow arrow represents single resistance force at base plane. Vertical yellow line approximates single element that would be free to deflect at top of pile.
Further modeling was conducted to evaluate pile interactions with various loading conditions, flow around the piles, spacing requirements, and structural evaluation using separate modeling programs. After review and consideration, the coupled shear pile system was selected, implemented, and carried forward as a construction contract.

**Construction**

Due to the various constraints on the project an oscillating casing type specialty drilled shaft system was specified in the construction documents. Many factors lead to specifying the oscillating casing system which included:

- known presence of boulders in excess of 6 to 10 feet in diameter in fill
- location of drilled shafts next to protected waterway (eliminated wet construction methods)
- absence of testing of shafts (use of high slump concrete mix and smaller aggregate size to minimize voids)

Figure 21 depicts the oscillating casing system and drilled shaft rig used for construction. Figure 22 shows the rigid pile caps with approximate location of drilled shafts.
Figure 21. Oscillating casing system specified for the project.

Figure 22. Coupled shear pile constructed caps looking downdip of landslide. Yellow circles show approximate location of drilled shafts.
Summary

The Narrows project was awarded in the November 2011 and was completed by August 2012. The total length of the project was approximately 600 feet of roadway using 48 inch drilled shafts, for $6.9 Million or approximately $11,500/LF of roadway length. While these costs may be greater relative to other landslide mitigation systems, the mitigation option was designed to accommodate the future embankment and/or retain wall system to be constructed. Long term roadway build out in 2020 is planned to accommodate a four lane roadway section will be constructed and supported on top of the coupled shear piles.

Overall, the CSP systems are performing as they were designed based on recent inclinometer readings. Again, it was imperative to have the geotechnical/geo-structural designer involved in both the design and the construction process since a multitude of construction issues arose during the construction. Communication between the Owner, Contractor, Subcontractors and Designer of Record were imperative for the successful completion of this project.
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Ground Anchor Selection and Testing – Matching Ground Anchors to Their Desired Function and Testing Their Serviceability

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ABSTRACT

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Ground Anchor Selection and Testing – Matching Ground Anchors to Their Desired Function and Testing Their Serviceability.

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Ground anchor installations cover a broad range of uses from simple tiedown anchors used for telephone pole support cables to multi-strand tendons used for dam tiedowns that carry millions of pounds of tension. Many of the principles of grout to rock/soil bond and mobilized soil/rock mass volume, weight and strength are common to all types of anchors; however the design elements of different anchor types require specific testing to verify correct installation and functionality of each design element in order to ensure that the constructed ground anchor meets its design requirements and service life. This paper reviews the different types of anchors, anchor materials, types of anchor loading and levels of serviceability for common ground anchors, and discusses the need to tailor construction verification testing to suit different anchor types, function, and serviceability requirements.
INTRODUCTION

Ground anchors come in a variety of types and materials and are commonly used to transfer tensile loads to rock or soil in tieback or cable stay applications, as rock bolts to secure large blocks or wedges in place by increasing the frictional resistance along potential sliding planes or they can be used to impart a zone of compression in rock between an anchor head assembly and a bonded zone creating a composite arch in the case of roof bolting in underground applications or a reinforced rock mass in a fractured rock. Simple un-tensioned bolts or dowels are often used to increase sliding or overturning resistance at the base of footings or they can be used to resist dilation of rock joints and can be used to develop a “nail-like” connection on a joint plane between two rock blocks Spang and Egger (1). High energy rockfall catch barriers and wire mesh drapes often make use of wire rope anchors to tie back barrier posts and suspend drape panels down rock slopes. Many papers have presented design methods and analyses of anchor behavior and load transfer mechanisms. This paper focuses on the various types of ground anchors and different materials used for anchor construction to highlight the critical components of different types of anchors and the types of testing that should be performed to verify proper construction and as-built serviceability of the installed anchors.

ANCHOR TYPES AND FUNCTION

Ground anchors generally fall into two broad categories; passive anchors that are un-tensioned and loaded in response to external loading or displacement and active anchors that are post tensioned to incorporate a tensile load into the anchor during installation.

Passive Anchors

Passive anchors commonly consist of un-tensioned steel dowels, pins or wire ropes that are grouted in place. Several friction-type bolts such as Split Sets or Swellex anchors that are held in place by friction between the steel bolt and the borehole walls act as passive dowels. Eye bolts used for guy wires for steel towers or posts, rock dowels for pinning rock blocks together and wire rope anchors for rockfall mitigation systems also fall into this category of anchor. The functional behavior of passive anchors is dependent upon the type of loading, the material properties of the anchor; the grout to rock bond strength; and the rock/rock mass strength and volume of rock mobilized by the anchor.

Rock dowels installed as un-tensioned rock bolts to reinforce a rock mass frequently are designed to pin blocks together to limit joint dilation and mobilize the intact rock strength to resist block sliding or movement. Bars used for rock dowels can be loaded in an axial, lateral or inclined direction relative to the bolt axis depending on the application; however, bars loaded along the bolt axis maximize the strength of the bar. For this reason, installation of dowels for rock reinforcement is commonly normal to the direction of anticipated movement to keep loading on the bars predominantly in tension and to minimize shear or bending forces.
In all cases the direction of loading and the subsequent potential mode of failure must be considered when designing anchors so that the correct type of anchor is specified and the appropriate strength capacity is used in the design. Bar anchors should be oriented to avoid bending stresses whenever possible and with a recognized reduced strength capacity for applications where shear stresses are induced. Anchor orientation and materials should also be selected considering installation issues. Figure 1 shows a bar anchor loaded laterally that failed in bending at a load equal to roughly 10% of the tensile strength of the bar.

![Eye Bolt Failure in Bending](image)

Figure 1 – Eye Bolt Failure in Bending

Wire rope anchors are commonly used where non-axial loads may be encountered and are now a preferred ground anchor for use in wire mesh drape installations and rockfall barrier construction where the top anchors are typically loaded laterally or at an angle to the anchor axis. In such installations, wire rope anchors can be installed through overburden and non-parallel to the orientation of anchored loads (e.g., vertical anchors for a sloped uphill anchor rope on a rockfall barrier) without inducing unacceptable bending loads on a rigid bar anchor. Passive upslope anchors can also include deadman-type anchors that rely wholly on the strength of the soil mass mobilized by the deadman structure. Wire rope anchors are also used in a large number of applications where large-scale reinforcement is required such as powerhouse caverns in high-stress squeezing ground. For the Mingtan Pumped Storage Scheme in Taiwan *Hoek (2)*, 2-inch diameter passive grouted wire rope anchors (degreased hoist cable) were used around a machine hall excavation to limit ground movements as the excavation was slashed downward under high horizontal stress conditions.

When a passive anchor is loaded axially in tension, critical elements to be tested during installation are:

- Anchor tensile strength;
- Grout strength;
- Grout to bar and grout to ground bond; and
• Ground resistance mobilized (conical failure).

When a passive anchor or dowel is loaded transversely in shear, critical elements to be tested when the anchors are installed are:

• Anchor shear strength
• Grout strength; and
• Lateral ground resistance.

Anchor material and grout strength are relatively easy to determine in the laboratory from field samples/cylinders/grout prisms and mill/manufacturers certification sheets. Grout to ground bond can be tested through a sacrificial anchor test with a reduced bond length to verify an assumed bond value. Ground mobilization capacity is more difficult to test in that a large cone of rock and/or soil may be mobilized around an anchor (Figure 2, Wyllie (3)) and founding a reaction load for a jack system outside the zone of influence for this cone during the test may be difficult or impractical on slopes where limited or only high-angle access is available.

Anchors in soil pose an additional issue in that there is a component of passive earth pressure resistance in lateral loading and deflections of a foot or more may be needed to mobilize a soil anchor resistance as high as that for a rock anchor deflecting roughly an inch. Creep testing in soil should be performed and maximum test loads held to verify anchor capacity.

Horizontal and sub-horizontal/lateral loading of vertically installed wire mesh drape anchors is discussed in the Washington State Transportation Commission Report “Analysis and Design of Wire Mesh/Cable Net Slope Protection”, Muhunthan, et al. (4). The study suggests extending an attached cable from the anchor to the base of a slope and pulling on the cable with a reaction load (winch on a truck or other heavy vehicle) and measuring the load on the cable with a load cell device. The report further recommends testing at least 25 percent of installed anchors.
Lateral testing of wire rope anchors in rock for wire mesh drape applications can follow the procedures outlined by Muhunthan et al. (4), but may be unnecessary when anchors are installed in hard competent rock types. Confirmatory bond tests can be performed on sacrificial bars/wire rope anchors with short bond lengths to verify assumed bond values for each lithology the anchors are installed in, and a few vertical tests completed to verify the axial load capacity of the installed anchors. Testing becomes more important in weak or heavily jointed rock, where installation difficulties have been experienced, or in cemented soils where strain softening can occur upon loading. Figure 3 shows a rockfall barrier anchor in weakly cemented sand that failed during a barrier impact.

![Failed Rockfall Barrier Anchor](image)

**Figure 3 – Failed Rockfall Barrier Anchor**

**Active Anchors**

Active ground anchors are post-tensioned to impart a load on a structure or rock block/mass and tie a retained object into the soil or rock mass behind. Applications can vary from simple soil nails that receive tension in a mesh soil retention application to complex tieback tendons for retaining walls and dam tiedown anchors. Where load transfer behind a potential failure plane is desired (Figure 4, Weatherby (5)) it is common to incorporate an unbonded free stressing length in an anchor to avoid loading the soil or rock mass that is being retained.

![Anchors with a Free Stressing Length](image)

**Figure 4 – Anchors with a Free Stressing Length**
In some applications where the tensile load in the anchor is a critical design component, anchors can incorporate a free stressing length to allow for regular testing and re-stressing of the anchor. Typical components of a post-tensioned tieback tendon, including a bonded anchor length and free stressing length, are shown in Figure 5 Wyllie (6).

Where load transfer to a depth is essential, bolts are installed with a free stressing length to develop the anchor bond length beyond a potential failure plane or joint surface describing a rock block to be retained. The free stressing length can be developed by using mechanical anchors, two-stage grouting, using different set-time polyester resin cartridges, or building the anchor with a greased and sheathed bond breaker length as shown in Figure 5 to allow single-stage grouting.

The free stressing length can therefore be permanent as in the case of the single-stage grouted anchor with a sheathed bond breaker, or temporary when mechanical anchors, bars with two-stage grouting or resin grouting with two set-speeds are used and fully grouted (entire length of the anchor is encapsulated with grout) after tensioning of the anchor. Materials for tensioned ground anchors include steel bars and tendons made of multiple 7-wire strands of high strength steel. Bars are limited in application to lengths of available steel and the ability to transport and handle the lengths of steel (particularly for high angle access where cranes cannot be used). Bars can be coupled but bond breakers for coupled sections are often not used because of difficulties in fabricating joints in unbonded zones and resin cartridge installations do not allow for couplings. In many long bar anchor applications two-stage grouting is done. Multi-strand tendons are usually used for high load applications such as dam tiedowns and where long free stressing lengths are needed.

Testing of tensioned anchors is typically oriented axially with the anchor and at a minimum the load-carrying capacity, anchor development length (length of grout to bar/strand debonding that occurs in the bonded length as the load is taken up), and potential for long-term creep and anchor
relaxation are tested. Where a permanent load transfer is required, the effectiveness of the bond breaker and theoretical full elongation in the free stressing length is also assessed to ensure the load is not being applied in the retained soil/rock mass.

Tensioned rock bolts can also be used to improve ground conditions by creating a reinforced rock arch or beam by imparting a zone of compression (Figure 6, Wyllie (6)). In underground applications, the zone of compression creates a composite arch effect in a tunnel or cavern roof (Figure 7, Lang 7). In these applications, mechanical anchorages and fully grouting the bolt once tensioned, or installation of two-stage resin grouting (fast set anchor, slow set encapsulation) and tensioning once the anchorage is set are common practice.

When an active anchor is loaded axially in tension, critical elements to be tested when the anchors are installed are:

- Anchor tensile strength (capacity);
- Grout strength;
- Grout to bar and grout to ground bond; and
- Ground resistance mobilized (conical failure).

ANCHOR TESTING

Anchor testing provides verification of design assumptions; (e.g., grout to rock bond strength) as well as verification of as-built anchor condition and overall capacity. The Post Tensioning Institute (PTI) developed guidelines titled “Recommendations for Prestressed Rock and Soil
Anchors in 1980, *PTI* (8) and Dave Weatherby prepared an FHWA Report Titled “Tiebacks” in 1982, *Weatherby* (5) that established design, construction and testing guidelines for tieback anchors for retaining structures. The primary thrust of the guidelines was to present a uniform approach to design, installation and testing of anchors for temporary and permanent tieback walls and uplift anchors for dam tiedowns. The testing methods reflect the need to verify grout to ground bond/anchor capacity, development length, and free stressing length (effectiveness of load transfer). Testing guidelines such as the PTI guideline commonly include three main types of testing:

- **Performance Testing** – Usually 1.33 times the Design Load and is performed on the first two or three anchors and up to 2% of the remaining anchors to verify: load capacity; free stressing lengths have been satisfactorily established; residual movement/elongation based on cyclic loading; and that the rate of creep at maximum test loading is within specified limits.
- **Proof Testing** – Usually 1.2 times the Design Load and is performed on the remaining anchors during tensioning. The anchors are loaded incrementally and creep tested on the maximum load to verify: free stressing lengths have been satisfactorily established; and that the rate of creep at maximum test loading is within specified limits.
- **Lift Off Testing** – is usually performed on the anchors to verify lock-off loads are accurately established. Additional lift off tests can be performed on anchors that had creep rates approaching the maximum acceptable, or where lock off of the anchor was difficult.

Rock bolts can be post tensioned using a proof test with a creep component to verify anchor bond and load capacity and ensure that the bond was not creeping at 120 percent of design load. Following tensioning, the bolts are typically fully grouted or the second stage of polyester resin grout solidifies and the tensile load is grouted in.

Soil nails that are slightly tensioned as a wire mesh soil retention system is cinched down can be tested using a vertical/normal jack arrangement bearing outside the potential cone of soil/rock mobilized by the nail as one would for a dowel capacity test.

Anchor testing equipment commonly consists of a hollow cylinder hydraulic jack with an electric or manual pump and a pressure gauge (Figure 8) calibrated for the jack to give readings of applied load. Displacements of the anchor are usually measured during testing with simple plunger type dial gauges secured to an independent reference point outside the zone of influence of the anchor (i.e. outside the conical failure zone in the ground) *Carter* (9). Dial gauges are usually set to bear on the end of the bar or on the load plate. Where gauges are set to read on the loading plate it is common practice to average the readings from two or three gauges distributed evenly around the plate.
Appropriate Anchor Testing, Testing Problems and Pitfalls

To ensure the serviceability of installed anchors and verify construction quality, testing should be tailored to the type, function, and capacity requirements of the anchors installed. Too often standard testing language borrowed from accepted specifications for post tensioned tendons are applied to any rock bolt or ground anchor, irrespective of anchor material or type. The designer needs to identify the key attributes of the anchors being installed and then use judgment to select
the testing elements that will verify design assumptions and confirm the proper construction and capacity of the installed anchors.

A common error when developing and carrying out an anchor testing program is to test anchors that are not fully representative of the ground conditions to be encountered during production installations. Anchor testing programs should always include a representative number of anchors for each type of ground to be encountered and each loading condition. When selecting anchors for testing, consideration must be given to the types of soil or rock to be encountered during installations and the quality of the ground; (i.e. is the rock more highly weathered or fractured more in some areas than others). Representative bond and capacity tests should be made for each condition. Similarly, testing should include areas where changes in installation difficulty such as problems grouting or drilling occurred.

As noted previously, a common pitfall is to apply testing procedures from previous projects to subsequent projects even though the application is quite different. Because many agencies have developed standard specifications and procedures for installation of tieback walls and prestressed rock and soil anchors for those applications, the testing procedures adopted for tieback walls have often been required for installation of other ground anchors. The PTI guidelines caution that for certain anchors; (e.g., 2-stage resin grout), cyclic loading and later lift off testing are not applicable, nevertheless rigorous testing including cyclic loading has often been “cut and pasted” from one anchor specification to another even though the approach/applicability of the test may not be germane. This is particularly true for wire rope anchors that do not behave elastically the way bars and tendons do. Wire ropes elongate, take a “set” and have an initial permanent deformation. This behavior can be misconstrued as anchor failure or pullout. Typical 7 wire strand anchors can also fail during testing by rotation (unscrewing) in tests with short embedment lengths due to the low torsional rigidity of the cables compared to steel bars. Bawden et al. (10) have proposed a test procedure that prevents rotation of the tendons during testing.

Applying the PTI acceptance criteria to performance tests on fully grouted bars can also sometimes result in difficulties in that the bars may fail to meet the acceptance criteria for elongation. In some cases, this is due to limitations in the precision of the test equipment and test method rather than a failure of the anchor. Given the very small apparent “free stressing” length during testing of a fully grouted anchor (often limited to the stick-up distance between the collar and the stressing point on the hydraulic jack) it is readily apparent that the allowable elongation is very small (often hundredths of an inch) and can be difficult to measure accurately given the limitations of the test procedure and equipment commonly used for anchor testing.

The theoretical elastic elongation of a steel bar with free length is given by:

\[ \delta_e = \frac{P \times L_t}{A_t \times E_s} \]

where:

- P is the load
- L_t is the free stressing length
A is the cross sectional area of the tendon
\( E_s \) is the elastic modulus of the steel

As an example, a 1 inch diameter bar with a stick-up of 1 foot and an axial load of 22,500 lbf has a theoretical elongation of approximately 0.01 inches. Measuring axial displacements during performance testing of less than 0.01 inches can be difficult to achieve using conventional testing equipment. When testing inclined anchors it can be difficult to properly align the bolt axis with the hollow cylinder jack using shims or bearing pads. If the bolt is not properly aligned with the test jack, bending of the bar can occur during alignment loading and testing sometimes resulting in small apparent movement of the anchor.

Another common mistake made when testing anchors is measuring displacements without fixing the gauges to an independent reference point. Dial gauges are sometimes fixed to the bearing plate or surface adjacent to the anchor or to the jack itself, all of which are often subject to movement during test loads and can therefore result in excessive displacements due in part to the movement of the gauge reference point and not elongation of the steel or anchor slip.

**Proper Anchor Test Program Selection**

In light of some of the testing problems and limitations, the focus during testing should be on the critical elements and functional requirements that need to be verified so that anchor testing can be simplified, better tailored to the specific anchor applications being used, and can properly verify the quality of construction and anchor serviceability.

When designing an anchor test program a number of factors must be considered. These considerations include:

- The direction of load application during the service life of the anchor (i.e. axial, lateral or inclined)
- The potential modes of failure (i.e. bond failure, tensile, shear or bending failure of the anchor and/or failure of the ground around the anchor (conical/passive earth failure)
- Capacity of ancillary components such as plates, nuts, washers, “eyes”, in the case of bars and clamps and thimbles shackles and swaged fittings in the case of cables.
- Allowable creep and relaxation during the service life of the anchor.
- Allowable load reduction of the anchor during the service life.

Where possible, anchors should be tested in the same direction as the load direction during their service life (Figure 9).
For example, a vertical anchor such as an eye bolt used for a draped mesh system that will be loaded laterally should not just be tested in the axial direction to check for pull-out without proper consideration of a shear or bending type failure. In most cases this type of anchor installation will likely fail in bending at a much lower load than the tensile strength of the bar or the bond strength of the grout rock interface.

Lateral loading of wire rope passive anchors can be conducted as outlined in Muhunthan et al. (4) as shown in Figure 9. Potential issues with these arrangements can be access to the slope crest for equipment, and limits to the available reaction load due to the weight of available equipment and load losses in the cable where it passes over a slope crest and is subject to friction. Wire rope anchors in soil or soil and rock that are loaded laterally as for upslope drape anchor installations, rely in part on passive soil resistance. To mobilize the passive resistance, the anchor must displace the soil to engage the shear strength of the soil. The curved nature of the wire rope deformation will engage bearing resistance as well when the pressure exerted on the soil by the wire rope becomes oriented lateral and downward due to resisting anchorage at the bottom of the anchor. Anchors in soil alone should be designed with a deadman anchorage to help mobilize this additional lateral resistance. One or two sacrificial tests should be considered for each soil type that anchors are installed in to verify design capacity and production anchors inspected for construction quality.

Where creep and relaxation of the anchor over the service life are critical design elements then extended creep testing should be considered in lieu of the standard 10 minute test called for under the PTI guidelines. Ground conditions that favor potential relaxation (strain softening or collapsing soils) warrant special consideration. The anchor shown in Figure 3 installed in weakly cemented Aeolian sand failed upon impact at greater than the design load; however, testing on the anchors indicated that pressure grouting was necessary to achieve acceptable bond values and anchors would fail suddenly, without exhibiting significant creep, during maximum load creep tests.
Where a design tensile load must be maintained over the service life of the anchor it may be necessary to design an anchor to facilitate long term testing and periodically test and if necessary re-tension the anchor. In such cases an anchor with a permanent free stressing length and adequate corrosion protection would be required. In this type of application it is also critical that the head assembly of the anchor (plate, washer and nut) be protected from corrosion.

CONCLUSIONS

Anchor installations are varied, constructed with different materials and designed for differing functions and serviceability requirements. Testing is required to verify design loads, as well as assess construction quality, and the as-built inclusion of design elements such as free stressing length and effective load transfer. Accordingly, simple testing and a one-size-fits-all approach to test methodology cannot meet the requirements for all types/styles of anchors. A key element in developing appropriate test procedures for installation of ground anchors is judgment on the part of the designer and field inspector.

Ground conditions, anchor materials properties, and anchor function play an important role in selecting testing appropriate to anchor installation. Grout to ground bond and mobilized rock/soil mass characteristics can vary within an installation and testing needs to be completed to assess the impacts of such variability where present. Similarly, changes in installation ease, drilling or grouting difficulties may suggest the need for additional testing to ensure uniform performance of installed anchors.

Different anchor types and materials call for different test procedures to verify installation quality and functionality of installed anchors. Wire rope anchors do not behave purely elastically and are installed as passive, fully grouted anchors. Testing of wire rope anchors for elastic behavior and as-installed free stressing length is therefore not appropriate; testing should be used to verify assumed grout to ground bond value and for overall anchor capacity. Maximum test loads should be held for 10 minutes or more to assess creep. If loaded laterally, testing in the direction of the applied load should be done where possible. The importance of directional testing increases when anchors are installed in soil, with deadman anchorages, or when installed in weak, highly fractured or saprolitic rock. In these cases verification of the mobilized soil/rock mass and deflection needed to mobilize the required strength is important.

Rock dowels should generally be loaded and tested axially. Design bond value verification and overall anchor capacity are key factors to verify with rock dowels. Where loaded in a non-axial manner, bending and shear can affect anchor capacity. While this may have been considered during design, field testing can prove important to verify design assumptions and the effects of changes in load orientation from the design. High fiber stresses in bending can fail bar anchors well below the shear strength of the bar, and are concentrated by limiting the length of bar over which bending occurs. Bars grouted to the collar of a borehole tend to have highly concentrated fiber stresses when loaded perpendicular to their axis. Capacity increases when the bar is installed 45 degrees inclined to the load or the grout column is recessed below collar several inches or more allowing the bar to curve instead of bend sharply at 90 degrees. Ideally bar loading should be tensile and axial in nature to draw on the full tensile strength of the steel used.
The percentage of anchors tested should reflect the difficulty and complexity of the installations, variability of ground conditions, and anchor attributes that need to be tested. PTI criteria are germane to ground anchors with free stressing lengths, load transfer requirements, and elastic behavior. In such installations, all anchors are proof tested and the first few anchors are performance tested to verify design applicability and installation quality before proceeding with the remaining production anchors.

Simple anchors require verification of design assumptions (grout to ground bond value) and anchor capacity. Where tensioned as with rock bolts, creep testing to verify the anchor is holding before lock off may be appropriate. Tensioned anchors should be loaded to test load, held and backed off to lockoff as the second stage grout/resin sets. Passive dowels should be tested to test load, held to assess creep, and unloaded. Tensioned rock bolts will all end up being tested in order to bring them to the lock-off load. Passive dowels are generally tested at rates of 10 percent to 25 percent.

Where changing ground conditions are encountered or installation (grouting/drilling) difficulties are experienced, the field inspection team should have the flexibility to add anchors to the testing program. In selecting the program and anchors to be tested, judgment is important. If the ground conditions are good and installations are going well and initial test results are all good – the program can probably be relaxed to a lower percentage of anchors tested. If installations are difficult, ground conditions are poor and there are failing or near-failing tests, then additional confirmatory testing is required – or there may be a need to review design assumptions, installation techniques and/or anchor types being used.
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LiDAR-Based Rock Slope Assessment
State Route 8
Sandisfield, Massachusetts

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ABSTRACT

In spring 2013, a wedge-shaped failure released approximately 100 cubic yards of rock and debris from a slope, filled the catchment and southbound shoulder, and partially blocked the southbound lane of Route 8 in Sandisfield, Massachusetts. Fortunately, no one was injured.

A GZA and MassDOT response team visited the site and found saturated, heavily vegetated, 20- to 55-foot high, near-vertical rock slopes extending along ½ mile of roadway. Concrete Jersey barriers were installed to temporarily enhance the catchment and to buy time to clean up the fallen rock, clear the vegetation, and make a comprehensive assessment of the slope and catchment.

The comprehensive assessment included a LiDAR survey and hand measurements to characterize joints in the rock mass. The LiDAR allowed the team to survey about ½ mile of slope, in places 50-plus feet above the ground in three days of scanning. The software package Split-FX® was used to create a mesh from the LiDAR point cloud, interpret the mesh to identify repeating planar features (joints), group the joints into sets, and display the joint sets on lower hemisphere stereographic projections.

GZA evaluated the completeness of the LiDAR data set, then compared the results to the Brunton Compass-based readings and assessed potential sources of error. The resulting stereonets provided the basis for kinematic analyses of the rock slope. Split-FX® provided a large volume of structural data for the exposed bedrock, which was used to characterize variations in the structure along the slope and identify areas most at-risk for future rockfall. Split-FX® was also used to develop cross sections and block sizes that were the basis for CRSP modelling.
INTRODUCTION

A slope failure occurred in late June 2013 on Route 8, a State Highway in Sandisfield, Massachusetts, that sent rockfall onto the paved travelway. The slope failure occurred in the southerly portion of an approximately ½ mile-long rock slope on the west side of Route 8, which begins approximately at the Connecticut-Massachusetts state line and extends north.

For several weeks prior to the rockfall, persistent heavy rainfall had occurred at the site, creating ideal conditions for rock slope instability. The rockfall occurred during a period of heavy rain, releasing roughly 100 yards of rock debris, filling the catchment and paved shoulder below and partially blocking the southbound travel lane.

The Massachusetts Department of Transportation (MassDOT) responded immediately to clear the heavily traveled roadway and mitigate the impact to the traveling public. Photographs showing the conditions during the initial response, following partial removal of the rock fall, are shown in Figure 1. Concrete Jersey barriers were placed along the west edge of the southbound travel lane, allowing MassDOT maintenance crews to remove the fallen rock and debris, and also to improve the catchment in case of additional rockfall.

Figure 1 – Photographs Following Initial Clean-up Response by MassDOT

At the request of MassDOT, GZA GeoEnvironmental, Inc. (GZA) visited the site with MassDOT representatives to observe the failure area and the approximately ½ mile rock slope surrounding the failure area. At the time of GZA’s initial visit about two weeks after the rock fall, the rock and debris had been cleared from the road and catchment, and the Jersey barrier had been placed.
During this initial site reconnaissance, GZA found a saturated, heavily vegetated, 20- to 55-foot high, near-vertical rock slopes extending along the roadway. The failure had occurred in what appeared to be the tallest portion of the cut, but the vegetation was so pervasive that it was difficult to ascertain the controlling bedrock structure, slope heights, or extent of problematic areas. It was apparent that the rock slope face consistently contained at least a surface layer of loose rocks, several of which could be dislodged by hand. The catchment was littered with similar rock fragments along the entire rock slope. The project team agreed that a field exploration and measurement program was warranted to provide data suitable to evaluate the rock stability.

Action items following the site reconnaissance included clearing of slope vegetation and optical survey of the roadway and catchment areas by MassDOT, and an in-depth evaluation by GZA.

PROJECT AREA

This portion of Route 8 is a high-speed, rural two-lane road that carries commuting traffic between Massachusetts and Connecticut. The roadway parallels the Farmington River which flows along the east side of Route 8 in a predominantly north-south direction. The location of the rock slope studied for this project is indicated on the aerial photograph shown in Figure 2.

A total of approximately 2,600 lineal feet of existing rock slope located along the west side of Route 8 were evaluated for this project, with a height between 43 and 56 feet in the vicinity of the failure (Station 9+80¹) and typical heights varying from 10 to 50 feet. The average heights in different portions of the investigated rock slope are summarized in Table 1.

¹ Stations referred to in this paper reference the plans entitled, “Plan of Topographic Survey of South Main Street (Route 8) in the Town of Sandisfield,” prepared by GCG Associates, dated September 24, 2013. The baseline from these.
The roadway configuration is the result of a circa 1965 widening project that included soil and rock cuts to form the current slope. The 1965 plans\(^2\) called for a vertical rock cut face. Above the rock face, soil cut slopes were designed to range from 1 horizontal to 1 vertical (1H:1V) to 2H:1V. The design ditch between the slope and the roadway was 21 to 26 feet wide and approximately 3 to 4 feet deep. It was not feasible for the survey crews to collect data near the top of the slope to develop full slope cross sections due to inaccessibility of the top of the slope. Due to the limited survey data, there was no estimate of the anticipated post-widening rock slope height.

Based on the recent survey, MassDOT developed topography and cross sections of the existing roadway and catchment ditch from Sta. 3+00 to 29+00. The sections show that the catchment geometry is consistent with the design shown on the 1965 plans, except that the low point of the ditch was to be at the base of the rock cut by design. In 2013 the ditch was found to slope from the base of the rock cut toward the road with its low point typically 5 to 10 feet from the toe of slope. This condition may have developed over time as the recent colluvium consisting of fallen rock fragments and eroded soil has accumulated since the 1965 widening.

The rock slope was created by blasting methods. The slope appears 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. Mid-slope benches were evident in several areas, and were most apparent in the vicinity of Station 9+50 to 11+50. The benches were probably created during the original rock excavation, having formed along moderately dipping joint planes and were probably enhanced by subsequent rock fall. The rock slope surface shows signs of significant overblast damage in some locations, evidenced by open fractures and partially displaced blocks. The defects in the original cut face have also been enhanced by water and freeze thaw in the 50 years since it was cut. Subsequent sections describe

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\(^2\) The highway relocation plans are entitled, “Plan and Profile of State Highway in the Town of Sandisfield,” dated 1965.
the site specific geologic and seepage conditions observed during the field mapping phase of the project.

Photographs representing the range of rock slope conditions for the Sandisfield rock slopes are presented in Figure 3.

Figure 3 - Existing Rock Slope Photographs

The slope was heavily vegetated prior to the rock fall. MassDOT removed the majority of the vegetation on the rock face and within the catchment prior to the survey (as shown in Figure 3);
however, small trees and shrubs as well as grasses, moss, and soil buildup were still observed on multiple rock shelves.

OBJECTIVES AND PROJECT APPROACH

GZA set out to assess the cause of the 2013 rock fall incident, evaluate the rock slope stability, assess the potential for future rock fall risk to the traveling public, and make recommendations to mitigate the risk. The site presented many challenges in planning for data gathering. The steep, relatively high slopes had loose soil and rock that made traversing or accessing the slope from above problematic. In addition, the length of at-risk slope, roughly 2,600 feet, would require a significant time to map by hand-measurement.

The challenge was to gather field data as efficiently as possible and still maintain a level of detail sufficient to assess risk and develop mitigation measures. To that end, GZA opted to use terrestrial laser scanning survey techniques (LiDAR) to develop a point cloud that could be processed to extract and interpret the key characteristics of the bedrock structure. This was combined with a limited program of traditional hand measurements, sufficient to substantiate the bulk of the data which was gathered via LiDAR.

Once the rock mass was characterized, GZA evaluated the cause of the 2013 rock fall by evaluating kinematic instability including wedge, planar, and toppling modes. The results were enhanced by review of the joint spacing and continuity to establish block sizes for catchment evaluations, which were completed using CRSP. Based on the results of the CRSP and the kinematic analyses, the rock slope was divided into segments for planning of various rock fall mitigation activities.

REGIONAL AND SITE-SPECIFIC BEDROCK GEOLOGY

Bedrock geology within the site vicinity is identified by the United States Geological Survey Massachusetts Geologic Map Data as the Washington Gneiss. Two primary units of the Washington Gneiss mapped within the project area are described as 1) “well-layered, rusty-tan weathering muscovite-biotite-plagioclase-microcline-quartz granofels containing layers of rusty sulfidic calc-silicate rocks;” and 2) “coarse- to medium-grained hornblende-garnet amphibolite, hornblende-plagioclase gneiss and phlogopite-hornblende-plagioclase amphibolite (metabasalt).” A minor unit is also identified in the vicinity and described as “rusty-weathering, muscovite-biotite-sillimanite and/or kyanite-garnet schist; blue-quartz ribbed conglomerate, interlayered garnet-plagioclase-quartz metadacite.” The mapped bedrock geology in the site vicinity is shown in Figure 4.

Field observations by a GZA geologist were generally consistent with the bedrock mapping for the project area. The field observations noted muscovite-biotite schist and biotite-plagioclase-quartz gneiss intruded by or interlayered with garnet-muscovite-biotite-plagioclase-quartz granofels. The schist was dark grey, moderately hard, fine-grained and slightly weathered. The gneiss and granofels were grey, hard to very hard, medium- to coarse-grained, and fresh to moderately weathered. Minor intrusions of granite and diorite were also observed.

The study area has been heavily metamorphosed, and localized faulting, folding and intrusions were observed throughout the outcrop. Faulting, folding and reintrusion of older intrusions indicate the area experienced several distinct periods of metamorphism.
GZA observed areas of fragmented rock and breccia on each outcrop face caused by folding and faulting of brittle rock.

SEEPAGE CONDITIONS

Seepage in the form of wet surfaces, flowing water, and ice was observed up to 20 feet above the base of the rock face. Heavy moss growth and areas of leached calcium precipitated on the rock face indicate that groundwater seepage is fairly constant in some portions of the slope. The slope faces predominantly north, so ice is able to accumulate sooner and last longer into the season than would be the case for a more southerly exposure.

Surface water was observed actively flowing over the top of the rock face at one point along the outcrop, and differential weathering and staining of the bedrock surface were indicative of additional areas of past surface water or seepage flow.

FIELD MAPPING

Site Preparation

GZA and our subcontractors mobilized in early December 2013 to conduct the field mapping activities prior to onset of full winter conditions. MassDOT maintenance crews removed as much vegetation as practical prior to our mobilization. Due to persistent seepage and early-winter freezing temperatures, a considerable thickness of ice had accumulated over the rock slope in several areas, most of which were critical to include to our evaluations. Since the ice would hinder the hand and especially the LiDAR measurements, GZA mobilized an excavator to remove as much ice as possible. Photographs of the rock slope before and during/after ice scaling activities are shown in Figure 5.

Figure 5 – Rock Slope Beneath Failure Area, Before and After Ice Scaling
Hand Measurements and Visual Observations

Geologic field mapping was undertaken to provide a traditional hand-measured data set for evaluating the stability of existing rock cuts. The initial effort was to conduct a two-day geologic mapping effort, undertaken by a GZA geologist, to identify and map primary variations in the bedrock geologic features, including variations in rock type, intrusions, faulting and folding, and areas of differential weathering and seepage. The findings from this study were used to focus structural geology mapping.

The structural geology field mapping effort was conducted by a two-person crew of GZA engineers over a 4 day period between 12/10/13 to 12/13/13. GZA made a total of 328 direct measurements of bedrock joints and features, including 18 exposed at the ground surface within 5 feet of the base of the rock slope and 310 accessible with a bucket truck with a 70-foot-long extension arm between 10 and 35 feet above the ground surface. An above-ground telecommunication line extends along the slope face, roughly 15 feet above the ground. The bucket truck was utilized to work safely above and below the line. In addition, the catchment became ice-filled during ice scaling activities, further compromising the ability to work from a ladder.

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, both efficiency and accuracy were improved.

Each feature was assigned an identification number. The approximate locations of the mapped features were annotated on photographs of the rock slope, an example of which is presented in Figure 6.
Terrestrial LiDAR Scanning

GZA subcontracted Lamb-Star Engineering, L.P., of Plano, Texas to conduct terrestrial LiDAR scanning of the ½-mile long rock slope. GZA selected Lamb-Star to conduct the LiDAR based on their significant experience using the technology for mapping slope features.

Lamb-Star utilized a Leica Geosystems ScanStation C-10 laser scanning instrument for their work. The ScanStation C-10 features a single point accuracy of 6 mm and a data collection speed of 30,000 to 50,000 points per second. The spacing between instrument setup locations varied from 50 to 200 feet along the roadway east of the rock slope, and the scan distance from the instrument to the rock slope varied from 50 to 300 feet. Lamb-Star utilized 26 instrument setups and collected 200 million points. The LiDAR scan was completed in 3 days. A color image of the point cloud with color variation based on the reflectivity of the surveyed surface is presented in Figure 7.

Figure 6 – Photograph Annotated with Mapped Features
EVALUATION METHODOLOGY UTILIZING LIDAR

The LiDAR data was georeferenced at the site by Lamb-Star using existing survey benchmarks, allowing the point cloud to be processed into a series of *.XYZ files. After the raw files collected by Lamb-Star were processed to remove data that were clearly not representative of the rock slopes, 120 million x, y, z data points remained. In the typical point spacing areas, the distance between adjacent points was approximately ½ inch.

GZA utilized the analytical software Split-FX®, developed by Split Engineering, LLC, to convert the point cloud into a model that could be used to assess the bedrock structure and kinematic stability of the slope.

Point Cloud Data Interpretation

The method used by Split-FX® to assess structural geology involves creating a “mesh” and “patches” based on the point cloud data. A mesh is a polygonal surface model generated using the point cloud data. It is essentially a reconstruction of the surface geometry from densely sampled points. The user creates the mesh either based on an average number of points per triangle, which is used to automatically calculate the size of each mesh triangle, or by defining a uniform triangle size.

After the mesh is created, patches are “found”. Patches are planes fit to real discontinuity surfaces present in a cloud. Patches are created by grouping adjacent mesh triangles based on similarity of the orientation. The orientation (i.e., dip and dip direction) of a patch is developed.
using least squares to fit a plane through the points bounded by the grouped triangles (not the triangles themselves). Based on this method, patches can be sensitive to individual “noisy” cloud points, and customizable filters are provided to filter points and noisy patches. The user identifies the minimum patch size (number of triangles), maximum neighbor angle between adjacent triangles in a patch, and the degree of filtering.

We selected an approximately 50-foot-long portion of the rock cut as a test section and conducted a parametric study to assess the quality of the bedrock structural model resulting from several combinations of mesh size, triangles per patch, maximum neighbor angle and degree of filtering. A section of the rock slope was selected that had relatively large, readily discernible discontinuities so that visualization and interpretation of the results would be more straightforward.

A photograph of the parametric study test section and an image of the point cloud in that area are shown in Figure 8.

![Figure 8 – Split-FX Parametric Study Test Section](image)

The initial input parameters were selected based on guidance from the Federal Highway Administration Ground-Based LiDAR publication (No. FHWA-CFL/TD-08-006), which provides recommended Split-FX best practices. The recommended best practices are listed in Table 2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Best Practice Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Points per Mesh Triangle</td>
<td>13</td>
</tr>
<tr>
<td>Minimum Mesh Size (Triangles per Patch)</td>
<td>4</td>
</tr>
<tr>
<td>Maximum Neighbor Angle</td>
<td>10</td>
</tr>
<tr>
<td>Point Filter Level</td>
<td>Medium</td>
</tr>
<tr>
<td>Noisy Patch Filter</td>
<td>Medium</td>
</tr>
</tbody>
</table>

Our parametric study first focused on the points per mesh triangle and minimum mesh size, setting the three other parameters to the best practice value. We developed a lower
hemisphere pole plot of dip and dip direction readings collected with a Brunton compass to understand the representative discontinuity sets. These results were used to color-code joint sets of similar orientation to the hand readings by selecting the poles on the pole plot. Following each iteration, the mesh model with patches was moved to many different viewpoints in Split-FX to assess the similarity of the patches to the mesh and assess the degree to which known discontinuities were generated in Split-FX. We identified two sets of parameters that provided the best automated fit; the best practice parameters listed above, and a combination consisting of three (3) points per mesh triangle and eight (8) triangles per patch (3/8 mesh/triangle combination). For the point spacing in our point clouds, these combinations resulted in a similar sized minimum patch size.

We followed by altering the maximum neighbor angle and filter settings for the 13/4 and 3/8 mesh/triangle combinations. The best practice maximum neighbor angle was identified to be suitable. Lower filtering levels were selected to improve the match between the patches and the mesh, thereby creating a least-squares fit that most closely matched the mesh. Following identification of these parameters, we selected the 3/8 mesh/triangle combination. The selected site-specific mesh and patch generation parameters are summarized in Table 3.

<table>
<thead>
<tr>
<th>Table 3 –Selected Site-Specific Split-FX Mesh/Patch Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>Points per Mesh Triangle</td>
</tr>
<tr>
<td>Minimum Mesh Size (Triangles per Patch)</td>
</tr>
<tr>
<td>Maximum Neighbor Angle</td>
</tr>
<tr>
<td>Point Filter Level</td>
</tr>
<tr>
<td>Noisy Patch Filter</td>
</tr>
</tbody>
</table>

A screen shot of the color-coded patches based on the selected mesh and patch generation parameters is shown in Figure 9.
During the parametric study, we observed that a persistent low angle joint set with a dip direction primarily oriented towards the survey instrument was rarely found in the auto-search function. When zooming in closely to the mesh in these areas, we observed that the point cloud density became sparse in these areas as a result of occlusion areas. Occlusion areas result when the angle of the laser scan either near-parallel to or above planes of interest as pictured in Figure 10.
The occlusion areas found in our data set generally included points around the perimeter of an occluded plane, with an insufficient density to create patches. An example of the point cloud showing erratic or missing mesh triangles is shown in Figure 11.

Figure 10 – LiDAR Occlusion Areas
Because the scan did identify points around the perimeter of these planes, the Split-FX manual patch selection function was used to create patches for known planes. The patches were selected based on detailed review of rock slope photographs and field mapping results. This allowed for the model to capture representative planes that were occluded in the LiDAR data.

We also observed irregularities in the mesh where small areas of soil, vegetation, debris or ice were present on the slope and captured in the point cloud. We rejected the auto-generated patches in these areas, and replaced them with appropriate manually-generated planes.

Comparison of Hand Measurements and Split-FX Results

After we selected parameters and a procedure for extracting discontinuity data using Split-FX and before we proceeded with analyses, we used the hand discontinuity measurements to directly compare and check dip and dip direction measured with the Brunton compass to the data generated for the same locations in Split-FX.

The data sets were compared first as a whole and then as individual data points. We collected 328 hand readings during the field mapping program, and a lower hemisphere pole plot was generated based on this data. The dip and dip direction data from the entire ½-mile long rock slope was too extensive to combine into a single data set, so we initially focused on the test area discussed above. In this approximately 50-foot test area, we generated 1,028 patches from Split-FX representing discontinuities. The dip and dip direction of the readings were extracted as a data file from Split-FX and imported into DIPS® Version 5.1 by Rocscience (DIPS) for plotting on a lower hemisphere pole plot. These data were compared with 27 hand readings
collected in the same region of the cut, also plotted on a lower hemisphere plots. The resulting plots from DIPS are shown in Figure 12.

![Figure 12 – Comparison of Split-FX and Hand-Read Pole Plots](image)

A comparison of these plots shows similar grouping of steeper and lower angle joint sets, initially suggesting a good fit between the two data sources. However, upon closer inspection, the dip values were consistent between the data sets, but the dip direction from the hand readings showed a typical approximate 20 degree shift counterclockwise from the Split-FX-generated readings at similar locations.

To further explore this discrepancy, we selected 75 hand readings from various slope heights, orientations, and physical locations to represent the hand-collected data, and we extracted dip and dip direction from Split-FX for patches generated at the same location. We were able to identify the properties in Split-FX by comparing the hand reading location denoted on our slope photographs with the shape of the point cloud and mesh. We then numerically compared dip and dip direction for each corresponding pair of hand reading and Split-FX readings. There was a significant degree of scatter in the variation between the two data sources, but the comparison yielded the same trend as displayed in Figure 12. Dip values were typically within a few degrees between the two data sources, but dip direction varied by an average of 24 degrees for nearly all data pairs. Out of the 75 hand readings, all but 7 hand readings showed a shift counterclockwise from the Split-FX-generated data.

As discussed previously, a telecommunication wire runs about 15 to 20 feet in front of the slope and approximately 15 feet above the ground. Consequently, the wire was as close as 15 to 20 feet from the compass when we recorded the nearest hand readings, and further away as we moved above and below the wire. At the outset of the project, we did not consider that a single telecommunication wire would carry enough current to affect the readings, but we became suspicious that it could be the cause. We compared the range in dip direction discrepancy and average discrepancy for readings binned in five-foot height intervals to assess the variation with height. The results are presented in Figure 13.
Figure 13 – Summary of Dip Direction Rotation between Data Sets

The results suggested that all of the hand readings at heights greater than 5 feet above ground surface (up to a maximum height of about 35 feet above ground surface) should be “corrected” by modifying the dip direction, based on the assumption that the compass needle was primarily influenced by a consistent current. We developed a correction factor for the hand reading dip direction values of 24 degrees clockwise and applied it to all data between heights of 10 and 35 above ground surface. The results presented in Figure 12 are repeated in Figure 14 below, with “corrected” hand readings.

Figure 14 – Comparison of Split-FX and “Corrected” Hand-Read Pole Plots
We concluded that the telecommunication wire exhibited an unexpected magnetic pull on the compass during the hand readings, but application of a single correction value reduced the statistical variation between hand readings and Split-FX data to a level that could be anticipated within a hand reading data set alone.

**Evaluation of Bedrock Structure**

The structural data obtained from Split-FX was analyzed to identify the significant sets of discontinuities. Through completion of parametric studies and data verification, we had developed preliminary grouping of discontinuities into representative joint sets. All of the hand readings were corrected as described above, and a lower hemisphere pole plot was created for the data set. The entire Split-FX discontinuity data set for the rock cut, which consisted of 28,388 unique poles, was exported and also plotted in DIPS. The pole plots are shown in Figure 15, along with a concentration plot of the entire Split-FX data set. Please note that the concentrations are also plotted on the Split-FX pole plot but they are hidden by the complete coverage of poles.

![Figure 15 – Pole Plots for Entire Rock Cut](image)

While the plot of hand readings showed easily identifiable trends, the Split-FX pole plot was too dense to easily interpret trends. Subsequently, the Split-FX discontinuity data was parsed into groupings based on station ranges. This was easily accomplished using the Split-FX data files by assessing the northing and easting coordinates corresponding to each 100-foot station using the survey data and cutting the discontinuity data file at the corresponding northing and easting location. The pole density for each 100-foot segment was more manageable for a single plot, with between X and X unique poles per plot. For each 100-foot segment of the rock
cut between Sta. 3+00 and 29+00, the density of poles was contoured and plotted to assess the central tendencies and orientations of the most frequent discontinuities. Representative contour plots from several 100-foot segments are shown in Figure 16.

Figure 16 – Pole Plots of Split-FX Data by 100-foot Segment

Based on our evaluation of these plots, the discontinuities were grouped into representative joint sets for stability evaluations. The central tendencies of the segments shown above are also plotted on Figure 16. In general, the primary joint sets were reasonably consistent across the exposure and in comparison to the hand reading data. However, local variations in the trend of dip and dip direction were linked to localized structural geologic tends associated with folding and intrusions.
The variation was assessed along the length of the rock slope by plotting the central tendencies of dip and dip direction from Split-FX for each 100-foot segment of the cut against a line representing the central tendencies of the same joint sets for all of the hand readings, as shown in Figure 17. The data summarized above was used to break the overall rock cut into four regions, and the central tendencies of the primary joint sets were calculated as the average of the values for each 100-foot segment within the region. The central tendencies of dip and dip direction for primary joint sets are summarized by region in Table 4.
Figure 17 – Summary of Central Tendencies
Table 4 –Central Tendencies of Primary Joint Sets by Region

<table>
<thead>
<tr>
<th>Region (Sta. X+00 to X+00)</th>
<th>JS1 (Dip/Dip Direction)</th>
<th>JS2 (Dip/Dip Direction)</th>
<th>JS3 (Dip/Dip Direction)</th>
<th>Cut Face (Dip/Dip Direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 to 12</td>
<td>85/284</td>
<td>85/54</td>
<td>54/66</td>
<td>75/70</td>
</tr>
<tr>
<td>12 to 17</td>
<td>84/294</td>
<td>86/58</td>
<td>45/59</td>
<td>75/65</td>
</tr>
<tr>
<td>17 to 21</td>
<td>89/118</td>
<td>87/48</td>
<td>28/56</td>
<td>75/67</td>
</tr>
<tr>
<td>21 to 29</td>
<td>85/95</td>
<td>89/48</td>
<td>42/87</td>
<td>75/72</td>
</tr>
</tbody>
</table>

JS3 represents a planar failure discontinuity set relative to the cut face for nearly all of the cut, with the average dip direction within 15 degrees of the joint face orientation in all locations. JS1 and JS2 are persistent near-vertical joint sets that can free rock falls. The clearest and most relevant variation between regions assessed from the data was the steeper dip of JS3 in the region from Sta. 3+00 to 12+00, the same region that included the 2013 failure zone. It was also observed that the joint spacing is generally larger in this part of the cut than in areas further upstation (north). Based on the combination of these factors, we concluded that the area of the failure is the highest risk location for future rock fall from a kinematic standpoint. Kinematic analysis plots for each region are presented in Figure 18.
Rockfall Analyses

As discussed and displayed in photographs previously, the existing rock slope geometry varies significantly along the alignment. In general, this is a result of the slope following persistent discontinuities, resulting in a series of near-vertical slopes and terraces of various inclinations. With a conventional data set consisting of rock slope survey data or cross sections and slope heights, it is very difficult to characterize critical cross sections accurately. This is an important limitation since computer-simulated rockfall analyses are highly dependent on the accuracy of the cross section geometry.

For the Route 8 project, there was no survey data available to characterize the slope, and it was not practical or safe to manually collect sufficient data to characterize the cross sections. Split-FX was used to develop cross sections using a simple and straight-forward cross section function. This procedure consisted of dragging a line across the mesh at the location and orientation of interest, and generating a graphical interpretation of the rock slope cross section in Split-FX. The cross section coordinates were then saved as text files representing the rock slope face in two dimensions. Since shoulder and catchment were occluded in the LiDAR survey, the rock slope face data were integrated with the 2013 MassDOT survey sections of the shoulder and ditch to create a representation of the complete slope geometry for use in catchment evaluations.

The resulting rock slope and catchment geometry 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. 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.

Cross sections were selected for rockfall analyses based on slope height and potentially adverse geometry, primarily due to terraced slopes identified as potential launch points. Two methods were investigated for assessment of representative rock sizes. The first method consisted of a graphical approach, which was based on approximate graphical representation of joint sets extending into the rock mass at the orientations exposed in the cross sections. For this method, a potential wedge two-dimensional cross section could be estimated, and the third dimension (normal to the cross section) could be estimated based on photographs and/or observation of the point cloud. The second method consisted of assessing key blocks in the point cloud and measuring traces. Based on the large-volume rockfall that occurred in June 2013, it was concluded that traces of smaller surficial wedges or blocks may be inadequate to fully characterize the rock fall, so the graphical method was used to select 1 to 2 representative rock sizes at each cut. Examples of representative cross sections analyzed using CRSP are presented in Figure 19.
The sections modeled between Sta. 8+00 and 10+00 show similarities in the degree of near vertical and moderately dipping areas. However, the precision of the LiDAR data suggests that the section at Sta. 10+00, just north of the failure area, has a higher likelihood of rock being released near the top of the slope, combined with a short, moderately to steeply dipping terrace which represents a launch point. It should be noted that at Sta. 9+80, the geometry was altered by the 2013 rockfall, which created the longer sloped terrace area in comparison to Sta. 10+00.

The results of the CRSP evaluations for these three cross sections are presented in Table 5. Catchment is judged to be inadequate at Sta. 10+00 and acceptable at Sta. 8+00 and 9+80, based on a criterion of at least 95 percent of fallen rocks retained outside of the paved shoulder.

<table>
<thead>
<tr>
<th>Station</th>
<th>Diameter (ft)</th>
<th>Rocks Passing (out of 500)</th>
<th>% Retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>8+00</td>
<td>2</td>
<td>3</td>
<td>99</td>
</tr>
<tr>
<td>8+00</td>
<td>4</td>
<td>48</td>
<td>90</td>
</tr>
<tr>
<td>9+80</td>
<td>2</td>
<td>19</td>
<td>96</td>
</tr>
<tr>
<td>9+80</td>
<td>5</td>
<td>24</td>
<td>95</td>
</tr>
<tr>
<td>10+00</td>
<td>10</td>
<td>104</td>
<td>79</td>
</tr>
<tr>
<td>10+00</td>
<td>10</td>
<td>105</td>
<td>79</td>
</tr>
</tbody>
</table>

The results presented above suggest that individual rocks falling at the site would generally be assumed to be retained in the catchment. The significant volume of small to medium blocks observed in the catchment support this conclusion. However, the extensive rock fall that occurred in June 2013 is not easily modeled as a single block, especially since it would
have filled the catchment at some time during the failure, allowing following rocks to travel more easily into the road.

FINDINGS AND LESSONS-LEARNED

MassDOT approached GZA with a rock slope assessment that had a number of challenges, including a potentially hazardous work environment, extensive rock face area, limited available data, and a desire to conduct the field work and develop evaluations and recommendations efficiently. Terrestrial LiDAR, processed using Split-FX, was identified as an excellent solution to all of these challenges. Some of the advantages gained using the LiDAR in comparison to conventional hand-readings and survey include the following.

- Approximately 80,000 square feet of rock face was mapped to collect 28,000 discontinuity measurements in three days. By comparison, collection of a conventional data set 10 percent of this size (which might have been the goal using hand measurements) would have taken a two-person crew several weeks.
- The field work was completed with minimal risk to GZA engineers, geologists and subcontractors. By comparison, performing extensive rappelling and taking hand measurements at the toe of the slope would require good planning and careful controls to limit worker health and safety risks.
- The significant volume of discontinuity data developed along the entire cut gave insight into localized variations in the trends of dip and dip direction along the slope. These data were used to characterize regions with similar structure and to flag the areas of most significant hazards. Identifying these trends would be much more difficult using the smaller data set typically attainable via hand measurements.
- Highly detailed rock slope sections were extracted from the Split-FX-generated mesh and combined with conventional survey of the catchment area for use in rockfall simulation. These sections are significantly more representative of the critical slope areas than sections derived from typical roadway survey or by collecting limited optical survey points along a section. Consequently, the reliability of the catchment evaluation was greatly enhanced by the use of the LiDAR based data.

There were also a number of lessons learned from our work, some of which are listed below.

- During the terrestrial LiDAR planning phase, careful thought should be given to locations of instrument setups. Topography and distance from the area to be scanned should be evaluated to limit the degree of occlusion. Even with detailed planning, limited occlusion is very difficult to avoid. Planning should also consider means of filling these data gaps, either with hand measurements or by using alternative scan angles.
- The accuracy and usefulness of the data set is negatively influenced by the presence of any deleterious inclusions, such as vegetation, soil, debris or ice. The slope to be scanned should be cleared as much as practical of obstructions prior to scanning to allow development of a high quality mesh.
It is important to substantiate the LiDAR data with hand readings (or in this case, visa versa), as the use of this technology for rock mass characterization is still in early development.

Brunton compass data should be checked in the field for influence in areas of transmission lines or other significant metal objects.

**FUTURE WORK**

The next phase of the project is planned for summer 2014 and will consist of scaling and stabilization of the rock slope. The remediation process is expected to include removal of additional vegetation that could worsen stability, hand-scaling of loose and/or particularly adverse wedges and planes to achieve relatively uniform surface conditions and geometries, drilling sub-horizontal holes into the slope to intersect and drain water-bearing discontinuities and strategic placements of rock dowels and shotcrete buttresses, potentially combined with rock fall drapes, to limit the impact of future rock fall.

GZA plans to monitor the scaling and stabilization work in the field and assess the resulting rock face for potentially unstable features. The Split-FX models will prove valuable during the remediation phase, allowing us to assess the properties of several nearby discontinuities, confirmed by a handful of hand measurements obtained by a GZA field representative rappelling over the slope using ropes and a harness. We anticipate that the combination of data sources, some of which can be assessed immediately when a wedge or plane is identified, will improve the accuracy and efficiency of the remedial slope improvements. GZA will analyze and provide design recommendations and details for the specific stabilizations, and the specialty contractor will evaluate and propose materials and methods.
REFERENCES


Utilizing InSAR for Landslide Asset Management in Colorado

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Disclaimer

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ABSTRACT

Interferometric Synthetic Aperture Radar (InSAR) is a satellite-based remote sensing technique that has the ability to observe ground deformation at a centimeter-scale. Currently, AMEC Environment & Infrastructure, Inc is performing a Landslide Asset Management Pilot Project for the Colorado Department of Transportation (CDOT) to assess the applicability of incorporating InSAR technology into a risk based Geotechnical Asset Management (GAM) plan.

The goal of this project is to evaluate InSAR as a method for monitoring ground displacement. A primary objective is to determine if InSAR can be used as an indicator of slope or other geological hazard-related movement that could affect the conditions of highways or indicate a potential for a geotechnical asset condition change. For the pilot study, CDOT select three existing landslides to evaluate: 1) Slide Creek Slide (I-70 MP 212), 2) Vail Golf Course Slide (I-70 MP 177), and 3) Jackson Mountain Slide (US 160 MP 151).

The ground deformation interpreted by InSAR in this preliminary analysis has been shown to closely match existing data for the three pilot areas and is able to delineate areas experiencing known and potentially unrecognized ground deformation. InSAR is also able to quantify a rate of movement as defined by line of site from the satellite that should prove to be useful in geotechnical asset management. Preliminary results including interferograms and deformation maps for the three pilot study locations are discussed.
INTRODUCTION

This paper discusses the preliminary results of the Interferometric Synthetic Aperture Radar (InSAR) analysis for a pilot project at three historical landslide areas in Colorado. This investigation was performed by AMEC Environment & Infrastructure, Inc. (AMEC) for the Materials and Geotechnical Branch of the Colorado Department of Transportation (CDOT). The goal of the investigation was to evaluate InSAR as a method for monitoring ground displacement. A primary objective is to determine if InSAR can be used as an indicator of slope or other geological hazard movement that could affect the condition of highways or indicate the potential for a geotechnical asset condition change.

CDOT selected three existing landslides to evaluate for the pilot study. The three landslides are summarized below and locations are shown on Figure 1 for Sites 1 and 2 and on Figure 2 for Site 3.

- Site 1: I-70 milepost (MP) 212 (Straight Creek Slide): This landslide is located on I-70 at mile marker 212, just west of the Eisenhower/Johnson Tunnels and is also known as the MP 212 Slide.
- Site 2: I-70 MP 177 (Vail Golf Course Slide): The Vail Golf Course slide is located on I-70 near mile marker 177, on the east side of Vail.
- Site 3: US 160 MP 151 (Jackson Mountain Slide): The Jackson Mountain slide is located on US 160 near mile marker 151, between Wolf Creek Pass and Pagosa Springs.
In addition to specifically evaluating the three pilot study landslides, this investigation also evaluated the potential of InSAR processing to identify areas that are susceptible to ground movement that has not yet been identified and could affect highway mobility in future. This additional evaluation is limited to areas in close proximity to the three landslide sites.

Additionally, this study will evaluate the potential for InSAR to be used as a method of monitoring ground movement rates. The current work is being performed with the overall goal of incorporating InSAR technology into a risk based Geotechnical Asset Management plan.

**InSAR**

Synthetic aperture radar (SAR) is a microwave imaging system capable of measuring ground deformation at a centimeter-scale. This technique requires two radar images of the same area of the Earth’s surface, taken from the same point in space at different times. When the phase history of reflected waves for the two images are processed and compared, a detailed look at any changes in the shape and position of the reflecting surface that occurred in the time elapsed between the two scenes is obtained. The image of the phase difference is called an interferogram and the pattern and amount of deformation is customarily shown by using color fringes from the visible spectrum. If the difference in phase is 360 degrees, the distance that a radio wave travels back and forth between a radar and the ground changes by just one wavelength. A full color cycle in the interferogram represents ground deformation equal to half the wave length.

**Figure 2 – Site 3 Location**

![Site 3 Location Map](image-url)
The change in the elevation of a point on the earth surface along the line of sight (LOS) can be measured from the phase difference obtained from two different images obtained between two time periods and represents ground deformation that occurred during that timeframe. Though InSAR is a very good technique for ground deformation observation and monitoring over large areas there are still some risks and limitations that exist for its use. For each selected image pair, several processing steps have to be performed after the data has been acquired to determine its quality and suitability for engineering and geologic use. The image quality, spatial and temporal baseline, and atmospheric factors all influence InSAR accuracy. The following risk factors influence the generation of an interpretable interferogram:

- Weather conditions (i.e. snow cover) are a critical factor in the data acquisition by the satellite and should be considered for the scheduled date of acquisition.
- Errors can be introduced to the data set due to atmospheric (rainfall and snowfall) and terrain (steep slopes and/or mountainous terrain) effects.
- Availability of future data due to technical issues with the satellite(s).

PRELIMINARY INSAR ANALYSIS AND HISTORIC GROUND DEFORMATION

The ground deformation from September 2007 to September 2009 was interpreted using Advance Land Observation Satellite (ALOS) L-band InSAR data. The preliminary results presented in this paper include the deformation analysis from following InSAR data sets presented in Table 1. The same InSAR data scene was used to analyze data for Sites 1 and 2 and a separate data scene was utilized for Site 3.

| Table 1 – InSAR Data Sets Used in the Deformation Analysis |
|-----------------------------------------------|-----------------------------------------------|-----------------------------------------------|-----------------------------------------------|
| Data Type | Location | Start Date | End Date | Pixel and Scene Size |
| ALOS L-Band | Sites 1 and 2 | 9-15-2007 | 9-20-2009 | 10m 70 x 70km |
| ALOS L-Band | Site 3 | 9-15-2007 | 9-20-2009 | 10m 70 x 70km |

The automated interferometric processing software DIAPASON (Differential Interferometric Automated Process Applied to Survey of Nature) was used to process the InSAR data. Post processing of the interferogram data was performed using the latest unwrapping tool with DIAPASON software, and programming developed by AMEC. The interferograms produced by InSAR analysis were georeferenced using geographic information system (GIS) programs.

Initially, interferograms were created for the whole data scenes without using the digital elevation model (DEM). The full scene interferograms for Sites 1 and 2 are presented in Figure 3 and for Site 3 in Figure 4. As seen in the figures, the interferogram contains numerous fringes mainly showing topographical terrain effect. To remove the terrain effect, the interferograms were processed with digital elevation data utilizing the Shuttle Radar Topography Mission (SRTM) DEM. A data subset was selected for each site for further processing to remove artifact
terrain effects. The subset site boundaries for each site are shown in Figures 3 and 4. The interferogram for each subset site was produced using the 10-meter horizontal resolution SRTM data and further refined using various filtering methods.

Figure 3 – Full Scene Site 1 & 2 Interferometric Image from September 2007 to September 2009 without DEM
Figure 4 – Full Scene Site 3 Interferometric Image from May 2008 to May 2010 without DEM

The subset interferogram for Site 1 is presented in Figure 5. In the interferogram, one complete color cycle represents about 4.6 inches (11.5 centimeters) of relative elevation change. A complete color cycle was not observed from the interferogram near Site 1, however ground deformation close to the highway was observed. Similarly though a complete color cycle was not observed in the interferogram for Site 2, as presented in Figure 6, ground deformation was observed. The interferogram for Site 3 is presented in Figure 7 and very little ground deformation is observed.
Figure 5 – Subset Site 1 Interferometric Image from September 2007 to September 2009 using DEM

Figure 6 – Subset Site 2 Interferogram Image from September 2007 to September 2009 using DEM
Successful unwrapping of an interferogram is a critical step in interpreting the displacement/deformation data from the interferograms. Unwrapping determines the actual phase difference due to subsidence in each pixel using algorithms within the processing software. Unwrapping was performed for Site 1, 2, and 3 to interpret the ground deformation at each site.

The contours of ground movement for Site 1 are presented in Figure 8. Many areas showing ground deformation occurring during the two-year period covered by the interferogram are present at Site 1 and other locations along the highway corridor. An area showing movement of up to 2.4 inches (6 cm) was observed near Site 1. The ground deformation for Site 2 as interpreted from the InSAR data is presented in Figure 9. Movement of up to 3.6 inches (9 cm) was observed at many places near Site 2. There was no ground movement found near Site 3 as identified from the ground movement contour map for Site 3 presented in Figure 10.
Figure 8 – Subset Site 1 Ground Deformation & Historic Landslide Area

Figure 9 – Subset Site 2 Ground Deformation & Historic Landslide Area
DISCUSSION

InSAR derived ground movements were interpreted within areas surrounding Sites 1 and 2. Site 1 is the Straight Creek Slide located on I-70 near mile post 212, just west of the Eisenhower/Johnson Tunnel. The historical landslide area near Site 1 is presented in Figure 8. In the past, this landslide has caused vertical and lateral movement of both the eastbound and westbound lanes of I-70. The majority of the observed movement has been in the form of roadway settlement and CDOT has displacement data from inclinometers for 2008 and 2009. Historically about 1 inch (2.5 centimeters) (CDOT Memo, 2005) of annual ground movement has been reported at this site from inclinometer data. The interpreted ground movement areas from InSAR data coincide with the location of the historical landslide at this location. Additional areas of movement are indicated in areas near Site 1, and most coincide with the corridor of I-70. These other areas of ground deformation may indicate landslide and/or rockfall areas.

Site 2 is known as the Vail Golf Course slide and located on I-70 near mile marker 177, on the east side of Vail. Historically, movement of the western lobe of this landslide causes heaving of the interstate as indicated by the highway damage (CDOT Memo, 2011). In May 2011, a slope failure occurred near the eastern edge of the landslide causing boulders and soil to fall onto westbound I-70. CDOT has displacement data from inclinometers for this landslide since 2005. USGS mapping also identifies the active landslide area close to the highway. The historical landslide area and USGS slide map (USGS, 2003) are presented in Figure 9. The InSAR derived ground deformation map coincides with the historical landslide area. It is not possible to directly compare the movements from inclinometer data to the InSAR since...
inclinometers measure lateral movement at depth while InSAR measures LOS ground surface movement.

Site 3, known as the Jackson Mountain slide, is located on US 160 near mile marker 151, between Wolf Creek Pass and Pagosa Springs. This landslide has caused damage to both the eastbound and westbound lanes of US 160 in the past and CDOT has inclinometer data from August 1987 to March 1988. As recent as 2007, ground movement was not reported at this site (Yeh and Associates, 2007). InSAR analysis didn’t show any deformation at this site during September 2007 to September 2009 as shown in Figure 10.

CONCLUSIONS

The ground deformation interpreted by InSAR in this preliminary analysis has been shown to closely match existing data for the three pilot areas and is able to delineate areas experiencing known and potentially unrecognized ground deformation. InSAR is able to quantify a rate of movement as defined by LOS from the satellite that should prove to be useful in geotechnical asset management. Advanced processing should be expected to obtain useful information in areas of mountainous terrain. InSAR can be a useful tool for assessing slope movement or other geological hazards that could affect the condition of highways or indicate the potential for a geotechnical asset condition change.
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Case Histories On Light Anchor Post System For Flexible Barriers

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ABSTRACT

The Geohazard mitigation has a well-established testing guideline for flexible fence for rockfall protection, like the European testing guideline ETAG 027 or the USA guideline NCHRP 20-07 Recommended Procedures for the Testing of Rockfall Barriers. These two test procedures describe very well the functionality of a rockfall barrier and describe all the steps necessary to carry out a full scale test and consequently outline the deformability, resistance, performance and forces acting on the anchor post system of a deformable rockfall fence.

The anchor post system (foundation) is not considered as part of these testing guidelines. Designers must design the anchoring system considering the different national standards and the geotechnical characteristics of the site.

Oftentimes problems arise from the post anchor system, due to remote access to site locations. Not only does a remote location interfere with the operation of heavy machinery, but also transportation is very difficult. In these situations, the anchor post system can become more expensive than the fence itself, which could result with a rejection of the rockfall barrier, due to the high global cost of the intervention.

When a severe rock impact damages a barrier, for safety reasons, a large portion of the structure must be replaced and oftentimes the anchoring system should also be replaced. Then the question becomes whether or not heavy anchor post systems make sense.

This paper presents case studies of Maccaferri rockfall barriers impacted by several blocks. Even though the fences were installed with unusual light post anchoring systems, the fences were able to withstand impacts exceeding their nominal capacity. The analysis of the structure suggests that in certain conditions, displacement and settlement on the anchor post systems represent a benefit.
1.0 INTRODUCTION

Even if rockfall barriers are one of the most cost effective remedial solutions as protection against rock fall events, quite often their design remains quite uncertain and requires a certain degree of engineering judgment. The resistance and the height of the structure seem to be the only parameters to be considered during rockfall barrier designs. In reality, several practical problems must be taken into account: rockfall barriers are frequently built in remote or inaccessible areas, where their construction can be extremely difficult and expensive. For instance, several times the anchor post systems are designed with large reinforced concrete plinth and deep micro piles (see figure 1.1).

Figure 1.1 – Very large concrete plinth specified on a 2000 kJ barrier project.

Also maintenance costs must be evaluated, because after severe impacts barriers must be completely replaced and their anchor post systems may not be reliable anymore. At the end, designers should have answers to the following issues: how is it possible to reduce the barrier maintenance while increasing the lifespan? Can the anchor post systems, which require a large construction effort, be minimized?
2.0 ROCKFALL BARRIER BEHAVIOR AND DESIGN APPROACH

The behavior of rockfall barriers can be defined mainly by full-scale crash tests, which can be used also for the calibration of numerical models. At present the most important procedure to carry out the crash tests is described by the European Guideline ETAG 027 - 2008 (“Guideline for European Technical Approval of Falling Rock Protection Kits”). Unfortunately, ETAG 027 must be considered as index test and the rated energy capacity of the barrier as nominal (Brunet et Al., 2013). For this reason designers should apply reduction coefficients to the nominal energy capacity of the barrier defined by means of a crash test. Designers should also consider the uncertainties affecting the rockfall analysis such as the parameters related to the geomechanical, topographic and geologic surveys.

The new Italian standard for “Rockfall protective measures” (UNI 11211:4 – 2012) represents a very interesting and innovative design approach that takes into account the reliability of the mentioned surveys suggesting the safety factors to be considered in the various situations. Moreover, the standard allows considering both the maximum (MEL) and serviceability energy level (SEL), as defined by the ETAG 027. 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 requires a barrier with a capacity 3 times higher than the minimum required. A typical application of SEL-design is the protection for tunnel portals. MEL criterion is normally adopted when there is a low frequency of rock fall or only one boulder is expected to fall, if the maintenance can be easily done and/or if the risk level allowed is high. 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 (Grimod and Giacchetti, 2013). This design approach represents the first solution aimed at increasing the lifespan and reducing the maintenance costs.

3.0 ANCHOR POST SYSTEM DESIGN AND FIELD EXPERIENCE

Barrier anchor post systems should be designed considering the forces directly measured or calculated by crash tests (Turner et al., 2009). According to ETAG 027:2008 and to UNI 11211:2012, rockfall fences can be designed at the service or ultimate limit state. In both these cases, anchor post systems must be designed considering the forces at the maximum energy level, the geotechnical parameters of the soil and the appropriate national provisions. For many designers this means transferring their knowledge from the structural engineering to the deformable rockfall barriers. In these terms the post foundations are generally built with large reinforced concrete plinths, as they have to resist any settlement and force.
Field experience shows that when the barrier withstands a MEL impact, the damages can be very severe and often it is more economical and safer to rebuild the barrier than trying to repair the damaged one. Even after a large impact the damages seem to be light or negligible, the probability of micro ruptures on wire rope cables, posts, meshes and connection components is very high. Thus, the barrier anchor post systems could suffer of any type of cracks especially on the pins connecting the footplates. With regards to the footplate system, very rarely the “fuse” device aimed at saving the anchor post system is effective.

It is a totally different behavior for a fence designed as per the SEL energy impact. In this case, the damages are usually very small and the required maintenance is negligible. The anchoring systems usually do not have any damages and rarely anchors need to be replaced. In this case, very large concrete plinth designed for MEL impacts, appears redundant and not cost effective.

Additionally, the stiffness of the concrete plinth might cause issues during impact for the fence, because the bottom of the post is not able to deform as much as the upper portion. Thus, a “deformable” anchor post system may help to dissipate even more energy and make the barrier safer. This concept forces many designers, who are used to building and structural design, to change drastically their mentality when designing rockfall barriers. In these terms the concrete plinths have to be thought as aimed at getting a regular support surface for the footplate of the posts and making their installation easier. Its construction should be fast and easy in any environmental condition. The plinth contribution to the bearing capacity of the footplate may be negligible. With this approach, the contribution of the anchor post system can be neglected during the design of the anchor post systems.

The steel bars must directly connect the footplate to the ground or the rock mass. In the case where a concrete plinth is foreseen, the bars have to pass throughout the plinth in order to allow the support of the footplate and to make the anchor post system more flexible. By using a
flexible anchor design for the post anchor system and allowing the footplate to slightly move during impact will contribute to reduce the risk of ruptures on the footplate itself and of the collapse of the barrier. By allowing small movement of the post anchor system, the rockfall fence can maintain an appreciable residual height, and consequently increase the level of safety after the first impact. It has to be underlined that, if the residual height is greatly reduced, the barrier cannot catch any secondary impact.

It is important to highlight the fact that the Austrian agency for forest and avalanches (WLV) suggests that the footplate should lie directly on the ground, without any concrete plinth but only fixed by means of anchor bars (as shown in fig. 3.1).

### 4.0 CASE HISTORIES

4.1 Highway A3 Salerno-Reggio Calabria, Municipality of Scilla (Italy): 5000 kJ.

An interesting case of behavior of flexible anchor post systems was observed at the pk 425 of the Highway A3 (Salerno-Reggio Calabria), which is the most strategic and important road in southern Italy.

In May 2010 a large volume of rocks (approx. 8 m\(^3\) (10.5 yd\(^3\))) reached the HWY. Fortunately, they didn’t have any casualties from this rock fall event (fig. 4.1 – on the right). The detachment area was located approximately at 250 m (820 ft) in elevation above the road. In this area, a vertical cliff crowns a 35° inclined slope. The total volume of rocks was 1200 m\(^3\) (1570 yd\(^3\)), where 85-90% of the debris were still located within the first 50 m (164 ft) from the toe of the cliff, and 10-15% of the debris have continued their fall for another 60 m (200 ft).

![Figure 4.1](image_url)

Figure 4.1 Large block fell on the Highway A3 at km 425 (2010 - Italy).

The local administration asked for a very fast and economical remedial solution, because the road could not be closed for a long period of time, even if an alternative route was under construction but not yet operational. A geomechanical survey pointed out that on the detachment
area there was another 1000 m³ (1300 yd³) of unstable rock mass. This situation was very risky for the safety of the workers as well as for the traffic along the highway.

The intervention has been done in the following order: (a) temporary barrier was installed next to the highway made of two levels of containers filled of sands; (b) the unstable cliff was draped using an HEA cable panels aimed to protect the workers, and containing any possible rock fall and also reduce their energy; (c) primary rockfall barrier line was installed to catch large multiple rock impacts; (d) subsidiary rockfall barrier was installed just below the main primary barrier (figure 2 on the left).

The best location of the primary rockfall barrier has been determined using rockfall simulation software. The rockfall trajectories analysis was calibrated considering the distribution of the end-points in the case of blocks with 70 m³ (91 yd³) in volume. The energy capacity, the height and the location of the barrier were defined following the UNI 112111 design standard, the MEL design criteria, and previous experiences on large impacts (Giacchetti et al., 2010). As per the calculation, a 5,000 kJ deformable rockfall barrier, 6 m (20 ft) in height and 70 m (230 ft) long was required. In order to reduce the installation time, and consequently increase the safety condition for the workers, the posts footplates were directly installed on the soil of the slope by means of vertical steel pins diam. 26.5 mm (1 inch) steel 830 MPa (120 ksi), fully grouted and with a length of 5.0 m (16.4 ft), without any concrete plinth.

![Figure 4.2](image)

Figure 4.2 – On the left: two parallel lines of barriers. On the right: the primary 5,000 kJ rockfall barrier, 6 m (20 ft) high, impacted by the large multiple rockfall.
Seven months later, the unstable rock mass collapsed again. A large number of blocks hit the primary barrier, which resisted and maintained a residual height of 35%. Only one block was stopped by the second barrier placed just downslope the first. The total volume of rocks was estimated in more than 50 m³ (65 yd³), and the largest block had a volume of approx. 7 m³ (9.2 yd³) (fig. 4.1 – on the right).

The trajectories of the blocks of the second event were studied with a back analysis. It was suggested that the largest blocks mainly rolled down the slope, and have impacted the lower section of the net fence alignment. While smaller boulders (1 to 2 m³ (1.3 to 1.6 yd³) were bouncing along the slope and have impacted the barrier with a MEL of 5,000 kJ in various locations. It must be noted that only one of the rocks had reached the second barrier. Therefore the barrier had a great performance because the high residual height that allowed it to be effective for an event with multiple impacts. Furthermore, a relevant aspect of the behavior of the barrier was the performance of the footplates that allowed energy dissipation. Several footplates have sunk in the soil and have stripped the vertical anchor bars for approx. 0.2 m (8 inches). Also, several anchor bars have bent under lateral stress validating the theoretical calculations and the full scale test observations (fig. 4.2).

Considering the impacted area of the barrier and the actual falling velocity (estimated from previous analysis between 9 and 15 m/s (20 to 34 mi/hr) was 2 to 3 times greater than the design one (4.5 m/s (10 mi/hr)), and the total energy was at least 10,000 kJ exceeding the nominal capacity of the fence, which was severely damaged.

Figure 4.3 – Settlement of the footplates after the impact. The displacement was approx. 0.2 m (8 inches) vertically, and 0.05 m (2 inches) horizontally to downslope.
Figure 4.4 The vertical (y) and horizontal (x) settlements of the footplate represents a benefit in terms of energy dissipation by the stiffest zone of the barrier.

4.2 Colle Santa Lucia, Belluno (Italy): 2,000 kJ barrier

A 2,000 kJ barrier was impacted by 3 blocks. One of them, of approx. 4 m³ (6.7 yd³) (approx. 11,000 kg (24,200 lb)) crashed directly against one of the intermediate posts.

The impact velocity was estimated (by a simulation based on a back analysis) equal to 22 m/s (49 mi/hr). The other 2 blocks impacted against the barrier: the smallest one (~ 0.6 m³ (0.8 yd³)) impacted against the interception structure, while the largest one impacted on the upper part of the fence, by the upper longitudinal cable. The 3 boulders developed a total cumulative energy of 5,000+ kJ.

Even if one post was directly impacted, the structure was able to maintain a considerable residual height: 30-35% of the nominal one. The deformation of the mesh, after the 3 impacts, was approx. 6 m (20 ft).
The foundations of the post were composed of 4 steel bars. The base plates of the barrier were installed directly in contact with the ground, without any concrete base-plinths. After the impact, the footplate of the impacted post and the rod bars were lifted from the ground for approx. 0.3-0.4 m (1 to 1.3 ft). The steel bars bent due to the action of lateral shear stresses (fig. 4.5). It could be observed that, probably the designer under-designed the length of the rod bars. In any case, these foundations allowed dissipating the energy on the stiffest portion of the barrier.

4.3 Pattrich, Neder (Austria): 3,000 kJ barrier

Even if rockfall barriers are not designed to stop avalanches, this case presents a 3,000 kJ that was able to stop a snow-slide during its motion (fig. 4.6).

The rockfall barrier was installed in order to protect a little village, but during the winter an avalanche occurred and impacted against the barrier (Size 2 according to the Canadian Classification, McClung et al., 1993: mass approx. 100-150 tons, run 100-150 m (330 ft to 492 ft), pressure approx. 5-10 kPa (0.75 to 1.5 psi). The high velocity, energy and pressures involved in the snow-motion damaged a few components of the barrier. The avalanche impacted against 1.5 spans (approx. 15 m (49 ft)) of the fence. The lateral post (impacted as well) bent and tilted downslope and almost all the energy dissipater devices of the involved spans worked at their maximum capacity.
To conclude, the 3,000 kJ rockfall barrier stopped an avalanche composed by rock debris and big trees. In the impacted span the minimum residual height was evaluated higher than 50% of the nominal one, and the downslope deformation approx. 3.5-4.0 m (11.5 to 13 ft).

The posts were anchored with 2 steel rebars, 5 m (16 ft) long. A small concrete plinth 40x40x h=10-15 cm (16x16xh=4-6 inches) were built to give a flat base to the footplate of the barrier. After the impact no damages and no-pull-out were remarked on the bars. Only a few cracks were noticed on the concrete blocks, due to the high normal stress acting on the posts.
5 Conclusions

ETAG 027 testing guideline provides comparison between different rockfall barriers in terms of main performances, such as; energy capacity, maximum elongation, residual height, deformability, resistance, performance and the forces acting on the anchor post system of a deformable rockfall fence.

It is important to mention that anchor post systems are not considered part of the tested kit in ETAG 027 standard and their design is left to the designer choices. In any case, the designer has to consider the forces measured during the MEL test, even if the barriers are designed with the service energy limit state. Designers must therefore design the anchoring systems considering the local national standards and choosing the best solution taking into account the geotechnical properties of the soils and rock masses present in the site.

It can be underlined that, the whole flexibility of the barrier made of steel cables, mesh and post anchor systems that could be designed considering some settlements during a dynamic impact of falling boulders.

This settlement should be considered acceptable even if they are too large for building code anchor system. The rockfall flexible barriers have a very different behavior than a building considered that the whole structure is designed and manufactured to accept large deformations. This concept is very effective for barriers with upslope anchor cables that connect the top of the post to the upslope soil for the transfer of load from the mesh-cables to the soil.

In many cases the barrier needs to be built in remote area where heavy machineries cannot operate or where material transportation is very difficult creating a problem for the constructability of a large foundation for anchor post systems.

In some situations, the anchor post system becomes more expensive than the rest of the barrier by itself, resulting by the rejection of this type of solution due to its global cost. Furthermore when a severe impact strongly damages a barrier, for safety reasons, the whole structure, anchor post systems included, should be replaced. Then the basic question becomes whether strong anchor post systems should be designed as a massive foundation system is really necessary or a different strategy should be considered.

Furthermore the benefit of this novel design approach can provide a system that is able to dissipate even more energy itself therefore making the barrier better performing during the impact.

In these terms, instead of “foundation system” it would be better to talk about “anchor post system” that should be designed not as a massive foundation system able to adsorb all the impact force but as a light “anchor post system” that can accept deformations and displacements during the impact.
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GeoTechTools – Geotechnical Solutions for Transportation Infrastructure – Implementation

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ABSTRACT

A comprehensive, web-based information and selection guidance system compiling knowledge for more than fifty geoconstruction technologies applicable to transportation infrastructure has been developed. The system is available at www.GeoTechTools.org. The two primary components of this comprehensive toolbox are a Catalog of Technologies and a Technology Selection Assistance System. For each technology, the following documents can be accessed through the Catalog of Technologies: Technology Fact Sheet, Photographs, Case Histories, Design Guidance, Quality Control/Quality Assurance Guidance, Cost Information, Specifications, and Bibliography. Technology selection assistance is provided by either viewing technologies by classification or by use of an interactive selection system. The interactive selection assistance system aids the user in identifying a short-list of potential geoconstruction technologies for a particular project. Selection guidance is provided for the four broad application areas of: construction of embankments over unstable soils; construction or widening of embankments over stable or stabilized soils; geotechnical pavement components (i.e., base, subbase, and subgrade); and construction working platforms.

This comprehensive toolbox can (and should) be used during all phases of project delivery including planning, scoping, design and construction. GeoTechTools was developed to assist engineers, and others, involved in project development, scoping and/or the execution of highway projects, make more informed decisions on geotechnical issues to reduce risk and minimize construction surprises. The value of the web-based system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions on a readily accessible website. Additionally, it is a living system; users are urged to contribute to it and keep it current. The target audience for the system is primarily public agency transportation managers, geotechnical engineers, pavement engineers, and decision makers at local, state, and federal levels. Additionally, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia will also find the system very useful. This paper discusses the system and its application to project delivery and geotechnical solutions for transportation infrastructure. Goals in the implementation of GeoTechTools, from the Round 3 of the FHWA Implementation Assistance Program of SHRP 2 Research, are summarized.
INTRODUCTION

Although in existence for several decades, many geoconstruction technologies face both technical and non-technical obstacles preventing broader utilization in transportation infrastructure projects. The Strategic Highway Research Program 2, Project Number R02 (SHRP 2 R02) Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform investigated the state of practices of transportation project engineering, geotechnical engineering, and earthwork construction to identify and assess methods to advance the use of geoconstruction technologies. Such technologies are often underutilized in current U.S. practice, and they offer significant potential to achieve one or more of the SHRP 2 Renewal objectives, which are rapid renewal of transportation facilities, minimal disruption of traffic, and production of long-lived facilities. Project R02 encompasses a broad spectrum of materials, processes, and technologies within geotechnical engineering and geoconstruction that are applicable to one or more of the following “elements” of construction (as defined in the project scope): (1) new embankment and roadway construction over unstable soils; (2) roadway and embankment widening; and (3) stabilization of pavement working platforms.

The overall vision established for the project is “to make geotechnical solutions more accessible to public agencies in the United States for rapid renewal and improvement of the transportation infrastructure.” Phase 1 of the R02 project consisted of six tasks focused on identifying those geotechnical materials, systems, and technologies that best achieve the SHRP 2 Renewal strategic objectives for the three elements. Explicit in the tasks was the identification and evaluation of technical issues, project development/delivery methods, performance criteria and quality control and quality assurance (QC/QA) procedures, and non-technical issues that constrain utilization of geotechnical materials, systems and technologies. After identifying obstacles, and mitigation strategies to overcome these obstacles, the research team developed an approach to identify existing and innovative technologies to enhance geotechnical solutions for transportation infrastructure.

Phase 2 focused on 46 geotechnical materials, systems, and technologies that best achieve the SHRP 2 Renewal Strategic Objectives. These identified technologies are listed in Table 1. Phase 2 included development of a catalog of materials, processes, and systems for rapid renewal geoconstruction projects; and the evaluation and listing of design guidance and QC/QA procedures; methods for estimating costs; and sample specifications. Guidance on design, QC/QA, costs, and specifications for each technology were developed and integrated into a catalog and the selection support system.
A web-based information and guidance system was developed to provide a framework for applying the technologies. The system promotes more widespread use of ground improvement and geoconstruction technologies to achieve SHRP 2 Renewal objectives. This system provides the data necessary for determining the applicability of specific technologies to

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<tr>
<th>Aggregate Columns</th>
<th>Geotextile Encased Columns</th>
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<td>Beneficial Reuse of Waste Materials</td>
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<td>Bio-Treatment for Subgrade Stabilization</td>
<td>Hydraulic Fill + Vacuum Consolidation + Geocomposite Drains</td>
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<tr>
<td>Blasting Densification</td>
<td>Injected Lightweight Foam Fill</td>
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<tr>
<td>Bulk-Infill Grouting</td>
<td>Intelligent Compaction</td>
</tr>
<tr>
<td>Chemical Grouting/Injection Systems</td>
<td>Jet Grouting</td>
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<td>Chemical Stabilization of Subgrades and Bases</td>
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<td>Column-Supported Embankments</td>
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<td>Fiber Reinforcement in Pavement Systems</td>
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<td>Geosynthetic Reinforced Embankments</td>
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<td>Geosynthetics Reinforcement in Pavement Systems</td>
<td>Vacuum Preloading with and without PVDs</td>
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<tr>
<td>Geosynthetics Separation in Pavement Systems</td>
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</tr>
<tr>
<td>Geosynthetics in Pavement Drainage</td>
<td>Vibro-Concrete Columns</td>
</tr>
</tbody>
</table>
specific project constraints and conditions, and then guides the user to information needed to apply and engineer with a selected technology. The information, guidance and selection system will guide the user to a short-list of potential technologies. From these potential technologies, the user can access the catalog which includes information necessary for screening (i.e., depth limits, applicability to different soil types, acceptable groundwater conditions, applicability to different project types, ability to deal with project-specific constraints, general advantages/disadvantages, etc.), as well as guidance on design methodologies, QC/QA, costs, and specifications. Experienced engineers will benefit from the design, construction, and cost information provided in the catalog. Less experienced engineers, planners, etc. will also benefit from the technology selection assistance portion of the system to assess the feasibility of technologies to address project requirements/constraints.

Geoconstruction Technologies

Geoconstruction technologies provide for modification of site soils and/or construction of earth structures which are constructible and perform well under design and operational loading conditions. The growth in geoconstruction technologies, products, systems, and engineering tools has been tremendous, with a significant body of knowledge and large number of technologies available. Progress in this development has been chronicled by means of many conferences, workshops, papers and reports, too many to be cited herein. However, a few comprehensive references that describe many of the technologies included in this web-based system are: ASCE (1), ASCE (2), ASCE (3), Chu et al. (4), Elias et al. (5), Holtz (6), Mitchell (7), Munfakh and Wyllie (8), and Terashi and Juran (9). The information system described herein builds upon these earlier works and provides a comprehensive reference for each geoconstruction technology. The web-based system allows this information to be easily accessible and readily updated based upon user input.

A large number of geoconstruction technologies were initially identified at the start of system development. The number of technologies was winnowed to 46 based on their applicability to transportation related projects. The technologies included in the system are shown in Table 1 and come from the following areas: geosynthetics; ground improvement; grouting; slope stabilization including chemical and mechanical processes; alternative materials; and recycling. Excavation and replacement, and traditional compaction are two traditional technologies included as they are frequently utilized “base” technologies to which other technologies are often compared, and will be preferred technologies for many projects. The information system has deliberately avoided endorsing certain geoconstruction technologies over others. To the extent possible, naming specific manufacturers and contractors was intentionally avoided, though proprietary technologies are included.

THE WEB-BASED INFORMATION AND GUIDANCE SYSTEM

The vital information available through the web-based information and guidance system allows for selecting, applying, designing, cost estimating, specifying, and monitoring construction of the 46 geoconstruction technologies. As many of the technologies have subsystems, the system contains information on more than 50 different geoconstruction technologies. The web-based system does not replace the judgment of the engineer or other user.
The system does assist the user with selection and implementation of geoconstruction technologies for a specific project. The system is a comprehensive toolkit of geotechnical information to address all phases of decision making to allow transportation projects to be built faster, to be less expensive, and/or to last longer.

Development of the information and guidance system began in Fall 2009. A constant cycle of review, commenting, and revision was interwoven into development with every revision resulting in a more usable, intuitive system developed by engineers for engineers. Eight reviews of the system during development with input from potential users provided valuable comments and suggestions regarding the development of the system. The reviews included state and federal transportation agency personnel, as well as academia, practitioners, and specialty contractors. The overall concept of the information system is illustrated in Figure 1. The web-based system allows multiple users to access the technology information over the internet, and encourages users to contribute and update the information contained on the website. The system is available at www.GeoTechTools.org.

![Figure 1 – Information and guidance system overall concept.](image)

**Main Components of the System**

The two main components of the information system are Catalog of Technologies and Technology Selection System. The dissemination of information through the Catalog of Technologies provides the mechanism to facilitate technology transfer to everyday practice. One of the goals of the Technology Selection component is to refer the user to the appropriate Individual Technology Information page within the Catalog of Technologies. The other features of the website, such as the Project Background, Glossary, Frequently Asked Questions, Contributions to GeoTechTools, Links, and About This Website, support the primary components and usability of the website. The details of this development are summarized in the web-based system development report (10). The vital information available through the system allows for selecting, applying, designing, cost estimating, specifying, and monitoring
construction of the 50+ geoconstruction technologies. The web-based information and guidance system does not replace the judgment of the engineer or other user. The system assists the user with selection and implementation of geoconstruction technologies for a specific project.

**Catalog of Technologies**

The Catalog of Technologies webpage provides a listing of the 50+ geoconstruction technologies in the system. The name of each technology is a linked button that takes the user to a Technology Information webpage for that technology. The Technology Information page represents the technology transfer for each geoconstruction technology included in the system. Included on each Technology Information page is a series of ratings. Technology ratings were developed through the completion of a qualitative assessment to rate the technologies for each of the following: (i) Degree of Technology Establishment in the U.S., (ii) Potential Contribution to Rapid Renewal of Transportation Facilities, (iii) Potential Contribution to Minimal Disruption of Traffic, and (iv) Potential Contribution to Production of Long-Lived Facilities. The complete Catalog of Technologies can also be viewed with these ratings.

From the individual Technology Information page, the user can access the following documents which are generally provided as Portable Document Format (PDF) files: Technology Fact Sheet, Photographs, Case Histories, Design Guidance, Quality Control/Quality Assurance Guidance, Cost Information, Specifications, and Bibliography. These documents resulted from comprehensive analysis and evaluation of each technology to produce an in-depth overview that included advantages, potential disadvantages, applicable soil types, depth/height limits, groundwater conditions, material properties, project specific constraints, equipment needs, and environmental considerations. Comprehensive assessments on each technology were completed for design, QC/QA, and specifications to identify key engineering guidance. The downloadable documents available on the Technology Information pages result from the completion of these assessments and evaluations for each technology.

**Technology Selection System**

A Technology Selection System was developed to aid in identifying a short-list of potential geoconstruction technologies for a user defined set of project conditions. Technology selection contains both a listing of the technologies sorted by classification and a dynamic, Interactive Selection Tool. After the user identifies potential technologies, Technology Information pages can be accessed which includes information necessary for additional screening (i.e., depth limits, applicability to different soil types, acceptable groundwater conditions, applicability to different project types, ability to deal with project-specific constraints, general advantages/disadvantages, etc.).

The Interactive Selection Tool allows the user to assess technologies based on several applications. The uniqueness of the Interactive Selection Tool is the approach of assigning a geoconstruction technology on the basis of application. The first decision in the tool is to select one of the four listed applications, which are: Construction over Unstable Soils; Construction over Stable or Stabilized Soils; Geotechnical Pavement Components including Base, Subbase, and Subgrade; and Working Platforms, as shown in Figure 2. The Interactive Selection Tool is a knowledge based system. Special programming formed the logic and the knowledge is contained in a series of tables within the database. Each selection queries a database column and utilizes a nested if…then statement to sort the appropriate technologies. A significant benefit of the rule-based approach is the sharing of knowledge, especially when the knowledge is not the type of knowledge typically published in scholarly publications (Spring et al. (11)).

Figure 2 – Screenshot of Interactive Selection System webpage with four application areas.

After clicking on one of the four application areas shown in Figure 2, the user will encounter a page requesting additional information to narrow the list of candidate technologies.
for the particular application and project. The number of possible queries for additional information is quite large and is dependent upon the application selected. The requested input and order of queries to the user were selected after considering the effect of the requested information on the determination of the potential technologies list. The potential queries (in no particular order) generated during development of the system are:

- What type of project is being constructed?
- What is the size of the project being constructed?
- Are there any project constraints to be considered in selecting a possible technology?
- What is the soil type that needs to be improved?
- To what depth do the unstable soils extend?
- At what depth do the unstable soils start?
- Is there a “crust” or “rubble fill” at the ground surface?
- What is the depth to the water table?
- How does the water table fluctuate?
- What constraints exist? (i.e., utilities, material sources, existing adjacent structures, etc.)
- What is the desired outcome of the improvement? (i.e., decrease settlement, decrease construction time, increase bearing capacity, etc.)
- What technologies does the user already have experience with?

The questions used to narrow the technologies are dependent upon the application selected. Generally, three or four questions are used to develop a short-list; which can then be further defined with answering additional questions. The user can select which questions to answer.

Like most geotechnical analytical solutions, the results of the analysis must be measured against the opinion of an experienced geotechnical engineer practicing in the local area of the project. The Interactive Selection Tool does not replace the project Geotechnical Engineer. The Geotechnical Engineer’s “engineering judgment” should be the final selection process, which takes into consideration the following project specific items: construction cost, maintenance cost, design and quality control issues, performance and safety (pavement smoothness; hazards caused by maintenance operations; potential failures), inconvenience (a tangible factor, especially for heavily traveled roadways or long detours); environmental aspects, and aesthetic aspects (appearance of completed work with respect to its surroundings) (Holtz (6), Johnson (12)).

When one arrives at a list of candidate technologies, the Technology Information pages for each technology can be examined. For example, clicking on Prefabricated Vertical Drains and Fill Preloading link will bring up the screenshot shown in Figure 3. The documents listed can be accessed through hot-links on the website. Ratings are provided for each technology on the degree of technology establishment and a technology’s potential application to SHRP 2 objectives.

As shown in Figure 3 a number of information documents about a given technology are accessible from the system. The list of documents available is shown in Table 2, which also indicates the format for the document. These documents are hot-linked and can be opened from
The information documents are generally provided in Adobe pdf format. The Technology Fact Sheets are two-page, summary information sheets that provide basic information on the technology including basic function, general description, geologic applicability, construction methods, SHRP 2 applications, complementary technologies, alternate technologies, potential disadvantages, example successful applications, and key references. The Photos show pictorially the equipment or methods used in the technology and can be valuable to obtain a perspective on the technology. The Case Histories provide 2-page summaries of projects (which were preferably conducted in the U.S. by a state department of transportation (DOT)) and contain project
location, owner, a project summary, performance, and contact information. The Design and QC/QA Procedures documents provide a summary of recommended procedures for the technology. The recommended design and QC/QA procedures come from an assessment of the current state of the practice of each technology. In cases where a well-established procedure (e.g., a FHWA manual) exists, that procedure is recommended. In cases of technologies with multiple design procedures, the assessment led to a recommendation of a procedure(s) to use. For a few technologies, design and/or QC/QA procedures were established based on additional research conducted during the project. For most technologies, there are two Cost Estimation documents available. The first provides an explanation of the cost item specific to the technology, generally emanating from the pay methods contained in specifications. Available regional cost numbers, generally from DOT bid tabs or national data bases, are compiled for each technology. The second document for Cost Estimation consists of an Excel spreadsheet developed to aid in estimating costs for use of the technology. The second document could not be prepared for some technologies (e.g., emerging technologies) due to insufficient information. The spreadsheet can be modified by the user to estimate specific project cost based on either a preliminary or final design. Guide specifications are provided for each technology in Adobe pdf and Microsoft Word (if available). The final document available for each technology is a bibliography compiled during the research project.

<table>
<thead>
<tr>
<th>Table 2 – Documents Available Through the Information and Guidance System</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Available for Review or Download</strong></td>
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<tr>
<td>Technology Fact Sheet</td>
</tr>
<tr>
<td>Photos</td>
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<tr>
<td>Case Histories</td>
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<tr>
<td>Design Procedures</td>
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<td>QC/QA Procedures</td>
</tr>
<tr>
<td>Example and/or Guide Specifications</td>
</tr>
<tr>
<td>Cost Estimation (General, Spreadsheet)</td>
</tr>
<tr>
<td>Bibliography</td>
</tr>
</tbody>
</table>

**IMPLEMENTATION**

The SHRP 2 program has entered the implementation phase wherein selected research and development projects are being moved to implementation through a targeted Implementation Assistance Program (IAP) administered jointly by the Transportation Research Board (TRB), the Federal Highway Administration (FHWA), and the American Association of State Highway and Transportation Officials (AASHTO). The first two rounds of IAP occurred in 2012 and 2013. Projects for the third round of the IAP were selected in mid-2013, in which GeoTechTools (R02) was one of five projects moved to the implementation stage. A web search on GoSHRP2 will take one to the SHRP2 Solutions website where more information on the SHRP 2 implantation phase can be found.
The first step in the IAP program is the development of an implementation plan for the project. This began with an Implementation Planning Workshop (IPW) for GeoTechTools, held in Washington D.C. on November 21 and 22. At the workshop, a group of state department of transportation practitioners joined representatives from academia, TRB, FHWA, and AASHTO to assess the GeoTechTools products and to make recommendations on how it could be deployed nationally to ensure wide integration into the transportation community. The attendees considered the barriers and opportunities facing adoption of the product, and identified goals and tactics to include in the product's implementation plan.

Opportunities for Implementation Identified

The GeoTechTools website offers significant value by bringing together the key resources needed to address geotechnical issues in one place.

- Users of the website and related tools can enhance their expertise, increase both their own and their agency skill sets, and enable better design decisions earlier in the project delivery process.
- It was noted that chief executive officials of transportation agencies can promote the use of this web tool within their programs to show support for innovation.
- Many of the technologies included on the website have consistently proven to be effective solutions, but owners each have their own culture, design preferences, and experiences that affect the solutions they select. Engineers and contractor’s whose projects need innovative geotechnical solutions can explore, select, defend, and find resources to apply these techniques with this web-based tool.
- Potential users can be targeted immediately.
- The web tool retains geotechnical knowledge in the event of staff turnover.
- The tool can assist in risk-based decision making since it allows for scoping on the front end of projects—particularly helpful for designers—and, at the same time, it will be useful to those conceptualizing and managing the project.
- The tool can also be used in the middle of projects when geotechnical problems or complications arise that require immediate decisions be made.
- The tool directly benefits value engineering assessments by quickly filtering applicable solutions and allowing direct comparison of options and relative deployment costs.

Goals

The IPW participants reinforced the power and usefulness of GeoTechTools, both technically and at the decision-making and cost-estimating levels. It was generally recognized to be a relevant and accessible website, a living encyclopedia of reference materials, and a tool for allowing comparisons that would place time- and cost-saving information at the user’s fingertips.

The summarized goals are to:

- Maintain the website as a relevant, live, and regularly updated site.
• Develop a community of users who will generate market demand and contribute to *GeoTechTools*.
• Incorporate the use of *GeoTechTools* earlier in the design process; possibly within programming and planning, and surely within the early scoping stages of preliminary design activities.
• Continue to advance already available innovative geotechnical solutions that will save time, money, and resources.

The implementation plan seeks to use the resources available to accomplish these goals and bring measureable benefits to the transportation community. Table 3 lists the goals for *GeoTechTools*, and the desired outcomes for these goals.

<table>
<thead>
<tr>
<th>Goal #</th>
<th>Goal (in order of priority)</th>
<th>Near-Term/Long-Term</th>
<th>Desired Outcomes</th>
</tr>
</thead>
</table>
| 1      | Keep site relevant and updated. | Near-term | – Identify sustainable host with integrity  
– Add more case studies and additional vetted technologies as they emerge.  
– Provide transparency. |
– Broaden use in academia.  
– Create executive buy-in.  
– Make people’s jobs easier. |
| 3      | Add this tool to design process. | Near-term | – Encourage widespread use by DOTs (as a tool, not just an occasional view) for information and as a decision-making tool used in project planning and preliminary design.  
– Embed into existing FHWA training.  
– Increase skill sets within agencies—change the way geotechnical issues are handled and broaden the knowledge base.  
– Demonstrate value to potential users (planners versus technical users). |
## GeoTechTools Implementation Assistance

The Implementation Assistance Program is available to help State departments of transportation (DOTs), metropolitan planning organizations (MPOs), and other interested organizations deploy SHRP 2 Solutions. A range of opportunities is available to raise awareness of SHRP2 Solutions and to encourage early adoption of these products. GeoTechTools was part of Round 3 of the IAP, with the applications due February 14, 2014. Awards to 15 entities were announced on March 28, 2014. The implementation assistance generally takes the form of providing seminars on the use of GeoTechTools and technical assistance to the organizations on the application of GeoTechTools to specific projects. The ultimate goal of SHRP 2 implementation assistance is to demonstrate the integration and routine use of the GeoTechTools product within agency planning and project delivery practices.

## Summary

A knowledge base has been compiled for more than 50 geoconstruction technologies and a web-based information and selection guidance system has been developed to facilitate and organize this knowledge so that informed decisions can be made by users. The system assists users in the selection and implementation of suitable geoconstruction technologies for site specific conditions. Detailed information provides for optimization of design, cost estimating, specifying, constructing, and assuring quality to meet specific project requirements. Even with the wealth of information provided in the system, proper application of a geoconstruction technology requires extensive background knowledge of available ground treatment technologies and careful evaluation of several factors. These factors include understanding the functions of the method, utilization of several selection criteria, the use of appropriate design procedures, implementation of the right methods for quality control and quality assurance, and consideration of all relevant cost components and environmental factors. The value of the system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions on a readily accessible website. The technical information provided in the information system combined with the engineering judgment of the user will result in transportation projects that are built faster, cost less, and last longer.
References

Emergency Rockfall Mitigation BNSF Railway, Yakima Subdivision, Washington, December 25, 2012

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The BNSF railway passes through the deep Yakima River Canyon between Ellensburg and Yakima in Central Washington. Rock units along the river consist of a 1000 feet of interlayered Columbia basalt flows that are a source of historic rockfall.

On Christmas 2012 a rockslide was reported by the BNSF track inspector on the railway downstream of the Roza Dam. One of the basalt columns collapsed onto and blocked the railway requiring immediate cleanup. This is a blind curve and there are no early warning slide detector fences in this area for the trains.

Upon discovery BNSF immediately alerted Jacobs Associates to mobilize with a rockfall contractor to the site to evaluate the rockfall. It was established that the rockslide failed primarily by toppling of the basalt columns onto the railway bed.

The emergency project involved coordination of engineers, contractors and BNSF employees under challenging working conditions on a main line live-track in a remote location that was subject to heavy BNSF freight traffic. The contractor mechanically scaled loose rock and debris from the face using scale bars and pneumatic Jack Mats. BNSF provided a maintenance crew to muck out the rock debris from the ditch and roadbed with a backhoe and hyrail track crane. Over a four day period the team scaled and removed about 110 CY of unstable rock columns and loose rock that threatened the railway. The railway was reopened for routine traffic on January 8, 2013.
INTRODUCTION

The following is a short case history attendant to a rockfall emergency mitigation problem along the Yakima Subdivision of the BNSF railway in Washington State. The BNSF railway passes through the deep Yakima River Canyon between Ellensburg and Yakima in Central Washington (Figure 1). The railway is one of the mainline loops for freight in Washington and Oregon. Freight traffic is typically heavy with few windows to conduct work along the line.

The rock units along the river consist of a 1000 feet of interlayered Columbia basalt flows and are a source of historic rockfall. In addition, this stretch of river is a pristine wild trout fishery and popular with sports fishermen.

Figure 1- General location map of rockslide on the BNSF railway in the Yakima River canyon about 20 miles south of Ellensburg, WA.

On 25 December 2012 a rockslide was reported by the BNSF track inspector on the railway in the vicinity of MP 102.3 about ½ mile downstream of the Roza Dam (Figure 2). One of the basalt columns cropping out above the river collapsed onto and blocked the railway requiring immediate cleanup (Figure 3). The location of the rockfall occupies a blind curve on the railway especially for east bound trains. In addition, there are no early warning slide detector fences in this area for the trains as there are in other areas.
After the rockfall discovery, BNSF immediately alerted Jacobs Associates (JA) and asked them to visit the site to evaluate the rockfall problem and propose mitigation. JA immediately mobilized to the site with the BNSF Roadmaster, hi-railed to the site and assessed the rockfall problem. It was established that the rockslide failed primarily by toppling of the columnar basalt. The inspection revealed that additional basalt columns were unstable and threatened to collapse on the railway bed (Figure 4).

Figure 2 - Location of the Christmas 2012 rockslide on BNSF railway, just downstream from Roza Dam on the Yakima River, WA. Note rockslide is on a blind curve for east bound trains.
Figure 3 - Rockslide observed by the BNSF track inspector on 25 December 2012. Rock debris fouled and obstructed the rail-way (Photo by BNSF, 2012).
Figure 4 - Source area of rockfall. Note tension fracture behind unstable rock column next to the failure area.
PROBLEM ASSESSMENT

While emergency scaling operations were occurring, we evaluated the engineering geology and failure mechanisms of the rock slope. To assess the problem, the team conducted a series of horizontal and vertical scan lines to characterize the rock mass and collect information on the attitudes of the discontinuities. Vertical scan line mapping was accomplished by rappelling down the face of the slope. Mapping and assessing the rock face was extremely hazardous because of the active rockfall exacerbated by additional rockfall produced by the scaling crews.

Engineering Geology

The rock mass consists of strong columnar basalt from the Columbia Basalt flows. Because of the columnar structure, the joints are typically subvertical with subhorizontal tops mirroring the interflows. The flows are interspersed with subhorizontal flow contacts consisting of weathered, broken, vesicular basalt. In some areas, the rock slope overhangs the railroad.

The Christmas 2012 rockslide occurred from the central portion of the rockslope above the track (Figures 2 and 4). When BNSF and the author arrived on site, we observed an obvious scar from where the column collapsed. At that time we also observed another basalt column next to the recent scar with a large tension fracture opening between the column and main rock face. The tension fracture suggested that the rock column was kinematically unstable and subject to collapse similar to the first column of basalt. This was deemed an emergency problem which needed immediate attention. In addition, we observed loose rocks resting on top of the columns and at the brow of the slope. The primary failure mechanism for the columns on the rock slope appeared to be toppling resulting in individual rockfall or rockslides. There was also evidence that raveling of rock debris was occurring from the scree covered slope above the rock face as the sun warmed the canyon. Primary mechanisms which appear to have triggered the rockslide include precipitation and freeze-thaw wedging. During December and January the ambient temperatures at night dropped below freezing in the canyon and didn’t warm until the sun rays reached the rocks. Water runoff from snowmelt froze on the rock ledges and in the cracks at night and thawed during the day forming cyclic ice wedges behind the blocks.

In general, the rock mass exhibits a Rock Mass Rating (RMR) of about 65 which suggest the main rock mass is of good quality. Intact rock strength of the competent rock was very strong (about 75 MPa) based on hammer blows. Rock quality (RQD) by Palmström’s method ranged from approximately 35 to 55 suggesting poor to good quality rock depending on the location on the face. Spacing of the discontinuities near the flow contacts averaged about two inches because of the poorer broken quality of the rock mass. In contrast between the subvertical columns, spacing was as much as 40 inches.
Kinematic Analysis

During mapping, we collected approximately 75 attitudes of the discontinuities controlling the structure of the basalt rock mass. The discontinuities were uploaded to Dips, V6 by RocScience for analysis. As displayed on Figure 5, we observed at least four joint sets (Table 1). Joint Set 1 is a subhorizontal flow contact that dips south towards the Yakima River at about seven degrees. At least three subvertical joint sets form the sides of the columns that daylight the slope making the shape of the columns appear somewhat triangular.

Even though it was evident that toppling was the apparent primary failure mode, we verified the kinematic stability using Dips, V6 by RocScience. As was expected the rock mass was stable with respect to planar and wedge sliding. However, as displayed on the stereonets (Figures 5 and 6), the primary modes of failure are flexural, direct and oblique toppling. Flexural toppling occurs primarily around Joint Set 2 (refer to the southeast frontier of stereonet). All poles that plot in the pink critical zone of the stereonet represent a flexural toppling risk.

Direct and oblique toppling apparently occurs around the basal and subhorizontal Joint Set 1 (refer to the center of stereonet). Poles that plot in the critical pink zone on Figure 6 are subject to direct toppling and poles that plot in the yellow zones are subject to oblique toppling.

Figure 5 - General Stereonet of the rock mass structure, note four joint sets.
Table 1 – Discontinuity sets recorded during scan line mapping

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip and Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>07/181</td>
</tr>
<tr>
<td>2</td>
<td>87/329</td>
</tr>
<tr>
<td>3</td>
<td>89/065</td>
</tr>
<tr>
<td>4</td>
<td>82/032</td>
</tr>
</tbody>
</table>

Figure 6 - Kinematic analysis displaying flexural, direct and oblique toppling.

Catchment Zone

The present ditch in the catchment zone at the base of the cliff consisted of a V-ditch with a depth of about three feet and a width of about seven feet. Width of rock catchment zone is approximately 10 to 15 feet between the toe of the slope and the foreslope shoulder of the railway. The toe of the rock slope is approximately 80 feet from the Yakima River.

Rock debris from the rock fall event on 25 December 2012 landed in the ditch and on the rails blocking any rail vehicles. When the JA team scaled and dropped the second rock column, the column broke apart during toppling and landed primarily in the ditch. In general, with its
present configuration, the ditch captures about 75% of the rockfall (Figure 7). The ditch was deemed adequate because there is no room to increase the width of the ditch.

PROBLEM MITIGATION

Because of the severe unstable rock and rockfall problems, JA contracted with North Coast General Contractors (NCG) (since renamed to Rock Supremacy) to bring a team of rock scaling specialists in to scale and remove the loose rock from the face of the rock slope.

Safe access to the top of the rockslide and rockslope was difficult because of lack of adequate anchorage, icy conditions and loose scree covering the slope. Therefore, the team accessed the top of the ridge via a fisherman’s trail from a road turnout on SR-821. Anchor locations were selected on a rock outcrop along a ridge about 500 feet above the rock outcrop and rockslide and over 700 feet above the railroad and Yakima River. NCGS drilled rock anchors in competent basalt rock and employed the SPRAT anchorage system for their ropes. From this location the team rappelled down to the top of the rock outcrop above the railroad to begin scaling operations (Figure 7).

On day one, NCGC scaled about 30 CY of rock using scaling bars in preparation to access the kinematically unstable rock columns observed on 27 December 2012. During a period of four days, using the rail time windows that were allotted to the team from BNSF, the NCG crews scaled and removed about 110 CY of obvious loose and unstable rock from the rockslope that directly threatened the railway (Figure 7). Unstable rock columns and large blocks were removed using Mat Jacks (Figure 8). A Mat Jack is a pneumatic pillow with steel ribs that can be inserted in an open fissure. Once the mat is inserted, then the pillow is inflated with air from a remote compressor sufficient to expand the joint and push the rock column from the face (Figures 8 and 9).

![Figure 7- Rock scaling with bars (left photo), geologic mapping (right photo) note rock fall from above.](image)
The emergency work was conducted during “live track” conditions in four days. Because of the short track windows (four hours maximum), BNSF elected not to cover the track with mats or flood the track with ballast to protect the rails from the rock fall. Rather what rock fell on the track during scaling was immediately cleaned off and the rail was inspected for any damage. However, most of the rock and debris that was scaled from the face landed in the ditch next to the railway and did not damage the rail. BNSF supplied a crew aided with a rubber-tired back hoe and a Hyrail Track Crane to remove the rock from the railway and the catchment ditch.

Once the obvious unstable rock was removed by scaling methods, draping the rock slope with steel mesh was contemplated as an additional means to protect the track from rockfall. However, considering the history of rockfall in the canyon and cost of installation of the drape, it was decided to rely primarily on the catchment ditch. Furthermore, as before, the BNSF track inspector would provide early warning should a rock slide occur. In addition, the team evaluated the rockslope for spot rock bolts but there was no obvious need for rock bolts.

CONCLUSIONS

The following is a summary of our conclusions from this case history of emergency rockfall mitigation.

1. The rockslide failed primarily by toppling of the columnar basalt triggered by apparent precipitation and freeze-thaw wedging during December.
2. Emergency rock scaling removed about 110 CY of the obvious unstable rock columns and loose rock that threatened the railway. Scaling occurred during short work windows under “live track” conditions over a period of four days.
3. Rock scaling appeared to address the primary emergency rockfall issues.
4. The catchment ditch was adequate for most of the rockfall and caught approximately 75% of the debris. However, the ditch did not adequately catch the rock columns that collapsed and fell across the rail.
5. Draping was not considered economically viable because of multiple rockslide issues in the canyon. Rather the BNSF decided to rely on the early warning observations provided by the track inspector.
Figure 8 - Removal of basalt column using a pneumatic Mat Jack. The mat is inserted into the open tension fracture and then inflated with air until the block is pushed over.
Figure 9 - Series of photos demonstrates removal of an unstable rock block using a pneumatic Mat Jack.
Assessing the Capabilities and Limitation of Terrestrial LiDAR, Terrestrial Photogrammetry, and Airborne LiDAR for Mapping Differential Slope Changes

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ABSTRACT

Assessing transportation corridors exposed to the hazard of rockfalls and rockslides traditionally involves mapping and measuring physical characteristic of the visible cut or natural slope face. These assessment systems rely on the ability to visually identify the source zone, and determine the possibility of it releasing material and affecting the corridor. Conducting this form of traditional rockfall hazard analysis is extremely challenging in mountainous terrain where site accessibility is often limited, visibility is obstructed or minimal, and the terrain is extremely complex. Advancements in remote sensing technology and processing capabilities have enabled engineers to obtain valuable 3-Dimensional (3D) information in mountainous terrain that directly facilitates the understanding of the physical environment. Using 3D remote sensing data collected at different points in time enables the monitoring of differential slope change processes, which can be used to track rockfall frequency and magnitude. This information can be critical in assessing a transportation corridor for risk from natural threats. Various remote sensing technologies are capable of generating data suitable for differential change analysis, including terrestrial LiDAR, terrestrial photogrammetry, and airborne LiDAR. However, the advantages and limitations of these technologies and when they should optimally be deployed is not widely published or clearly defined. Between December 2012 and December 2013 the efficacy of three remote sensing technologies: terrestrial and aerial LiDAR, and gigapixel photogrammetry, were compared for detecting natural and anthropogenic changes at a location along the CN railway, in British Columbia, Canada. The results demonstrate a high degree of interoperability between the different technologies, the ability to map topographical change with all three technologies, and the limitations/weaknesses of each technology with respect to mapping change. These results will aid decision-making with respect to implementation of remote sensing technologies to monitor changes to rock slopes transportation corridors, which would lead to better hazard assessments.
INTRODUCTION

Transportation corridors in mountainous and other steep, rocky terrain are routinely subject to natural hazards, which threaten physical infrastructure, the safety of workers, travelers, and the delivery of shipped goods. Canadian railways and highways are exposed to such hazardous places in many locations across the country, with the highest concentration through the Rocky, Columbia, and Coast Mountains in western Canada. The Trans-Canada Highway #1 (TCH), CN and CP railways traverse the Coast Mountains by following the Thompson and Fraser River valleys. There, the transportation corridors are subject to multiple hazards including rockfalls, rockslides, landslides, debris flows, and snow avalanches (1-4).

For operational management of rockfall risks, CN Rail employs the ‘Rockfall Hazard Risk Assessment’ (RHRA) rating system (5-6) to determine the risk of derailment from cut and natural rock slopes, where a known source zone can be seen and evaluated from track level (7). Similar approaches are implemented along highway corridors with variant input parameters specific to assessing risk to vehicular traffic, opposed to rail traffic. One limitation of this approach arises for slopes with obscured source zones that are not visible from track-level. In areas of steep or rugged terrain, this is a common occurrence and poses significant challenges for geotechnical engineers. The ability to map topographical slope change and slope morphology with high accuracy and repeatability would allow these factors to be incorporated into a risk assessment program, in conjunction with traditional approaches.

Recent technological and computational advances in the field of remote sensing and 3-dimensional (3D) data interpretation have enabled geotechnical engineers to understand natural and anthropogenic slope processes at unprecedented resolution and accuracy, based on: remotely mapping geological structure and kinematic analysis (8-9); mapping topographical change (10-11), assessing rockslide susceptibility (12), rockslide characterization and monitoring (13), and 3D rockfall kinematics and runout modelling (14-15).

This paper focuses on the use of Terrestrial Laser Scanning (TLS), Airborne Laser Scanning (ALS), and terrestrial gigapixel photogrammetry for the purpose of mapping natural and anthropogenic changes to a natural rock slope at the site of a 1,765,000 ft³ (50 000 m³) rockslide which affected the CN Railway directly below and the TCH above. The objective was to use remote sensing technologies to assess slope changes caused by rockfall and the removal of unsafe rock blocks, in order to determine how each technology might be used optimally as part of a rock slope hazard assessment scheme, with the goal of overcoming the limitations of traditional approaches.

Test Site

ALS, TLS, and photogrammetric technologies were deployed at a site located approximately 90 km north of Hope, BC, on the east side of the Fraser River Valley (155 miles (250 km) northeast from Vancouver, BC) (Figure 1). The CN Railway is located near the toe of the slope, on a cut bench approximately 200’ (60 m) above the river, while the TCH is located on
a second bench approximately 720’ (220 m) above the railway. CN milepost (MP) 109 (Ashcroft Subdivision) is a short distance north (railway east) of the study area, which is centered on mile 109.4 of the Ashcroft Subdivision (Ashcroft MP 109.4). At this location the track was protected by a concrete rockshed, and slide detection fencing.

At 00:54 on November 24, 2012, a 1,765,000 ft$^3$ rockslide occurred at Mile 109.4 of the Ashcroft subdivision (Figure 2). A 330’ (100 m) long section of track was buried in over 50’ (15 m) of slide debris, and the concrete rockshed was completely destroyed. Prior to the large slide, on November 23, 2012, a smaller rockslide overtopped the rockshed and covered the adjacent track. Continued rockfalls were observed during initial clearing work. Further investigation in the crest area, downslope from the TCH revealed large open tension cracks in the rockmass, which outlined a larger potential failure. The railway was then closed to all traffic (16) as a risk avoidance measure, approximately eight hours prior to the large slide.

After the initial cleanup of the 1,765,000 ft$^3$ rockslide and rebuilding of the railway, an extensive site remediation, stabilization, and protection project was initiated by CN. The project involved blasting, scaling, and bolting loose rock blocks, installation of a large rockfall fence, and the planned construction of a concrete rockshed larger than the one destroyed in the 24 November rockslide. Alongside the anthropogenic changes made to the slope, continued natural failure of the rockmass was expected, and observed.

Figure 1: Site location map of the Ashcroft subdivision Mile 109.4 rockslide location along the CN Railway (Source Base Map: Imagery).
Figure 2: Mile 109.4 of the Ashcroft subdivision of CN Railway in southwestern British Columbia, Canada, 36 hours before a 1,765,000 ft$^2$ rockslide (a), 12 hours after the rockslide (b), and one year after the rockslide (c).
DATA COLLECTION

The site is uniquely situated in that it is easily observed from the opposite (west) bank of the Fraser River (viewpoint in Figure 2c). At a distance of 1300’-2000’ (400-600 m) away, the site is amenable to investigation by terrestrial-based 3D remote sensing technologies. Between December 2012 and December 2013 the slope was studied using TLS, ALS, and photogrammetry techniques, with the objective of mapping changes on the slope and assessing the optimal operating conditions for the different technologies. Terrestrial and airborne LiDAR and terrestrial photogrammetry technologies were deployed at the Ashcroft MP 109.4 site between December 2012 and December 2013 (Table 1). The deployment of various technologies at a single site facilitated a robust and direct assessment of strengths and limitations.

<table>
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<th>Point density (points/ft²)</th>
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</tr>
</tbody>
</table>

ALS data for this project was collected on 12 December 2012 and 25 November 2013 (Table 1). Raw ALS point clouds were processed by the suppliers for georeferencing and to classify the data into ground and non-ground points.

CN provided TLS data from 16 January 2013, which was collected by a contractor using an Optech Iliris 3D laser scanner. The TLS data from 2013 was collected by Queen’s University, using an Optech Iliris 3D Enhanced Range (ER) laser scanner. Figure 3 shows the two sites from which TLS data were collected (TLS-1 and TLS-2).

The terrestrial photogrammetry data was collected by Queen’s University on 28 November 2013 using a 36 megapixel Nikon D800 DSLR camera, equipped with a Nikkor 85 mm f/1.8G prime lens and a GPS peripheral. The images were captured at ISO 1250, f/5.0, at 1/125th of a second. One hundred and twenty six images were collected from three locations (Figure 3: PG-1, PG-2, and PG-3). The images were captured in 14-bit lossless compressed RAW-format in the field and processed to JPEG using Adobe Lightroom. The relatively high ISO and shutter speed were necessary given the low-light conditions present in the later afternoon of 28 November 2013; normally a lower ISO and a faster shutter speed would have been preferable. The data were collected handheld with approximately 50% vertical overlap and 30% horizontal overlap between individual images. The photographs were processed to a 3D point cloud using PhotoScan (17). An oblique view of the TLS, photogrammetry, and ALS models are presented in Figure 4.
Figure 3: TLS and photogrammetry setup locations with respect to the Ashcroft MP 109.4 slide.
Figure 4: Oblique view of the 3D polygonal mesh surface generated from the TLS point cloud (a); the photogrammetry point cloud (b); and the ALS point cloud (c).
DATA PROCESSING METHODOLOGY

Mapping change on natural slopes using ALS, TLS, or photogrammetry essentially examines the 3D surface topography at different points in time. The 4-Dimensional (tracking 3D changes with the 4th dimension being time) analysis examines surface information at ‘time 0’ with a direct spatial comparison at ‘time 1’. The limits of the comparison can be examined along set axes, for example defining change in the positive vertical direction is commonly used to map new construction/building in urban areas using ALS data (18). Mapping change on a natural slope is typically an unrestricted process, as the direction of change cannot be predicted and both zones of erosion and accumulation (positive and negative change) provide critical information. The necessary remote sensing data and processes required in order to assess a slope for change is (i) a high quality 3D baseline dataset and (ii) high quality 3D datasets collected at different points in time to align and compare to the baseline dataset for differential changes. If either of these criteria are not satisfied, the ability to assess the slope for change may not be possible.

3D Data Alignment

The baseline dataset in the project was comprised of TLS and ALS data was collected after the Ashcroft MP 109.4 rockslide failure in 2012. The ALS data, as anticipated, was occluded across all vertical outcropping features, and the TLS data was occluded across all horizontally outcropping features. In order to generate a full coverage 3D model of the Ashcroft MP 109.4 site, the TLS and ALS data was fused into a common spatial environment using the PolyWorks IMAlign module (19). The fusion of multiple datasets generated by different scanning technologies and different scanning platforms requires advanced data alignment workflows and quality control measures (9, 20).

In this project, the baseline ALS data was known to have the highest geospatial accuracy as it was collected with both differential GPS and strict ground control measures for reference and calibration. The baseline TLS point cloud was subsequently aligned to the baseline ALS point cloud using an iterative process of visually identifying common “tie-points”, identified in Figure 4, and mathematical iterative point best-fit algorithms (21). The fused dataset can be treated as a single point cloud, and is a near full coverage 3D surface model of the Ashcroft MP 109.4 site. Figure 5 shows the fused datasets: the green sections of the image are from the TLS data, and the grey sections are from the ALS data. The ability to create a fused model satisfies the high quality baseline data criterion for assessing the slope for differential change.
Figure 5: Fused ALS and TLS data from 2012. Note all vertical surfaces are represented in the TLS data while non-vertical surfaces are represented in the ALS data. The occluded vertical surfaces remaining in the dataset were not within the range of the TLS scanner.

Defining Change Detection Thresholds
Each of the comparison datasets (#3-5 in Table 1) are individually assessed relative to the baseline dataset for spatial change. The assessment is conducted by calculating the shortest distance between each node of the 3D mesh in the comparison dataset to the baseline dataset (22). In general, the 3D models collected at different times are aligned with the best possible fit, and the residuals or error in the alignment are treated as change or difference on the slope. Note that the residuals combine error in the alignment due to differences in the source data, due to alignment errors, as well as due to the true change in the slope topography.

Defining the limits of “actual change” versus alignment error and signal noise was determined though analysis of portions of the dataset where minimal change is thought to have occurred. A region of minimal change was visually mapped in all datasets using an iterative analysis of change detection calculations in combination with a visual examination of the terrain. Figure 6 illustrates a normalized histogram and probability density plots for a subsection of the Ashcroft MP 109.4 datasets where minimal topographical change is assumed to have occurred. The normal distribution is narrowest for the TLS data, then the photogrammetry data and greatest for the ALS data.

The limit of ‘actual change’ would be typically set at two times the standard deviation for a perfectly normal distribution. Given the skew in the distributions the 2.3rd and 97.7th percentiles for the respective distributions represents a conservative calculation of what can be considered “actual change” (22). Within these limits the difference between error and change cannot be accurately determined.
Figure 6: Normalized breakdown of a subsection of the 3D datasets with minimal observed natural and anthropogenic slope change mapped using ALS, TLS, and photogrammetry data collected in 2013 compared to the fused baseline data. The inset image identifies the zone in which the subsection of data was selected.

Volume change between two 3D surfaces is calculated by gridding the comparative dataset into domains, measuring the difference vector between the comparative surface and the baseline surface, and integrating each domain across the entire failure surface. The algorithm employed in PolyWorks enables the user to specify the gridding size, the orientation of the gridding, and whether holes in the dataset are filled.

RESULTS

The results of the comparison between ALS, TLS, and photogrammetry models collected in 2013 with the baseline data collected in December 2012 – January 2013 are presented as colour coded difference maps in Figure 7a-c. The figures display the negative change (loss of material) calculated on the surface of the slope. The minimum colour contour threshold for displaying ‘actual-change’ is unique for each of the three technologies used in the comparison. The values, as explained in Section 3.4, are -0.816’, -0.180’, and -0.512’ m for the ALS, TLS, and photogrammetry models respectively.
Figure 7: Negative differential change (material loss) analysis between fused baseline data (2012-12-12) compared to TLS data (2013-11-28) (a); photogrammetry data (2013-11-28) (b); and ALS data (2013-11-25) (c). Note the different limitations of minimal detectable change for each sub-image.
Inspection of these figures indicate that changes to the slope which occurred after the collection of the baseline data were equally mapped at a regional scale by analysis of the ALS, TLS, and photogrammetry models. All three comparisons illustrate the same spatial trends, in the changes detected, regions of minimal change, as well as in the approximate quantity (e.g. depth, volume) of change within each. This analysis, and visual examination of the results, provides confidence that each remote sensing technology is capable of assessing natural and anthropogenic change. There are indeed changes less than the calculated thresholds that have occurred on the slope that are omitted in this analysis due to lack of confidence in the alignment and noise in the datasets.

To quantitatively compare the performance of each technology in detecting changes to the slope, the calculated volume of discrete areas of changes using the different technologies was considered. Unfortunately no ‘ground truth’ is available to provide a basis for this comparison; the state-of-the-art method for calculating rockfall source volume is differential TLS analysis. Three zones, identified in Figure 7a, were analyzed for material loss. Changes calculated in zones ‘a’ and ‘c’ are the result of anthropogenic changes caused by the use of explosives and hand scaling to remove large unstable rock blocks above the railway line. In zone ‘b’ the changes result from the natural erosion of loose soil formerly resting on the steeply inclined bedrock.

The results of the discrete volume change comparison are presented in Table 2. There is a strong agreement between the TLS and photogrammetry calculated volumes for all three zones of change, with the differences ranging from 0% (Zone a) to a maximum difference of 1.6% (Zone b). There is a reasonable agreement between the ALS and TLS/photogrammetry, with a maximum difference of 10% for Zone a, 5.8% for zone b, and 11% for Zone c.

### Table 2: Volumetric analysis of discrete topological changes between fused baseline data (2012-12-12) compared to TLS, ALS, and photogrammetry data. Zones of loss are identified in Figure 7a.

<table>
<thead>
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<th>Comparative dataset</th>
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<th>Zone of loss ‘c’</th>
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<td>108027 ft³</td>
<td>40859 ft³</td>
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<tr>
<td>Photogrammetry (2013-11-28)</td>
<td>12783 ft³</td>
<td>109793 ft³</td>
<td>40435 ft³</td>
</tr>
<tr>
<td>ALS (2013-11-25)</td>
<td>14231 ft³</td>
<td>114772 ft³</td>
<td>45873 ft³</td>
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</tbody>
</table>

**CONCLUSION**

The primary object of this research project is to determine the applicability of Terrestrial LiDAR Scanning, Airborne LiDAR Scanning, and gigapixel photogrammetry for mapping natural and anthropogenic changes on a natural slope over time. The results presented in Figure 7 and Table 2 illustrates the capability of all three technologies to map both natural and anthropogenic slope changes. The clarity of the results differs between the technologies based on the spatial accuracy and resolution of the data, but all technologies exhibit the ability to assess change within their spatial resolution at this site.

The results further demonstrate the ability to fuse data sources and perform spatial analyses of data of different technological origin. The interpretation of the results generated from
the datasets individually lead to the same conclusion regarding the zones of activity, magnitude of activity, and volume of specific failures. The minor differences in volume do not change the interpretation of the terrain conditions, and would not affect decisions regarding risk management if the primary input was the rate of slope activity.

The spatial resolution and accuracy at which the data from the respective technologies can be used to detect change is variable. The ALS data collected in 2013 has a spatial resolution of 3 pts/ft², equivalent to 20 cm point spacing, compared to >40 pts/ft² for the TLS data and >20 pts/ft² for the photogrammetry data. The lower spatial resolution of the ALS data does not facilitate the mapping of small topographical features, resulting in less accurate assessment of detailed topographical changes. The decision as to what remote sensing technology should be deployed for a given task is a technical, logistical, and financial decision. This paper demonstrates the spatial limitations that can be expected for the deployment of a given remote sensing technology.

All three technologies are capable of mapping natural and anthropogenic slope change with variable confidence and resolution. TLS data offers the most accurate and spatially dense mapping solution while ALS data can be deployed over spatially larger regions. Photogrammetry does not offer the resolution or accuracy of TLS, but it requires minimal, affordable equipment and can produce true colour datasets. This paper is based on a journal paper submitted to the Canadian Geotechnical Journal, the journal paper contains a more rigorous discussion of site limitations, data analysis, data processing methodologies, and additional results (23).
REFERENCES:


Design & Performance of Flexible Debris Flow Barriers in a Narrow Canyon

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-John Kalejta

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ABSTRACT

Design & Performance of Flexible Debris Flow Barriers in a Narrow Canyon

Prepared for Submission to the 65th Highway Geology Symposium
Laramie, Wyoming

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Implementation of engineered, flexible debris flow barriers to protect roadways and other infrastructure is increasing. Simultaneously, design methodology has advanced rapidly. An example of a state-of-the-art installation was constructed in Colorado Springs, Colorado.

The Waldo Canyon Fire (July 2012) resulted in significant debris flow risk to watersheds affected by the burn area. These watersheds are characterized by rugged topography, historically intense precipitation events, and narrow outlets. Public and private entities supported the construction of debris flow barriers to protect infrastructure and lives within the floodplain of Camp Creek. The USGS estimated a probability of 45% for a debris flow volume greater than 100,000 m³ for the ten-year precipitation event.

A design methodology based on field and laboratory testing and computer modeling was utilized. A thorough field assessment was performed to determine input parameters for barrier design using proprietary software. Two sites were selected based on channel morphology and storage capacity.

Limited access required helicopter transport of construction materials. Alluvial material presented drilling challenges and high anchor design loads required significant installation depths. Load-testing sacrificial anchors to determine actual bond strengths allowed anchor depths to be reduced, and considerable time and money savings were realized.

Daily construction oversight by the engineer allowed rapid adjustments to design based on field conditions, permitting the project to be finished within the allotted timeframe.

The barriers were successfully completed and contained several debris flow events. Extreme precipitation in the fall of 2013 led to some design modifications to the installations.
INTRODUCTION

Debris flows can be described as a natural hazard that behaves like a combination of a flood, rockslide and landslide. They are, in essence, a high-energy mixture of sediment and water. In much of the western United States, seasonally severe rainfall and wildfire events interact to trigger intense debris flows, which can threaten lives and infrastructure. Burned areas in rugged topography adjacent to population centers are especially of concern to officials, planners, engineers, and geologists. Many communities face the reality of wildfire-induced debris flows by mitigating the hazard through engineered solutions. Methods that are commonly used include detention basins as well as rigid and flexible steel debris flow barriers. Both approaches seek to retain the volume of debris to prevent catastrophic flooding and damage to downstream developments.

Advantages of detention ponds include total detainment of debris and floodwaters, but are typically quite large and require a significant construction effort. This type of mitigation is common in southern California, especially on large watersheds originating in the San Bernardino Mountains. Within smaller watersheds, flexible steel debris flow barriers can be constructed quickly and have the advantage of being installed in almost any location within the drainage. They also can be installed in series along a stream channel to retain larger quantities of debris where topography is steep or where the channel is incised and does not offer adequate accommodation space for a detention basin or a single large flexible barrier. Of particular interest to the transportation designer, flexible debris flow barriers can be constructed directly adjacent to roadways in small drainages to prevent hazards to motorists, Figure 1.

Figure 1. Flexible debris flow barrier installed on State Highway 1, near Big Sur, California.
Although not necessarily inexpensive, flexible barriers offer unique options to the designer faced with difficult constraints and extreme hazards. The ability to quickly construct flexible debris flow barriers in response to a debris flow hazard is a key advantage to the community reacting to natural disasters. A key function of flexible barriers is that water is allowed to pass through them, while preventing the debris from jamming under bridges and inside culverts, causing flooding.

This paper presents a case study in Colorado Springs, Colorado where three Geobrugg debris flow barriers were installed in a narrow canyon draining a rugged watershed in the Rocky Mountain Front Range. Upon exiting the canyon, the watercourse runs through a densely populated area of Colorado Springs, filled with parks, schools, and churches.

BACKGROUND

The Waldo Canyon Fire, Figure 2, burned 18,247 acres of rugged topography between its ignition on June 23, 2012 and its containment on July 10, 2012 (InciWeb, 2012). Lands burned include portions of the Pike-San Isabel National Forest and private ownership surrounding
Colorado Springs and are shown in red, Figure 3. The upper Camp Creek watershed covers approximately 11 square miles of land belonging to the United States Forest Service, a private resort and citizens of Colorado Springs. The creek flows through the Garden of the Gods Park after exiting its upper watershed through a narrow canyon cut in Pikes Peak granite. Due to the homogenous rock type, debris from this watershed was expected to be granular, decomposed granite and organic matter not consumed in the fire.

Subsequent to the Waldo Canyon Fire, the United States Forest Service Burned Area Emergency Response (BAER) team performed a rapid assessment of the USFS lands within the Camp Creek watershed. The BAER report analyzed the burn severity of the watershed and offered predictions of hydrological change likely to be experienced within the basin. Additionally, the United States Geological Survey produced a report entitled “Probability and Volume of Potential Postwildfire Debris Flows in the 2012 Waldo Canyon Burn Area near Colorado Springs, Colorado” which estimated that a 10-year storm event within the Camp Creek watershed could produce a total volume of debris greater than 100,000 m³ (Verdin, et al, 2012). The National Weather Service in Pueblo, Colorado, in conjunction with the BAER team, set a rainfall rate of 0.50-in/hr as the threshold for flash flood warnings in areas impacted by the Waldo Canyon Fire (WFO Pueblo, 2012).
Facing the predictions for catastrophic debris flows, facilities personnel at the private conference center situated at the outlet of Queen’s Canyon and City of Colorado Springs officials quickly teamed together to avert potential disaster. The conference center desired to have protection for its guests and for irreplaceable iconic structures on the property. The City of Colorado Springs needed to address flooding within highly developed communities adjacent to Camp Creek as well as within the Garden of the Gods Park. The stakeholders required an assessment of the debris flow hazard and to design a mitigation solution to protect lives and infrastructure likely to be impacted by post fire precipitation events.

DEBRIS FLOW INVESTIGATION

A field investigation was begun in November 2012 to assess the debris flow hazard and to collect data for the design of flexible debris flow barriers within Queen’s Canyon. Using the BAER and USGS reports as references for the hydrological response and debris likely to be produced by the upper watershed, seven potential sites were identified where debris flow barriers could be constructed to provide protection to downstream communities and vital infrastructure.

The rugged nature of the terrain dictated that the barriers would have to be built in deeply incised canyons with limited access by conventional construction equipment. Another constraint faced by the designer was that the debris must be fully contained on private property and not allowed to back up onto adjacent USFS lands. The blue polygon in Figure 3 indicates the extent of private property within the upper Camp Creek watershed. The remainder of the upper watershed is within the Pike-San Isabel National Forest.

To fully contain the 100,000 m³ volume of debris predicted by the USGS would have required barriers at all seven sites. However, it was noted that up to 45% of debris flow volumes are comprised of water (Lips, 1993). This knowledge allowed the designers to identify two barrier sites, depicted with yellow markers in Figure 3, which could retain the remaining solid debris volume for the design 10-year precipitation event. Additionally, choosing one site at the outlet of the canyon, accessible by excavating equipment, provided the opportunity to clean out the retention area between debris flow events. Given the short time frame between the investigation and the next year’s rainy season, swiftly constructing debris flow barriers at the two locations was critical to protecting the at-risk communities.

DESIGN METHODOLOGY

Debris flow barriers were designed using the web-based dimensioning software DEBFLOW. Using data collected at the project site as well as from literature references, geological characteristics of debris flows expected at the site were analyzed to quantify the dynamic and static loading of the debris flow barriers. Thorough data collection regarding channel morphology during the initial investigation allowed the designers to recommend debris flow barriers manufactured by Geobrugg Protection Systems for installation at the site. Geobrugg is a Swiss company that has done substantial research regarding debris flow mitigation (Geobrugg,
Thus, the debris flow barriers are the result of extensive field and laboratory testing as well as finite element modeling.

Compared to rockfall barriers, debris flow barriers must have robust components to withstand the static and dynamic forces placed upon them. Generally, they are comprised of all steel components including: wire rope anchors and support ropes, ring nets connected by shackles, braking elements, abrasion protection, and, depending on channel width, support posts.

Geobrugg offers two debris flow barrier models, VX for installation in small channels and UX for larger channels. The UX barriers feature posts which help support the height of the barrier over wider channel distances. The VX model does not require posts. Figure 4 shows elevation and plan views of Geobrugg UX and VX debris flow barriers designed to be installed in a stacked configuration at the lower site, as well as earthen abutments, which the anchors run through and into bedrock. Due to channel morphology, a single UX barrier system was designed for the upper site.

Figure 4. UX and VX debris flow barrier schematic diagram depicting elevation and plan views at lower barrier site on Camp Creek showing lateral anchors extending through abutments and into bedrock.
CONSTRUCTION

Plans and specifications were turned around quickly, and an experienced and qualified contractor was selected to perform the installation of the debris flow barriers in January 2013. The completion deadline was set as March 30, 2013. Construction began in early February 2013. Engineer’s personnel were onsite for the duration of the project to provide quality assurance for the client and rapid technical support to the contractor. The construction sequence involved drilling holes for the wire rope anchors, grouting the anchors in place, testing the anchors to verify pull out strength, and assembly of the barrier components. The two sites chosen presented individual challenges for the contractor. The upper site was only accessible by foot through the narrow canyon and required helicopter transport of all equipment and materials necessary to build the barrier, Figure 5.

![Figure 5. Helicopter transport of construction materials to upper barrier site. Downtown Colorado Springs is seen in the background. Garden of the Gods Park is to the right of the frame.](image)

The lower site was easily accessible, but presented drilling challenges in saturated alluvial material. Two debris flow barriers were designed to be stacked vertically while acting independently, Figure 6. A smaller barrier was constructed in the incised stream channel while a larger barrier was built above it on top of the stream terrace. Additionally, to retain the massive volume of debris expected at the two sites, clean out of the lower barrier site was desired. To make clean out feasible with a long reach excavator, abutments were necessary to provide access behind the barrier in the narrow canyon.
Otherwise, expensive finite element modeling by the manufacturer would have been necessary to ensure acceptable performance of a barrier large enough to retain the required volume of debris. Due to the extreme forces on the UX debris flow barrier lateral anchors, up to 78,700-lbf on each anchor, it was necessary to drill into granite bedrock and grout them in place prior to constructing the abutments. This required very long anchor lengths through the abutments that had to be precisely placed while the contractor added the lifts and compacted the abutments. This unique solution proved to be effective at retaining a larger amount of debris than could otherwise be retained in a commercially offered standard-size barrier. The VX barrier in the channel required that anchorages be drilled into alluvial material that was saturated by underflow. Although the stream had little surface water in its channel, a significant volume of water was observed to move through the subsurface in drilled holes.

Using air rotary drilling methods, the holes often became enlarged and collapsed due to the saturated material. Dry grout was used to stiffen the walls of the drill holes with much success. The anchorage depths required in the alluvial materials were in excess of fifteen feet and were drilled perpendicular to the ground surface. The engineer’s representative observed drilling operations and was able to shorten these depths upon conferring with the engineer when bedrock or large boulders were encountered, providing cost savings and ease of construction.
Drilling for anchorages into the granite bedrock was comparatively easy, as the engineer’s representative worked with the contractor to align the barrier with competent rock outcrops and avoid fractured rock as much as possible. Drilling in granite was performed with pneumatic hand drills and air rotary drill rigs at depths up to 8-ft, Figures 7, 8.

Once drilling was completed, the contractor installed tremie tubes and PVC centralizers onto each anchor and inserted them into the drill holes. Pressure grouting was accomplished using a hydraulic grout pump and the contractor had laboratory testing performed on the non-shrink grout mix, as required by the engineer. After a curing period of 72-hrs, the contractor pull tested anchors selected by the engineer’s representative.

Anchors in the alluvial material were chosen for testing based on the weakest material encountered while a sacrificial anchor was tested in the granite bedrock. The sacrificial anchor was used because the lengths of anchors protruding from the drill holes were a minimum of 15-ft due to the abutment construction and it was not feasible to set up pull test cribbing at that scale. Figure 9 shows a pull test assembly set up on the sacrificial anchor drilled into granite bedrock. The lengths of production anchors can be seen in the background.
PERFORMANCE

The debris flow barriers were completed by the contractor, on time and before the March 30, 2012 deadline. A light snowpack in the Colorado Rocky Front Range resulted in little spring runoff within the Camp Creek watershed. After the barrier installations were completed, the watershed did not experience any precipitation events with enough intensity to trigger debris flows until July 1, 2013, when the first debris flow volumes were retained by the barrier systems at the lower site. An intense thunderstorm cell traveled northward through the upper watershed, with enough precipitation intensity to initiate debris flows that were full of burned forest materials and ash, along with rocky debris varying in size from sand to cobbles. Figure 10 shows the debris retained at the lower barrier site. The volume of debris was wholly contained by the lower VX barrier system at the lower site, an estimated 1,000 yd$^3$ of material.

The UX barrier at the upper site did not retain any measureable debris during this event, meaning that the majority of the debris retained at the lower barrier site originated in burned areas between the two barrier locations.
Torrential rains in early September 2013, Figure 11, resulted in widespread flooding throughout the Colorado Rocky Mountain Front Range. Within the Camp Creek watershed, precipitation exceeded 8-inches over a three day period, but was not intense enough to produce debris flows or other mass movements. Instead, the Camp Creek stream channel experienced extensive scouring by the flashy discharge produced within the post-fire watershed. The upper UX barrier partially filled with organic debris and rocks, Figure 12, but did not withstand any damage.

The stream channel at the lower site exceeded its bank full capacity at the crest of discharge. Although the barrier systems performed as designed by catching debris, the relative amount of water compared to debris was phenomenal. The stream channel adjacent to the barriers eroded enough that the lateral anchors of the lower VX barrier system experienced sagging, although no anchors failed. This created an opening between the two independent barriers and allowed some debris to flush through. Figure 13 shows the lower barrier site during flooding.
Figure 11. USGS streamflow gauge in Camp Creek showing daily precipitation for September 2013.

Figure 12. UX barrier at upper site partially filled with debris after September 2013 flooding.
Figure 13. Lower barrier site during flooding, September 2013.
After a site visit by the engineer, the contractor was retained to perform repair work at the lower barrier site. The barrier systems sustained minimal damage, but the channel scoured beneath the abutments, undermining the gabion armoring. Additionally, the foundations for the UX barrier system posts had been exposed due to erosion. The contractor dismantled the lower VX barrier from its anchors, and formed up concrete reinforcements to repair the erosion damage. The stream channel was armored by locking large boulders in place with an excavator. The entire stream channel was encased in shotcrete for a short distance upstream and downstream of the barrier alignment to provide protection against scour during rainfall events of high precipitation, but low intensity.

Although the abutments had been faced with gabions to protect against erosion, the massive discharge experienced during September 2013 was much greater than anticipated. The additional shotcrete armoring placed during the repair provides a greater degree of protection against future scouring of the barrier.

**Summary and Conclusions**

A planned, phased approach enabled the project to be completed quickly and effectively. The initial investigation by a team of engineers and geologists was critical to the design and production of construction plans for a successful project. The DEBFLOW software is intuitive and provides effective dimensioning parameters for the designer. Working in conjunction with the client, the design team, the manufacturer, and the contractor were able to usher the project to completion under a tight schedule. Daily construction oversight by the engineer’s representative allowed for cost effective quality control through anchor testing and inspection of the installed systems.

Extreme precipitation events in September 2013 resulted in some design modifications to the barriers. Although the designers had protected the abutments with gabions, scour in the channel was unexpected. In response, adding a coating of shotcrete atop the boulders arranged in the streambed will account for less intense rainfall events that produce scour rather than debris flows. The contractor who installed the debris flow barriers was familiar with the site and was able to effectively repair the systems and construct the armoring to the satisfaction of the client.
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Using New and Old Technologies for an Economical Approach to Rock Stabilization on TN I-75

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ABSTRACT

The Highway & Mining industries have historically used benches as an effective technique for rockfall mitigation as described in The Transportation Research Board’s “Rockfall: Characterization and Control” handbook. This paper describes how the latest advances in rockfall technology can be coupled with the older technique of benching to effectively and economically mitigate a rock slope along I-75 in Tennessee; north of Knoxville and close to the Tennessee / Kentucky border.

By rehabilitating the existing benches, scaling off the loose rock in critical areas, using pinned mesh, a mesh drape, and rock bolts, the Tennessee Department of Transportation (TNDOT) was successful in mitigating this near vertical rock slope that is over 200 feet high and almost two miles long.

This paper will focus on the approach and design parameters utilized to develop this innovative and economical approach. In addition, this paper will explain the thought process involved, tools utilized, construction sequence, and traffic control steps needed to complete the project.
INTRODUCTION

As a response to a maintenance request on June 28, 2010, the Tennessee Department of Transportation (TNDOT) Geotechnical Section investigated an apparent accident caused by a rockfall on I-75 northbound between exits 156 and 159 in Campbell County. During this investigation, TNDOT discovered that the rockfall event occurred in an area that is subject to frequent rockfalls. Evidence was found that many of the past rockfall events resulted in rocks of various sizes landing near or on the roadway (Figure 1). After discussing the findings with the TNDOT Regional Director and Regional Maintenance Engineer there was a joint consensus that this segment of I-75 needed to be studied for a comprehensive rockfall mitigation solution through TNDOT’s Rockfall Mitigation Program. The Geotechnical Office and the Design Office of Region 1 began the process of developing mitigation plans for the northbound segment of I-75 between L.M. 27.23 and L.M. 29.15. While the entire project length was 1.92 miles, the critical sections of concern for rockfall were concentrated in less than a mile long segment, which also contained one large outcropping near the highway.

The slopes in this section ranged in height from 150 to 250 feet in vertical height with an existing 20-foot wide bench located approximately 50 feet above the toe of the slope. Above the bench the rock slope rose vertically until a mid-point where the slope angle changed slightly to approximately 75 degrees. This mid-point inflection was referred to in the project as the breaking point. The entire rock mass is comprised of a relatively hard limestone that is blocky in nature. The discontinuities of the rock mass are susceptible to weathering and erosion by forces such as increased pore water pressure, freeze-thaw, and root wedging that result in rockfalls ranging from 6 inches to over 4 feet in diameter. The catchment area between the toe of the slope and the roadway varied greatly and included a rigid 50-inch tall jersey-type barrier at the shoulder of the interstate highway.

Figure 1 - Station 180 before repair

Figure 2 - Bench condition (lower bench) before grading
To assist with the development of mitigation plans, LIDAR (Light Detection, And Ranging) and aerial photography along with the past rockfall history were used to help develop CRSP (Colorado Rockfall Simulation Program) rockfall models. These models were used to determine the potential trajectories, kinetic energies, and bounce heights to be expected at different locations within the rockfall area.

DESIGN

TNDOT speculated that for the initial design of the slope, rockfall would be controlled by the existence of the relatively wide mid-slope bench (Figures 2, 3 & 4) and the barrier at the shoulder of the road. After construction of the rock slope, the bench may have been effective, but during inspection of the present slope condition it was discovered that these benches had become completely full of rockfall debris and other colluvial material over the years due to the lack of maintenance. In fact, the benches were not only full of material but formed a shape on the benches that created a significant launch feature for rockfalls. Falling rocks from above the bench that initially would have been arrested on the bench were now rolling along the debris and being launched towards the roadway.

The barrier mentioned above had been added after the initial design of the slope; none of the rock falls to-date had launched over the rigid barrier. Rocks only landed in the roadway in locations with no rigid barrier. The ineffectiveness of the bench was exacerbated by the limited catchment area in certain zones of the slope where the bench was practically the only means of rockfall containment.
The goal from the designer’s perspective was to develop a cost effective rockfall mitigation design that would incorporate the existing benches into the design parameters and reduce the energies of the falling blocks where the rock slope had limited catchment areas to the interstate.

Strategies of mitigating rockfall for the critical areas of this slope included improving the condition of the slope, controlling the energies of rockfall, and using barriers to prevent rockfall from entering the roadway. To improve the slope condition, techniques such as scaling and trim blasting were recommended to remove rocks that were prone to immediate release. To control future rockfall from above the bench area, the bench condition was improved [Figure 5] (material removed and bench reshaped) and rockfall drapes were installed. To further prevent rockfalls from entering the roadway, a Geobrugg flexible barrier was also considered for installation throughout this zone.

![Figure 5 - Bench Grading](image)

The final design incorporated the use of pinned mesh systems using Geobrugg’s SPIDER (TN DOT Type IV) mesh where the geotechnical analysis indicated the possibility of large-scale failures in a limited area due to the heavily fragmented/broken rock combined with a narrow bench and catchment area. Outside of this pinned area and due to the availability of wider bench and catchment area, TNDOT opted to use Geobrugg’s Rolled Cable Net (RCN) (TN DOT Type III) mesh as a drape in areas outside of the pinned mesh.

**INSTALLATION**

At the beginning of the project and before the scaling was initiated, GeoStabilization International (GSI) team leaders noted that large slabs of rock perched on the slope’s crest in the pinned mesh area needed to be removed to ensure the safety of their men working underneath.
During scaling of the critical areas, TNDOT could not practically allow closure of the interstate; consequently, GSI and TNDOT used 25 minute rolling roadblocks to ensure the safety of the workers and traveling public. Work had to be performed in a manner that any scaling process would be safely performed within these time constraints. During the operations, it became apparent that removal of the one of the larger blocks of rock could not be safely completed within the allotted time periods by scaling alone. The only logical solution was to blast out the material.

Kesco, GSI’s blasting consultant, developed a blasting plan around the site’s time and geometry constraints. The unstable block of rock was then drilled and the explosives were loaded. Blasting was complicated due to the open voids within the rock mass. Kesco and GSI closely scrutinized the drill logs in order to finalize the blast plan and use the minimum amount of explosives to minimize the final impact to the rock slope. This was accomplished by “stepping” the explosives, or the placement of stemming between the charges to localize the energy of each explosive charge. Kesco utilized remote detonators that allowed the blaster to verify that all of the contacts and wiring were intact moments before the blast. The blast was successful with only a small amount of debris entering the roadway.

The manual scaling process began as GSI rockfall teams scoured the slope face and removed over 5,000 yd\(^2\) of loose material to ensure the slope was made safe to work; then the scaling was begun in earnest. Once the scaling was completed, the original benches were cleaned and remediated. Approximately 90,000 yd\(^3\) to 110,000 yd\(^3\) were removed from the bench areas during this process.

In the pinned mesh area, GSI used Grade 75 #8 bars as rock dowels. The anchors were drilled and installed (Figure 6) into the slope at the locations specified in the project plans. Due to the potential wedge failures that existed in this fragmented rock mass, GSI recommended that the dowels be installed to a depth ranging from 10 to 20 feet. During the drilling process, it was apparent in some specific locations that 30 foot-long dowels would be required as the fragmentation went further back than originally anticipated.

Figure 6 - Installation of Williams Form Dowels
During the installation process, GSI installed dowels along a perimeter 20 feet beyond the fragmented area. TYPE IV Mesh (Figure 7) was then installed via helicopter (Figure 8) to hold the fractured rock to the slope. The mesh was secured tightly against the slope using spike plates and then tensioned using 256 ft-lbs of torque on the nuts transmitted through the spike plates. Once secured to the slope, it was safe for the crews to drill and install the rest of the dowels on an 8-foot center-center pattern to secure the mesh.

Due to the fact that rolling roadblocks had to be used on the Interstate and the contract imposed $1,000/hour liquidated damages if traffic was delayed past the tight work stoppage schedule, the use of a tall crane on this project was impractical. Instead, GSI opted to use a helicopter to lift the mesh panels up to the top of the slope where 3/4 inch shackles were used to secure the top support rope/mesh to the wire rope anchors. The use of the helicopter greatly reduced the time frame for installing the drape, and also ensured that the road remained open as much as possible without sacrificing public safety.

Figure 7 - Installed Type IV Drape
GSI began the work on installing the TNDOT Type III mesh by locating, drilling, and installing the drape’s wire rope anchors along the top of the slope. At a staging area near the site, a ¾-inch wire rope was weaved through the Rolled Cable Net mesh to serve as the drape’s top support rope. To make the north and south panels, 5/16-inch wire rope was used to vertically seam lengths of the Rolled Cable Net mesh together to make each panel.

SUMMARY

One of the most critical lessons learned on the project was that the monitoring and cleaning of highway slope benches is imperative to their long-term success. If the benches had been maintained, many of the launch features would have been eliminated.

With the benches remediated back to their original design (Figures 9 & 10) and the mesh installed in pinned and drape configurations, “old” and “new” rockfall mitigation technologies now serve to protect the traveling public. With a project design life of 75 years, this solution will continue to contain and control any falling rock for many years to come.
Figure 9 - Bench Final Shape

Figure 10 - Final Bench Behind Catchment Barrier
REFERENCES:

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Practical Considerations for the Design and Construction of Landslide Mitigation using Horizontal Drains

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ABSTRACT

Significant study and theoretical discussion has been devoted to drainage and horizontal drains for landslide mitigation. However, in practice horizontal drains have a reputation for being risky or potentially unreliable. Drains are sometimes installed without much engineering investigation, analysis and design which may contribute to the reputation for poor reliability. At the other end of the spectrum are extensive groundwater monitoring studies, transient flow modeling, and parametric analysis of slope stability. In addition, commonly accepted design and construction procedures are still evolving. Therefore, this paper presents an engineering design method using commonly available data and analysis tools with an emphasis on a constructible system to provide reliable landslide mitigation.

Drawing from a series of case histories and published literature, practical considerations for the site investigation, design and construction of horizontal drains were developed to improve the reliability of horizontal drains. A simplified slope stability model is used to illustrate why drains work and what site conditions are needed for reliable mitigation with horizontal drains. Detailed site conditions and groundwater fluctuation data are used to develop a groundwater seepage and slope stability model using site data and back calculation of soil strength and groundwater parameters. Horizontal drains are added to the seepage model and the effect on the slope stability is calculated to provide an analytical evaluation of the mitigation potential of the drains. Design details, specifications, construction observation, and design adjustments provide the construction control needed for the drains to perform as intended.
BACKGROUND AND PURPOSE

Abundant theoretical and research publications are available that address horizontal drains for landslide mitigation. However, how this information is applied to the practical design of a landslide mitigation project is left largely to experience and no commonly accepted design procedures or standards have evolved.

The purpose of this paper is to share some experience and mostly qualitative concepts for the practical design of horizontal drains with real world site and budget constraints. This discussion assumes that the investigation is limited by access and cost. Subsequently, the methods discussed may not be appropriate for critical facilities or high value applications that justify more extensive investigation, analysis and design. However, the concepts presented herein should be applicable to most roadway situations.

This paper is organized following an approach. However, the intent is to provide background and concepts rather than a formal design procedure. This discussion is largely based on the experienced gained from a series of horizontal drain projects over the last 20 years. The proposed methods follow the typical procedure of any engineering endeavor, starting with an investigation of the existing conditions, followed by interpretation, modeling and analysis. The results of the analysis are then applied to a design that considers the limitations of the available construction tools. Finally, the constructed condition is compared to the intended design before accepting the final product.

Background

The design of horizontal drains is based on an understanding of landslide investigation, slope stability analysis, and the fundamentals of groundwater hydrology. Concepts specific to practical horizontal drain design are discussed herein, but a more theoretical and broader background is provided in numerous publications. *Subsurface Drainage for Slope Stabilization* (Forrester, 2001) provides an overview and extensive discussion of a broad range of drainage options. Much of the theoretical work on horizontal drains discussed in this literature was adapted from research on agricultural drains intended for shallow application in homogeneous soil. Recently the Washington State Department of Transportation published a research report titled, *Design Guidelines for Horizontal Drains used for Slope Stabilization*, (Pohll, 2013) which provides a comprehensive discussion and guidelines for groundwater analysis for the design of horizontal drains. In addition, numerous texts on slope stability analysis and landslide mitigation are available including *Landslide Investigation and Mitigation*, (Turner, 1996). The examples used in this report were developed using the commercial software Slide 5.0 (RocScience, 2010) which provides integrated 2D steady state seepage and slope stability analysis.

*Horizontal Drains – State of Practice: The last Seven Decades in the US* (Lee, 2013) presents a discussion of the state of practice for horizontal drain design in the US. Design practices vary widely, but typically include local experience, subjective methods and numerical simulations with validation through field testing. While design methods vary, the literature appears to support that horizontal drain installations based on formal investigation, analysis and design are more reliable than drains installed without formal design.
Basic Horizontal Drain Considerations

The relationship between groundwater and slope instability is well established, so placing drains in the slope to improve stability is an obvious approach. However, the simplicity of the concept has lead to the installation of horizontal drains without much analysis and design. Understandably, installations without investigation, analysis and design will be less reliable. If higher risk is acceptable, or in emergency situations, the time and expense of investigations, analysis and design may not be justified by the increased reliability. On the other hand, modeling and analyzing the complexity of groundwater and slope stability requiring 3D transient flow and slope stability modeling (Pohll, 2013) may require more resources than is available for many projects. However, with appropriate assumptions and design details that consider the assumptions and site variability, horizontal drains may be designed with limited resources and site data.

Horizontal drain considerations based on the author’s observations that are generally consistent with literature include:

1. Horizontal drains are generally effective in reducing movement (Martin 1984).
2. Drains purposely designed with slope stability analysis are more effective than drains installed without supporting analysis.
3. A small percentage of the drains produce the vast majority of the discharge (Martin 1984, Lee 2013).
4. Long drains deep in the landslide are far more effective than shallow or short drains (Pohll, 2013).
5. Drains installed to intercept groundwater above the slide are less effective than drains installed in the landslide (Martin 1984).
6. Protection of the outfall from freezing, roots, and physical damage is required for long term performance (Lee 2013).

These basic considerations for the design and construction of horizontal drains are woven into the following discussions.

SITE INVESTIGATIONS

All landslide investigations should begin with thorough mapping of the site. The mapping should extend from well above the headscarp to the base of the hill and include any ground cracks, wet areas, seeps, springs and areas of surface water. Mapping is likely the most important part of the investigation and should be performed by experienced personnel who are assisted by less experienced professionals to provide training for the next generation.

Design of horizontal drain mitigation requires the same basic investigation that should be performed for any landslide mitigation. However, when considering horizontal drains, additional data regarding groundwater conditions are required. Realizing that the location and number of exploratory borings is limited by access and economics, additional groundwater data often needs to be collected without adding substantial project cost. At least one boring is required for small
slides with an obvious slide surface and at least two borings, located in-line, are strongly recommend. Additional boring should be added for larger landslides or more complicated conditions. *Landslide Investigation and Mitigation* (Turner, A. K., Schuster R.L., Ed., 1996) presents five chapters on landslide investigations.

Slide surfaces often consist of a clay rich material with relatively low permeability that forms a partial aquaclude. Typically, the slide mass consists of fractured rock or soil with a much higher permeability than the surrounding material. Therefore, the appropriate location for monitoring the pore pressure is typically just above the slide surface. However, there are cases where the pore pressure below the slide surface controls. Clayey colluvium, alluvium and embankment fill over fractured rock may be of lower permeability than the underlying material. If this is the case, the pore pressure in the underlying rock may be much higher than above. If this situation is suspected, nesting a piezometer above and below the suspected slide surface is recommended.

Landslide analysis requires an understanding of the pore pressure at the slide surface, so piezometers needs to be designed and installed to monitor pore pressure at a specific elevation. Preferably, the piezometer sensor should be located to monitor the pore water pressure close to the slide surface. An open stand-pipe piezometer or stand-pipe with more than about 5 feet of sanded section may provide results that do not reflect the pore pressure. The piezometer should be installed in an identified layer and not straddling a potential aquaclude or seepage zone. Vibrating wire piezometers (VWP’s) are ideal for this application and can be nested or installed in the same boring as a slope inclinometer. A stand-pipe piezometer may also be used, but careful thought should be given to sealing the piezometer and the location of the slotted and sanded zone.

Landslide mitigation design requires an understanding of the peak pore pressure and ideally how the water pressure relates to the slide movement. Collecting data on peak ground water conditions requires monitoring through the wettest weather of the year. Fortunately, data loggers for VWP’s or stand-pipe piezometers are commonly available. Data loggers can record pore pressure on an hourly to daily basis without need to visit the site and therefore, can be cheaper than manual monitoring. The value of continuous groundwater monitoring is demonstrated in the following sections.

Finally, monitoring the slope movement and investigating the site history provides valuable clues to understanding the site conditions. Surface point monitoring provides information on movement rates and the relation of movement to groundwater conditions. However, when long term monitoring data is not available, maintenance records and verbal histories are still valuable.

**APPROPRIATE SITE CONDITIONS**

Horizontal drains are seldom be the first choice for landslide mitigation, but under the right circumstances, drains provide reliable and cost-effective mitigation and in some cases they may be the only reasonable option. The following provides a discussion of the quantitative conditions that represent a site suitable for mitigation using horizontal drains:
1. Horizontal drains work best at sites with high groundwater conductivity. Since conductivity in landslides is frequently provided by fractures, low permeability soil and rock may still provide the required hydraulic conditions. Aquifer testing like pump and slug tests can provide data to calculate conductivity, but a vertical well may not identify the lateral hydraulic properties of a vertically jointed material. Also, in staying with the theme of designing with limited resources, less expensive sources may be used to identify appropriate groundwater conditions. For example, sites with high groundwater conductivity will show rapid fluctuations in the groundwater elevation in response to precipitation.

Figure 1 shows an example of groundwater monitoring in a slide where the water level increased 20 feet in three days and then dropped 15 feet in 5 days. The rapid fluctuation provides a qualitative assessment of groundwater conditions indicating relatively low effective storage and relatively high effective permeability. These conditions represent a site where removal of small volume of water will result in large drop in head.

![Figure 1 – Groundwater response to precipitation](image)

2. Slide movement should be observed only following high precipitation events. Slides that continue to move during relatively dry conditions where the groundwater is at a minimum may not be good candidates for mitigation with horizontal drains. At the site in Figure 1, movement was only observed following extreme precipitation events, indicating that the base groundwater level represented a stable condition. These conditions indicated the site had favorable groundwater conditions for mitigation with horizontal drains.
3. Typical landslide soils, consisting of medium stiff to hard colluvium or decomposed rock with high clay content, are usually highly preconsolidated and will tend to have the fractures required for horizontal drain performance. Soft and near normally consolidated soil or high plasticity clay may not have the permeability for effective drainage. Knowledge of the local conditions and the observations discussed above may be used to judge the suitability of the site and soils.

4. In addition to favorable groundwater conditions, an accessible site is required to construct the horizontal drains. While drains have been constructed from deep excavations, shafts and tunnels, these methods are not consistent with the limited resources theme of this discussion. Therefore, we will assume the drains will be constructed from an open excavation of moderate depth. The drains should start from near the toe of the slide and preferably be located entirely within the slide mass. The drains should parallel the slide surface as much as possible and angle up at greater than 3 degrees. The availability of access to a suitable drain construction site often limits the application of horizontal drains.

While the ideal site matching all of these conditions may not exist, the further the site varies from the described conditions, the less reliable the outcome and the more complex the analysis and design. In emergency conditions, the groundwater monitoring and movement history may not be available and therefore, the assessment of the site for horizontal drains will be based more on judgment and experience.

MODEL DEVELOPMENT AND ANALYSIS

Even relatively simple landslides are complex when considering the limited data and analytical tools. Data will always be inadequate for an absolute solution. Therefore, it may be helpful to think of the analysis in terms of a learning tool rather than a mathematical solution. By basing the analysis on a comparison of the conditions at failure to the proposed conditions, the analysis results can be used to provide a rational basis for design. The following discussion applies the concept of back calculation to the groundwater and slope stability analysis. Design and construction concepts are presented to help develop reliable designs that mitigate the effect of the limited data and numerous assumptions used in the analysis.

Groundwater Modeling

Just as 2D analysis of a 3D landslide is a conservative assumption, steady-state seepage analysis is a conservative assumption for transient flow. This discussion assumes the groundwater model will be set up for 2D steady-state seepage analysis and the same subsurface model will be used for slope stability analysis.

1. Set up the model area larger than would be needed for the slope stability analysis. This helps reduce the effect of the boundary conditions.

2. Set up the subsurface material layers based on the subsurface exploration and local experience. Typically, the slide surface is represented by a relatively thin zone of weak, low permeability material. Keep the model as simple as possible.
3. Select typical hydraulic parameters for the soil layers. Jointed rock and cracked materials within the slide area may have hydraulic conductivity values several orders of magnitude higher than the intact material properties. Include a vertical-to-horizontal hydraulic conductivity ratio for layered conditions.

4. Select a reasonable infiltration rate and boundary flow rates for the site conditions. The infiltration rate may be higher than the published values due to tension cracks and disturbance of the ground surface.

5. Run the analysis and adjust the parameters until the resulting model appears consistent with the observed conditions. The modeled groundwater condition should show a groundwater level consistent with the anticipated groundwater at failure. Surface seepage should be consistent with the site observations.

While this method is not intended to develop a comprehensive model of the groundwater condition, it does provide a reasonable model for evaluating the affect of horizontal drains. Alternatively, instead of modeling the steady state seepage, the groundwater model can assume the groundwater elevation at failure for back calculating the slide surface strength. Using assumed water levels requires additional judgment and does not provide guidance regarding drain spacing and effectiveness. However, seepage modeling generally provides lower water levels than the assumed conditions and therefore, may be less conservative.

Figure 2 shows a groundwater model developed using SLIDE 5.0 (RocScience, 2010) for a site with an aquaclude below the slide surface and perched groundwater within the slide. Infiltration and hydraulic conductivity values were adjusted until the model agreed with the field observations of the groundwater elevation and seepage area.
Slope Stability Analysis

Stability analysis for landslides is the comparison of an existing condition at failure to the proposed mitigated condition. As long as a reasonable model is used, the results are not highly sensitive to the analysis details. Therefore, developing a model that represents the site conditions is the critical step.

Circular slip failures are commonly used in analysis of slopes and embankments. However, circular failure surfaces do not occur in landslides and may provide misleading results when comparing landslide mitigation alternatives. Slide surfaces follow the weakest available layer, which is typically the soil-weathered rock interface or pre-existing weak layer such as bedding or relic rock structure. These surfaces are seldom circular. Therefore, a block failure should be used in the analysis. The following briefly describes a procedure for slope stability analysis with horizontal drains:

1. Typically model the slide surface with a thin low-strength layer and higher strength material in the slide mass.

2. Assign a residual shear strength to the base failure surface based on material type, local experience and residual direct shear strength tests if available. Assign a shear strength to the slide mass that is typically at least 10 degrees higher than the slide surface.

3. Include a tension crack consistent with site observations and include water in the tension crack equal to the groundwater elevation or observations.

4. Use the pore pressures from the seepage analysis or the assumed groundwater elevation.

5. Calculate the limited equilibrium factor of safety (FS) using any method intended for block failure analysis.

6. Revise the shear strengths such that the analysis provides a factor of safety near 1.0. Compare the observed search surface and analysis with the field observations and adjust the model as needed such that the model results agree with observations and local experience.

Figure 3 shows the slope stability analysis developed using SLIDE 5.0 (RocScience, 2010) for a site with a nearly horizontal slide surface at the top of a layer of weathered tuff. The results provided a back calculated shear strength value similar to the values calculated for similar sites in the area, and the results included a tension crack near where the scarp was observed.
Horizontal Drain Analysis

The final analysis step is to evaluate the effect of horizontal drains on groundwater and stability. When using a two dimensional slope stability model for a three dimensional slide, a discharge point is used to represent the linear drain. The following outlines the analytical approach:

1. Place drainage nodes at the elevation of the proposed drains and use an initial spacing of about twice the depth of the groundwater above the slide surface.

2. Rerun the seepage analysis and use the resulting groundwater pore pressure or assumed groundwater elevation to rerun the stability analysis.

3. Adjust the drain spacing until the resulting slope stability analysis shows the target increase in stability, typically 25% to 50% (FS=1.25 to 1.50), but other values may be appropriate. Once the groundwater elevation between the drains is close to the drain elevation, additional drains will not result in additional lowering of the groundwater.

4. Check the sufficiency of the drainage and factor of safety by increasing the infiltration to a worst case condition (try doubling the infiltration rate). The factor of safety should still be close to 1.0.

Alternatively, if a seepage calculation is not being used, the designer may choose a groundwater elevation based on an assumed drained groundwater level. Typically, assuming a drained groundwater level equal to ¼ the drain spacing above the drains should provide a reasonable starting point. Figure 4 shows the drained slope stability analysis developed using...
SLIDE 5.0 (RocScience, 2010). The results showed a factor of safety of 1.5 for the drained condition and 0.95 with double the initial infiltration rate.

**Figure 4 – Slope stability analysis with horizontal drains**

All of the conditions and methods described may not be applicable to all sites, and compromises will be required. The desired factor of safety may not be supported by the analysis or the preferred elevation for the drains may not be constructible. For large slides or when horizontal drains are the only remaining option, designing for a factor of safety less than 1.25 may be the only reasonable alternative. Again, the designer must consider the risk of the compromises against the available alternatives.

**DESIGN AND DESIGN DETAILS**

The results of the analysis must be interpreted and design details developed such that the constructed product provides the results simulated in the analysis.

**Interpreting Analysis Results for Design**

The method described above uses drainage nodes spaced along a cross section. The node spacing may be used to provide a reasonable starting point to guide the selection of the horizontal drain spacing in the upper portion of the slide. Use the node spacing in the cross section to estimate the drain spacing in plan view. Calculate the number of drains needed to provide the estimated spacing.

The analysis provides guidance regarding the spacing and minimum number of drains. However, considering the assumptions that have gone into the analysis and the reality of the construction, more drains are required than the analysis shows. Try doubling the number of drains if space allows and consider reserving 1 or 2 drains as contingencies that can be added during construction. The purpose of the additional drains is to add redundancy. Since the
additional drains are not expected to lower the water level, so they do not actually increase the factor of safety. The reason for the additional drains will become more apparent in the construction discussion.

Following the selection of the number and spacing of the drains, the designer must lay out the drains to target the area identified in the analysis and drain to a convenient location for the drain gallery. The drain gallery should be located near the toe of the slide and as close to the slide surface as practical. The drains should extend past the headscarp and lateral cracks to collect water from these obvious fractures. Fanning the drains across the slide varies the angle of the drains and increases the chances of intersecting joints. Fanning also concentrates the outfalls and reduces the size of the drain gallery. However, a minimum of 5 feet between drain outfalls is recommended to facilitate construction.

In most cases, a single drain gallery at the toe of the slide is appropriate. However, multiple galleries may be required to cover all areas of the slide. Only the lowest drains produce, so rows of drains arranged vertically are ineffective. Figure 5 illustrates the location of a drain gallery and the drains fanned across the slide area.

Figure 5 – Drain gallery and proposed drain locations

Designing for Construction and Maintenance

The preceding discussion presented a rational design process for locating the drains and selecting the drain gallery location. Construction details are required to provide a suitable place for the drilling operation and to protect the outfall from freezing, roots, and physical damage.
The drains will typically be drilled from an excavation near the toe of the slide. The dimensions of the drill should be used to guide the size of the excavation. Typically, the excavation should be 4 to 5 feet below the drain collar elevation and at least 20 feet wide. Some equipment and drain angles may require more room. Placing at least 1-foot of quarry rock in the base of the excavation will help stabilize the drill because horizontal drain construction is a very wet and muddy operation. Figure 6 shows a drill working in a drain gallery.

![Figure 6 – Drain gallery during horizontal drain installation](image)

Drain clogging and maintenance is covered in the literature. However, actual reports of clogging are hard to find. In some environments, iron and caliche deposits could lead to clogging and maintenance should be included in the design. However, consider how realistic it is for local maintenance crews to service the drains every 5 to 10 years. Clogging from sedimentation in the drain may be controlled by providing at least a 3% slope to the drain.

Protecting the drain outfall is critical to long term performance. The outfall will become a wet area even if the drain water is collected. This wet condition leads to slumps and vegetation growth that can damage the drains. Therefore, protect the outfall with a length of steel pipe extending 5 to 20 feet into the slope and backfill the drain gallery with free-draining quarry rock to reduce the risk of freezing and protect from local slumps. Figure 7 shows a detail used to protect drain outfalls.
CONSTRUCTION AND CONSTRUCTION MONITORING

As with any construction, monitoring is required to confirm that the constructed product meets the design intent, and flexibility and design adjustments are required to provide the drainage as designed. Therefore, a skilled driller and cooperation with the design team is vital during construction. Typically, the plans include a drain gallery and drains fanned out across the slide as illustrated in Figure 5. In reality, adjustments are often required in the field to position the drill and angle the drain to the target area and elevation within the slide.

While drill steel may appear ridged in 10 foot lengths, 200 feet or more into the slope the borings begin to wander. In relatively soft soils, the boring may begin to dip below the target elevation. In hard material or cobbles, the drain may climb well above the target elevation. Drains may also angle off to the side. A skilled driller can adjust the pressure, water and bit selection to influence the angle, but expecting to hit the target every time is unrealistic. This is one reason for adding redundant drains as discussed above. Following the installation of the first drain, survey the drain position to assess the first drain and adjust the aim on the next drain. A mechanical Pajari has been traditionally used to survey the boring, but it is slow and more modern electronic survey instruments are preferred. After each drain, compare the drain location to the planned location and discuss with the driller how to angle the next boring and adjust the drilling to get closer to the target.

Monitor the initial drain flow and the change in flow over time. Typically, drain flow will drop dramatically between the initial flow and steady state flow. At steady state, the flow may only be a few drops, but it only takes a small volume of water to decrease the pressure dramatically. Drilling of adjacent drains should result in decreased flow, indicating that the drain spacing is appropriate. Consider adding a contingency drain into a zone with very high flow or if one of the drains was too far off target. Table 1 shows a series of flow measurements taken during installation of eight drains.
Table 1 – Horizontal Drain Summary

<table>
<thead>
<tr>
<th>Horizontal Drain Number</th>
<th>Collar Elevation (feet)</th>
<th>Angle (Degrees)</th>
<th>Total length (Feet)</th>
<th>Initial Flow (GPM)</th>
<th>End of Construction (GPM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>53</td>
<td>10</td>
<td>180</td>
<td>0.3</td>
<td>-</td>
</tr>
<tr>
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<td>52.3</td>
<td>8</td>
<td>330</td>
<td>15</td>
<td>4.6</td>
</tr>
<tr>
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<td>52</td>
<td>8</td>
<td>340</td>
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<td>1.2</td>
</tr>
<tr>
<td>4</td>
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<td>9.5</td>
<td>340</td>
<td>12</td>
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<td>13.6</td>
<td>1.1</td>
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<td>10</td>
<td>320</td>
<td>22.7</td>
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</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>17.5</td>
</tr>
</tbody>
</table>

Monitor the piezometers during the drain installation and for at least the following wet season to confirm that the intended drainage has been achieved. During installation of the drains it is common to observe a spike in the piezometer when drilling in the immediate vicinity followed by a rapid drop in the groundwater elevation. Post construction monitoring should show the groundwater staying within the assumed design parameters during precipitation. Figure 8 shows the groundwater elevation, rainfall and slope movement before, during and after the installation of horizontal drains.
FIGURE 8 – Groundwater elevation, rainfall and slope movement

FINAL DISCUSSION

Horizontal drains have been a relatively common landslide mitigation tool since the 1960’s and there are numerous research and theoretical publications available. However, a commonly accepted design procedure has not evolved. The procedure discussed here is based on observation and experience with horizontal drains over the past 20 years. This experience and the resulting concepts and procedure were shared in hopes of stimulating further discussion, and eventually working towards a common design procedure that reflects an improved understanding of the design and construction of effective horizontal drain systems. Therefore, please share your experience and ideas with the author and the geotechnical community.
REFERENCES:


Anchored Shear Piles for Landslide Mitigation:
Design Approach and Instrumented Performance

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ABSTRACT

Structural mitigation of landslides is increasingly necessary where existing geologic hazards intersect with displacement-sensitive infrastructure. Conventional shear pile systems require passive resistance be developed by displacement of the landslide. Where displacement of the landslide or structural system is undesirable, anchored shear piles (ASP) can be used to develop full-depth restraint of large landslide masses with limited post-construction deformation. No published standard for the design of such a system exists, and a typical ‘lateral-loading-of-pile’ approach can yield inefficient or even unconservative designs. This paper presents a design approach for landslide mitigation through the use of high-capacity ground anchors integral with cast-in-place concrete shear piles. ASP take advantage of increasing soil strength with depth to apply much larger ground anchor loads at the surface than could obtained through ground anchors on bearing blocks. In addition, construction of an ASP system can provide full-depth resisting force to a landslide, without the risk and right-of-way impacts of large open-cut excavations. A recent case history with instrumented structural elements will be discussed. An outline of the design method, construction considerations, and performance of an ASP system is presented. Inclinometer data is presented to outline in-situ performance, and to discuss the applicability and accuracy of design predictions.
INTRODUCTION

Structural mitigation of landslides is increasingly necessary where existing geologic hazards intersect with displacement-sensitive infrastructure. As infrastructure often needs be repaired or replaced within existing right-of-way or environmental boundaries, the available footprint and work area for such mitigations is frequently more constrained than the area required for the original construction of the infrastructure. Such restrictions can limit the applicability of toe berms, buttresses, drains, and multiple ground anchor rows due to the large fills, excavations, and footprints required to construct these measures.

In addition to footprint constraints, advances in the seismic design of structures have led to an increase in the evaluation of structure deformation under extreme events. When shallow or deep foundations elements are bearing on or passing through the landslide mass, it is often necessary to develop displacement estimates for design events of different return periods. In the case of large, translational landslides, it can quickly become cost-prohibitive to construct a mitigation with enough stabilizing force that the landslide mass remains stable during shaking. In this case, characterizing the load-displacement behavior of the mitigation system is necessary to develop an estimate of the mitigation and landslide response to seismic or other extreme event loading.

Anchored shear piles (ASP) can be used as a structural mitigation system that provides full-depth stabilizing force to a landslide mass with minimal excavation footprint. The system consists of a series of drilled shafts (shear piles) that penetrate through the landslide mass into the stable strata below, see Figure 1. The shear piles are connected at the top with a concrete grade beam. The grade beam contains blockouts for the installation of high-capacity ground anchors. The grade beam can be designed to allow for ground anchors to be relocated laterally along the length to avoid utilities or other subsurface obstructions.

![Figure 1 – Components of an Anchored Shear Pile System.](image-url)
In what follows, a methodology will be outlined that identifies the critical areas of ASP design. The lateral loading design of the piles will be completed using p-y curves, implemented by software such as LPILE or FBPIer. The design of each individual element in the ASP system is closely related to the performance of the whole system, and the effect of changes to any element or material on all other elements must be considered. The seismic performance characterization of the system will be evaluated by determining the load-displacement behavior of the entire system and making modifications to traditional Newmark seismic deformation analyses.

This paper will present and discuss the application of the ASP design methodology to a large landslide in Portland, Oregon. The project involves a bridge replacement, where an existing bridge is to be replaced within an existing bridge corridor. The existing bridge has been damaged by movement of a large ancient landslide discovered shortly after bridge construction in 1925. The landslide is approximately 800 feet long, 500 feet wide, and 65 feet deep, and has displaced in excess of 3 feet since the bridge was constructed, resulting in significant distress and requiring multiple rounds of repair work. Construction of the new bridge required large cuts at the toe of the ancient landslide. The landslide mitigation system needed to be constructed in short sections that would allow for multiple operations to occur simultaneously. Closely sequenced construction methods including ground anchors and balanced cut/fill were used to construct the landslide mitigation and other elements without significant landslide displacement.

**Cantilevered Shear Pile vs. ASP**

The use of shear piles, or reinforced-concrete drilled shafts that penetrate a shear zone, is becoming more common. Steel micropiles have also been utilized as shear piles in some landslide mitigation systems. Cantilevered shear piles develop loading through landslide displacement. As the landslide displaces, the relative movement of the landslide and pile generates loading to resist landslide movement. This load is resisted by developing a reaction in the socket material below the landslide mass. Since the mitigation system only develops loading with landslide displacement, mobilization of additional load requires additional displacement. In this case, the added load induced by seismic shaking may require substantial landslide displacement to develop the loading to stop the slide mass. In situations where this additional deformation can be tolerated, cantilevered shear piles are typically cost-effective.

In a cantilevered shear pile, the top of the pile is allowed to displace freely. With sufficient landslide displacement, the top of the pile will move more than the landslide itself, and will begin to develop a reverse curvature, see Figure 2. This behavior is typically not desirable, since this effectively pushes the upper portion of the landslide downslope. The shear pile can be terminated below the ground surface to limit this behavior and provide cost-savings where the contribution of the pile to slide stability becomes negligible.

In an anchored shear pile system, a vertical element is loaded at the top with a tension load against the direction of slide movement. The vertical element is typically very stiff relative to the surrounding soil, so the anchor loading will be transferred down the pile, as shown in Figure 2. Since the soil materials in the landslide mass typically become stiffer and stronger with depth,
anchor loading will be transferred into the more competent soils and allow for higher anchor loadings than could be attained at the surface.

The distribution of the landslide resisting load in a cantilevered system is concentrated towards the bottom of the landslide mass and requires landslide displacement; whereas ASPs distribute the surface-applied loading throughout the landslide mass, with a distribution of loading that better reflects the capacity of the soil with depth. As a result of this favorable distribution to deeper soils, high anchor loadings can be applied without being limited by surface soil bearing capacities. The concept of “stiffness matching” between the pile and soil can provide a means of efficiently transferring the loading into the landslide mass at depth, while not requiring the mass excavation and large footprint typical of top-down ground anchor construction.

Figure 2 - Pile Displacement and Developed Soil Reaction of Cantilevered and Anchored Shear Piles. Behavior Shown for Initial Loading (Solid Lines) and Loading Following Landslide Movement (Dashed Lines).
The anchor loading applied in an ASP system provides active restraint of the landslide, resulting in a Factor-of-Safety (FS) greater than 1.0. This raises the initial dynamic yield coefficient, and reduces the seismic landslide displacement. In a cantilevered approach, the system is always near FS=1.0, and the initial displacement required to mobilize the resisting load can result in substantial distress to the system and adjacent infrastructure.

**DESIGN PROCEDURE**

Many approaches to the design of laterally loaded shafts for use in landslide mitigation have been attempted. Procedures can be displacement-based as implemented by computer design programs such as LPILE (1), or by the use of numerical methods to solve the problem using finite-element or finite difference software. In this study, LPILE was used to determine the pile and soil response. Custom p-y curves were generated based on in-situ and laboratory testing.

The focus of this paper is on the aspects of the landslide mitigation related to the design of the ASP system. However, the success of any mitigation design is heavily dependent on the quality of the landslide characterization and interpretation. Care should be taken at all steps within the design to evaluate uncertainty and potential pitfalls associated with assumptions within the various models.

Selection of the optimal location for an ASP system will vary based on the particulars of the landslide geometry and materials. In general, the larger the landslide mass, the more ground anchor loading will be required to meet the same seismic response criteria. All critical project design elements should be located upslope of the mitigation, as this is the area of the most reliable predictions of landslide displacement. Minimizing the landslide mass to be retained has advantages, however the material type and loading requirements will sometimes utilize too much of the total capacity of the soils, resulting in creep and load loss in the anchor. In this situation, the section of the landslide that is the deepest is more desirable. The deep section of slide debris provides the greatest depth to develop the soil reaction and has the highest lateral capacity.

**Material Characterization**

Materials at all depths along the pile should be characterized as accurately as possible. In-situ tests of soil strength and stiffness should be supplemented with additional laboratory methods such as triaxial tests. Additional borings may be useful to capture material behavior and lithology at the final location of the piles while still in the design phase.

The lateral loading response of piles can be characterized by load-displacement curves called p-y curves. In the case of ASP design, it is important to characterize the lateral load behavior of the soils as accurately as possible. If the lateral loading model is softer or stiffer than the actual material response, there are implications for the design and performance of the shear piles. A parametric study can be useful for understanding the relative effect of uncertainty in the material properties. Soft soil response will generate more pile deflections, and thus higher moments and shears within the pile requiring high reinforcement requirements for the anchor loading. Soft soil responses can also be unconservative, as they will underestimate the magnitude of pile loading in a seismic event. When the seismic landslide displacement it applied, it will generate
lower loads than the pile will actually feel. Conversely, a stiff pile response will underestimate initial displacement, but overestimate the seismic requirements.

As discussed in more detail later, it is important to understand the creep potential of the materials immediately upslope of the piles. Consolidation of the landslide debris due to the loading from the anchors will cause the pile to creep, as will loading the soil above a certain percentage of its ultimate capacity. As with the other aspects discussed above, an accurate characterization of the material properties is critical to a successful and efficient ASP design.

**Limit Equilibrium Slope Stability Modeling**

The first step in the iterative modeling process is to perform static and pseudo-static limit equilibrium slope stability modeling. Although the details of the ASP load application are complex, this initial load calculation can be performed using a horizontal point load located at the lower third point of the ASP. The location, direction, and magnitude of the point load can be refined through subsequent design iterations. As the design progresses, locating the point force at the centroid of the soil response will most accurately model the pile-soil load transfer.

All normal precautions and model-proofing must be undertaken at this stage, as calibration of the stability model to all conditions will affect the design resisting force. The shear piles are typically installed on close spacing, which may cause partial blockage or constriction of groundwater flow paths. The analyses should consider the potential rise in groundwater head upslope of the shear piles, which can result in a higher required anchor force to offset this effect.

**Ground Anchor Preliminary Design**

Once a desired resisting force has been determined, a preliminary anchor design must be completed for input into the pile design. The load per anchor will typically be a function of the pile spacing. For constructability reasons, it is useful to limit the design to one anchor per shear pile. Mid-span anchors can also be constructed, but require a heavily-reinforced grade beam. The resisting force per foot and the pile spacing can be used to determine the required anchor load, and hence the number of strands required. The number of strands and design free length will determine the stiffness of the anchor. This stiffness will determine the response of the system to deformation following anchor lock-off. Such displacement could result from a seismic event.

The preliminary anchor design should take bond materials into consideration, in addition to the free length. Anchor loads can be substantially higher than conventional ground anchors on bearing blocks, where capacity is limited by the surface soils. As a result, more attention should be paid to the capacity and characteristics of the bond materials. Pre-production anchor testing to investigate ultimate bond stresses can provide valuable input during design. In general, fewer anchors of higher capacity are less costly than more anchors of lower capacity, due to the reduction in drilling.
Pile Design

Once the anchor and pile spacing have been loosely defined, the iterative process of designing an efficient pile can commence. Pile design can be carried out using commercially available pile lateral load programs, such as LPILE. An efficient pile design is one that balances the soil stiffness, pile stiffness, and anchor loading such that no deflection (and hence no load) is generated below the landslide shear zone. This ensures that the horizontal component of anchor loading is distributed along the pile within the landslide mass, while the vertical component is transferred into the socket material below. A check of the landslide resisting force is made by integrating the computed soil reaction profile within the landslide mass to determine the total force applied to the soil by the pile.

Stiffness Matching

The stiffness and strength of the soil will typically increase with depth. The amount of increase is dependent upon the material type, stress conditions, and pile diameter. The increase is reflected in the calculation of p-y curves. Pile deformations will roughly follow a parabolic shape, with the maximum displacement at the top of the pile. Although pile displacements are smaller with increasing depth, the increased stiffness of materials at depth increases the soil reaction for a given displacement. In a very soft or ductile pile, the pile will deform close to the surface in response to the anchor loading. This deformation will cause large soil reactions at shallow depths. In contrast, a stiff pile will not deform as readily, generating less near-surface soil reaction. The stiff pile will need to displace further down to develop the necessary soil reaction to put the pile into equilibrium. This in effect transfers the surface loading from the anchor down the pile. The depth to which the anchor load is transferred (i.e. where the pile displacement is zero) will increase the stiffer the pile. This concept of stiffness matching to obtain zero displacement at or near the shear zone is the key to developing an efficient ASP design.

The pile diameter, pile spacing, pile reinforcement, and anchor load per pile will be modified during the design iterations to obtain an efficient pile that meets the design criteria. The pile/soil stiffness matching concept outlined above must be also be evaluated in terms of soil strength. In general, it is desirable to keep the mobilized soil reaction of the pile due to the anchor loading to less than 1/3 of the ultimate soil reaction. This will reduce the potential for creep of the pile once it is loaded. The negative effect of pile creep is two-fold. As the pile creeps and deforms in the direction of the anchor loading, moments and shears generated in the pile increase. Secondly, this deformation results in a loss of pre-stressing load in the anchor, and also could cause deformation further down the pile, essentially offloading pre-stress loading into the socket materials beneath the landslide. These effects reduce the static factor of safety of the landslide.

Redistribution of the anchor loading to minimize creep potential in near surface soils can be accomplished by stiffening pile elements, although this can result in an inefficient design. By increasing pile stiffness, anchor loading is transferred deeper in the pile. This may reduce the displacement (and thus loading) at shallow depths, where weaker soils are more susceptible to creep. This transfer of loading may cause a portion of the anchor load to be transferred below the
shear zone, in which case the load will need to be increased to meet the landslide resisting force target.

**Seismic Analysis**

There are two primary methods for evaluating the seismic performance of the shear pile system. In the case where deformations of the slope are not of principal concern, a pseudo-static factor-of-safety approach can be sufficient. In many cases, the seismic performance of the mitigation system and landslide mass is required for use in the design of other structures. Modifications to the standard Newmark procedure can be made to account for the seismic response of translational slides, as well as for the load-deformation response of the ASP system. With sufficiently detailed explorations and more advanced model development, further refinements to the deformation estimates can be modeled using finite-element software packages, but is outside the scope of this paper. If other structures with significant mass are to be attached to the top of pile or to the grade beam that the full seismic response of the system has to be considered. The out-of-phase shaking of the other structures may compromise the performance of the ASPs.

*Pseudo-static FS Approach*

In the pseudo-static approach, an equivalent horizontal acceleration can be applied to the landslide mass in limit equilibrium analyses using traditional methods. The design resisting force can then be based on the required anchor loading to achieve a suitable seismic factor of safety (e.g. FS=1.1). In this approach, displacements of the slide mass will be small since the seismic acceleration will rarely exceed the yield acceleration of the slide mass. This approach is more applicable to small landslide masses. For large landslide masses, this approach will require very large anchor loads and may not be feasible to construct. This approach does not produce any estimate of deformation.

*Newmark Deformation Analysis*

In this case where deformation estimates are required, a more detailed evaluation of the response of the pile system to landslide displacements must be undertaken. After arriving at a pile system for a static factor of safety that meets all other ASP criteria, a stiffness curve can be developed using the built-in soil movement function in LPILE. In this method, the p-y curves representing the soil at the top of the pile are replaced with p-y curves that reflect the stiffness of the anchor. By imposing increasing soil movements, the soil reaction (see Figure 3(a)) may be summed above the shear zone to determine the increase in total resisting force with displacement. These displacement dependent loads can be entered back into the limit equilibrium stability analyses, and used to calculate a landslide yield acceleration, see Figure 3(b).

Combining these two processes will generate a yield acceleration versus displacement curve. This curve can be used in a traditional Newmark analysis, where the increase in mitigation load with displacement will be accounted for, see Figure 3(c). As the landslide displaces, the applied resisting force will increase, raising the yield acceleration. This will require stronger accelerations to induce movement during subsequent cycles. Accounting for the displacement
dependent behavior of the mitigation will provide more accurate prediction of landslide movement. Increase in anchor load due to pile-top displacements can be calculated. If Newmark deformations exceed project criteria, the ASP design must be modified to achieve the criteria and acceptable anchor performance during the design seismic event.

Figure 3 – Schematic of the Newmark Deformation Analysis Procedure. (a) Soil Reaction Computed Using LPILE (b) Yield Acceleration Computed From Slope Stability Modeling. (c) Newmark Deformation with Time Using ASP Load-Displacement Behavior
Observational Approach

Projects rarely have the available funds to perform substantial pre-construction full-size lateral load testing of piles to calibrate soil material models. In the case of an ASP system, each production pile offers an opportunity to perform such a test once it is constructed. Instrumenting a small percentage of piles with inclinometer casing and load monitoring devices is a relatively inexpensive method of design validation and verification. Deflected shape data obtained from the inclinometer casings can be used to back-calculate moments and shears in the pile, as well as calibrate p-y curves used in the design. This calibrated as-built model of the ASP loading can be used to determine if additional stabilization load is necessary to meet project performance criteria, prior to the completion of construction.

Following the completion of ground anchor installation and stressing, the ground that is loaded behind the piles will react and readjust to the new stress conditions. This will cause the pile to move upslope, relaxing the anchor. This process can happen quickly or take several months, depending on the landslide materials. Other interim construction conditions may cause landslide movement, even after ground anchors are installed and stressed. Cuts, fills, and changes in groundwater conditions can cause movement to occur. This movement can be quantified using a combination of landslide inclinometers, shear pile inclinometers, and load monitoring devices. Using load monitoring devices or in-place inclinometers equipped with dataloggers can allow the designer to understand the cause of movement. The onset of movement, together with current topography, anchor loads, and groundwater information, provides an opportunity to calibrate the strength parameters and geometry of the stability model.

If load is lost from pile creep, or if additional loading is required from changes in landslide interpretation, there are several methods that can be built into the initial design to increase the loading during construction. One way to add additional load is through ground anchor re-stressing. Load lost during pile creep can be restored by leaving the stressing tails on the anchors and shimming the anchor head. The tails should have some means to prevent corrosion of the strands. The final grouting and finishing of the anchors can be delayed until the end of construction, when the desired loading is attained. If re-stressing does not provide the required loading, additional anchors may have to be installed. Additional anchor blockouts may be included in the grade beam to accommodate additional anchor loadings. The beam should be designed to handle the full anchor complement. The anchor blockouts are relatively simple to install during the initial construction, compared to the arduous task of coring new anchor locations through a finished grade beam.

CASE HISTORY

The Sellwood Bridge in Portland, Oregon is being replaced within its existing corridor. A landslide at the west abutment of the bridge has caused distress to the existing structure since its construction in 1925. Geotechnical investigations were performed in several phases to evaluate the subsurface conditions for the design of the project, including drilled shaft foundations and landslide mitigation measures. Approximately sixty feet of highly variable landslide debris was encountered consisting of granular soil, fill, alluvium, and decomposed volcanic tuff. The
landslide debris also contains numerous boulders of R4-R5 basalt. The slide debris is underlain by volcanic tuff, which varies in consistency from a medium stiff clay to a weathered or decomposed weak rock. The shear zone consists of clay with a residual friction angle between 6° and 10°, based on the results of ring shear tests and back-analysis. A thick layer of the Waverly Basalt exists below the landslide. The Waverly formation is a moderately weathered to fresh, R4-R5 basalt, which was selected for the anchor bond zone.

Design

A mitigation system consisting of forty, 6-foot diameter shear piles connected by a grade beam and seventy ground anchors was designed and constructed within the landslide, near the shoreline of the Willamette River. The mitigation system was divided into eight grade beam segments, consisting of 3 to 8 piles each. A grade beam segment consisting of five shear piles is shown in Figure 4. The initial design called for one anchor per shear pile, with anchor loads up to 850 kips. The grade beam was constructed with extra blockouts between each shear pile to receive additional anchors if necessary. Additional, smaller anchors with capacities of 280 kips were installed on 10ft by 10ft concrete anchor bearing blocks. The grade beam segments were constructed and stressed individually. The system was evaluated at several key sections, and was designed to apply between 76 and 96 kips per foot width to the landslide.

Figure 4 - Five Shear Pile Grade Beam Segment with Ground Anchor Blockouts and Bearing Pads.
Each grade beam segment was instrumented with one shear pile inclinometer and one instrumented anchor. The inclinometer is contained within the black pipe extending vertically out of the grade beam in Figure 4. The inclinometer was initialized before anchors were stressed. The inclinometers are read as part of on-going construction monitoring program. The instrumented ground anchors are equipped with dataloggers and record the load in the anchor every 4 hours.

Performance

Actual pile top displacements of the ASP system were less than predicted during design. The displaced shape of the shear pile and grade beam is shown in Figure 5. The difference is partially due to the drilling contractor’s use of a surface casing which increased the pile diameter to 7-feet near the top of the pile. In addition, the grade beams were backfilled with low-strength concrete rather than compacted granular materials, and numerous boulders were present near the surface, providing a stiffer response than the soft slide debris tested during design.

Figure 5 - Inclinometer Data for Shear Pile SP206.
The design of the shear pile SP206 shown in the figure calculated a pile top displacement of approximately 4.4 inches. The shear zone was located 45 feet below ground surface. The actual pile top displacement after stressing was 1.6 inches, and since crept up to 1.9 inches. Pile creep was recorded in the uphill direction for about 8 months following anchor stressing and lock off, see Figure 6. Pile displacements for the shear pile are shown for different depths along the pile.

**Figure 6 - Creep of the Shear Pile Following Ground Anchor Lock-off.**

Verification anchor tests were conducted to verify the validity of the high anchor loads. The design anchor load was achieved in 37 of the 39 high-capacity ground anchors. Two anchors were unable to retain the full load, and were locked off at lower loads. Ground anchors in the grade beams lost approximately 14% of the lock-off load due to creep of the piles and stress-relaxation of the anchor tendon. Anchor loads were increased during the design process to account for estimates of these losses and total anchor loading is within 3.5% of the target.

Large cuts were required to construct the replacement bridge as well as a temporary detour bridge within the landslide limits. The individual grade beam segments were constructed as part of a detailed construction sequencing plan to reduce landslide movement caused by unbalanced...
cutting and filling during construction. In most cases, landslide movement upslope of the large grade beam segments stopped shortly after the stressing of the ground anchors. Small movement continued to occur upslope of the pile for several months as the slope closed internal tension cracks. No increase in ground anchor loading was detected as the take-up occurred.

Detailed as-built drilling records were kept for all shear piles and ground anchors. The records will be used to refine the assumed lithology used in the design phase. The construction of the shear piles, ground anchors, and other site deep foundations has blocked and reduced the groundwater drainage cross-section, and additional anchors necessary to offset the rise in groundwater are currently being evaluated.

CONCLUSION

Structural mitigations are rarely the most cost-effective means for providing static stability of a landslide. For an increasing number of projects, the small footprint and more reliable seismic displacement estimates are highly desirable. However, there are currently no broadly accepted design procedures.

This paper identifies and discusses several key issues associated with the author’s recommended iterative design process that is relevant for landslides, and discusses the aspects of an efficient and conservative design. As in all cases, the increase in sophistication of the design must be matched with a thorough field and laboratory exploration program.

The observational approach, together with some foresight in design to allow for adding additional structural mitigation if necessary, provides a method for dealing with the uncertainty inherent in landslide mitigation. Instrumentation of mitigation measures provides value by allowing the design to be validated and possibly modified to achieve the design criteria and performance objectives.

For the case study discussed, additional work will be performed to back-calculate the pile moments, shears, and loadings from the deflected shapes obtained through inclinometer data. Ground anchor monitoring will continue throughout the life of the project, and be used to monitor the landslide as well as the performance and creep-behavior of the mitigation system. The need for additional anchors will be evaluated based on the instrumented response and behavior of the landslide and structural mitigation elements.
REFERENCES:

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ABSTRACT

Conventional test methods for roadway compaction cover less than one percent of roadway area. Lack of coverage can lead to unrecognized areas of weak compaction and pavement distress. On the other hand, intelligent compaction (IC) offers a method to measure 100 percent of the roadway compaction. This paper presents the advances in IC technology, its viability through impacts on construction quality and cost, and challenges of implementing IC in Wyoming. A literature review was conducted, focusing on the theoretical background and case studies of IC. The review on soil layers demonstrates significant, positive correlation between measurement values from IC rollers and measurements from in-situ tests. Less reliable correlations were obtained for asphalt pavement layers; however, statistical, boundary element, and finite element methods are being used to improve correlations. Barriers to IC implementation include institutional disincentives and knowledge of its economic impact. A review of state transportation agencies’ quality control and assurance programs reveals that states require in-situ and/or laboratory testing for pavement and sub-surface layers even when IC is implemented during construction. This creates a disincentive for contractors to utilize IC because contractors must still perform in-situ testing. Acknowledging this institutional barrier, several state Departments of Transportation are drafting quality control and assurance specifications for IC. A survey and a two-day workshop on IC were conducted to facilitate IC implementation in Wyoming. The paper presents the outcomes of the survey and provides suggestions on the incorporation of IC into Wyoming’s quality assurance and control program.
INTRODUCTION

The quality of a roadway is related to the quality of the compaction of its pavement and underlying soil. State and local transportation officials have used two general techniques to evaluate the compaction of a pavement and soils: stiffness tests and density tests. These tests reveal properties of the soil or pavement that are measured at several points along a roadway during construction. Point measurement methods, such as the light weight deflectometer (LWD), nuclear gage test (NG), static plate load test (PLT), and coring, have been widely used to measure the stiffness or density of compacted soils and pavements during quality control and quality assurance (QC/QA). These point measurements typically provide compaction data for less than one percent and often much less, of the roadway area [1].

Intelligent compaction (IC) rollers provide a method to gather compaction data for 100 percent of the roadway area by measuring soil and pavement stiffness. IC rollers, also known as “intelligent soil compaction systems,” are defined by the National Cooperative Highway Research Program (NCHRP), Report 676 as having three characteristics:

1. Continuous assessment of mechanistic soil properties (e.g. stiffness) through roller vibration monitoring
2. On-the-fly modification of vibration amplitude and frequency
3. Integrated global position system (GPS) to provide a complete geographic information system-based record of the site

Rollers that integrate items one and three from the above definition are also considered IC rollers by several roller manufacturers, but are referred to as “roller-integrated continuous compaction control” by the NCHRP report. These types of rollers will be referred to IC rollers throughout this paper.

IC rollers obtain stiffness values of the soil and pavement in real-time using, at a minimum, an accelerometer, GPS, and computer outfitted to a vibratory roller compactor. IC rollers are capable of leading to decreased construction costs and duration, improved long-term pavement quality, and improved compaction documentation [1].

This paper presents the advances in IC technology, its viability through impacts on construction quality and cost, and challenges of implementing IC in Wyoming. A literature review was conducted, focusing on the theoretical background and case studies of IC. The findings from the literature review regarding IC as technology are presented in the first section. The second section summarizes the current state of implementation of IC technology in the United States as well as proposed QC/QA specifications and programs. The third section discusses the institutional knowledge about IC through the scope of surveys of private and public transportation professionals. The fourth section discusses a methodology for understanding the comparative costs of IC during construction and throughout a roadway’s life. The final section includes conclusions about IC and recommendations for further investigations.
REVIEW OF IC TECHNOLOGY

History

Dr. Heinz Thurner, a Swedish Highway Administration official, introduced the first derivative IC technology by instrumenting a 5-ton-tractor drawn Dynapac vibratory roller with an accelerometer. The research began in 1974, and the following year Dr. Thurner and his partner Ake Sandstrom founded Geodynamik to advance the technology. Geodynamik introduced first commercial IC roller and its index value related to stiffness in terms of a compaction meter value (CMV), in 1978. Several companies, such as Ammann, Caterpillar, and Ingersoll Rand, have introduced compaction meters with CMV. Bomag introduced its proprietary Omega value and Terrameter in 1982 and the $E_{vib}$ value in the late 1990s [2]. Sakai introduced its compaction control value (CCV) in 2004 [3]. The advancement in compaction technology allowed Austria, Germany and Sweden to establish QC/QA specifications for IC during the 1990s. The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) endorsed Austrian specifications for IC.

Technology

IC is described as having at least two components: continuous assessment of mechanistic soil properties and integrated global position system to provide a complete geographic information system-based record of the work site. IC includes an additional component, which is the ability to adjust the amplitude and frequency in real-time based on the mechanistic properties of the soil. Continuous stiffness measurements are calculated by roller manufacturers based on one of two different methods: harmonics and mechanistic methods. This information is received from the accelerometer and is translated into a roller Measurement Value (MV). The MVs are transmitted to a computer on-board the compactor, which also plots the MV in real-time to a geographical location on a map received from the GPS unit.

The recommended on-board GPS unit should be real-time kinematic, which exhibits better accuracy than a satellite differential GPS. Depending on the state specification, other GPS types may be used but a generally required to meet a specified accuracy. For example, Minnesota Department of Transportation’s draft IC specifications require the GPS accuracy as small as two inches. GPS accuracy and errors must be addressed prior to performing compaction. GPS positioning errors can occur by 1) offset of the GPS receiver to the drum center, 2) data averaging during calculation of the roller MV while the roller is moving, and 3) the roller’s travel direction. The error from offset can be remedied by calculating the offset distance of the GPS receiver into the software of the on-board IC computer. The software should also be designed to factor in the roller’s travel direction. Errors resulting from data averaging occur because the MV occurs at the end of the reporting resolution area. The software must be programmed to account for this error, adjusting the location of the MV to the center of the resolution area [4].
Measurement Values

Generation of Measurement Values

Caterpillar, Dynapac, and Sakai IC rollers calculate MVs based on a harmonics approach. Generally, these values take a ratio between the sum of the amplitudes of several vibrations to the sum of the amplitude of the initial amplitude after impact with a surface. The ratio is often multiplied with a scalar adjusted to the specific roller to produce more easily interpretable values [5]. In contrast, Ammann/Case and Bomag IC rollers calculate MVs based on a mechanistic approach, which was modeled from Geodynamik’s approach. Geodynamik used a mechanically implemented two-piece clamshell eccentric mass assembly within the drum to create a maximum eccentric mass moment $m_o e_o$, where $m_o$ is the mass of the weight within the drum and the $e_o$ is related to the distance of the weight from the center of the drum. This occurs when the roller is driven in one direction with frequency $\Omega$ (rad/s). The eccentric mass moment and frequency would create a maximum time-varying centrifugal force $F(t)$ given in Eq. 1, where $t$ is the time in seconds. Particularly, the vertical component of the eccentric force ($F_{ev}$) is used by Bomag in its variocontrol roller with counterrotating eccentric masses.

$$F(t) = m_o e_o \Omega^2 \cos(\Omega t) = F_{ev} \cos(\Omega t) \quad (1)$$

The roller amplitude $A$ is calculated using Eq. 2 on the basis of the eccentric mass moment and the drum mass $m_d$.

$$A = \frac{m_o e_o}{m_d} = \frac{F_{ev}}{m_d \Omega^2} \quad (2)$$

Intelligent compaction has been performed using servo-controlled rollers, which have the ability to automatically adjust the vibratory amplitude and frequency to improve roller performance. Ammann/Case, Bomag, and Dynapac IC rollers have the ability to automatically adjust vertical vibration force when operating conditions are not optimal. Further, Bomag and Amman/Case rollers can reduce the eccentric force amplitude as the user-defined roller threshold MVs are reached [1].

Relationship to Roller Operation and Site Conditions

Many research works revealed that roller MVs correlate well with conventional measurement tests, such as the plate load test (PLT) and lightweight deflectometer (LWD), due to the dependence of MVs on soil stiffness. However, the generation of MVs is affected by operating factors and site conditions. The amplitude, frequency, speed, and direction of the vibratory roller are examples of operating factors that MVs are dependent upon. Soil heterogeneity and lift characteristics are site conditions that affect MVs. The effects of these factors on MVs are described in the following paragraphs.

Amplitude of vibratory rollers is affected by the measurement depth. NCHRP Report 676 concluded through case studies that each increase of 0.004 in of amplitude corresponded to an increase of 1.2 in of depth [1]. Depending on the soil characteristics, the change in measurement depth had an impact on MVs. The sole dependence of MVs on amplitude was unpredictable, but
a correlation existed between the MV-amplitude dependence and soil type. Granular soils demonstrated a positive MV-amplitude dependence; whereas, cohesive soils demonstrated a negative roller MV-amplitude dependence. Due to the high variability of the effect of amplitude on MVs, constant amplitude is recommended while conducting QC/QA [1].

The effect of roller MVs as a function of frequency was tested with Sakai and Ammann/Case rollers. MVs depend on roller frequency due to the partial loss of contact with the soil. Sakai rollers experienced a higher loss of contact when the frequency was set to 20 Hz compared to 25 Hz [1]. Due to the MV dependence on frequency, constant frequency is also recommended while conducting QC/QA.

The Sakai and Dynapac rollers displayed a decrease in MV with increasing speed. The roller speed relationship in Sakai and Dynapac rollers occurs because partial loss of contact with the soil is reduced with the vibration energy spread over more soil at higher speeds. Constant roller speed is also recommended during QC/QA with Sakai and Dynapac rollers [1].

Rollers are typically used in forward and reverse directions, especially double-drum rollers. MVs for each direction were taken for Ammann, Bomag, Dynapac, and Sakai rollers. Discrepancies between the MVs for each direction were marginal for each of the rollers [1]. The MVs of each driving direction should be measured at the site and compared to determine the amount of discrepancy.

Soil heterogeneity can greatly affect MVs, which can vary by 100% due to the variability in transverse soil stiffness. Conventional testing is recommended across the drum width to evaluate the soil heterogeneity. If QC/QA is depended on repeated passes, then the passes should be conducted over the same area of soil.

The soil characteristics of the lift and the soil underlying the lift can also affect MVs. IC equipment measures at depths between 2.7 and 4.0 ft and an area from 0.1 to 0.3 ft in front of and behind the drum. The generation of MV based on the measurement of the depth is affected by the lift and layer thickness, relative stiffness of layers underlying a lift, vibration amplitude, and drum-to-soil interaction. The ratio of lift stiffness to sub-lift stiffness greatly affects MVs. MVs were especially unreliable measures of soil stiffness, with up to 50% variability, when 6-in lifts of stiff soil were placed above less stiff sub-lift soil [1].

The variability of underlying soil stiffness is challenge to IC because the measurement depth of the IC roller is so large. In order to capture the mechanistic soil properties of a lift, a method was developed to calculate the lift stiffness while accounting for sub-lift materials. This method involves forward modeling and inverse analysis using finite element (FE) and boundary element (BE) methods. Forward modeling utilized FE and BE to predict the expected MVs for individual lifts. The inverse analysis technique utilized FE and BE to calculate MVs for a lift in real-time; however, the calculation process was timely, requiring between 2.5 and 7.5 minutes to calculate MVs. Empirically based regression models were established from BE results to allow for real-time calculation of MVs. Three models were developed that successfully predicted FE and BE results with less than 3% error for 99% of the data. The lift calculation method is available to commercial equipment manufactures for integration into IC software. MV variability
still existed in small lifts, typically six inches, which contained greater stiffness then the underlying sub-lift soil [6].

Relationship to Conventional Methods of Testing

The ability for IC to be used for QC/QA rests on its ability to predict stiffness. A series of test-beds with differing soil types, moisture contents, and underlying soil properties were tested in several states. The soil material was broken into three groups: non-granular subgrade, granular subgrade, and granular subbase/base. Regression analysis was performed on all of the MVs produced by IC equipment. These MVs were compared to results obtained from conventional point testing methods. Good correlations were achieved with a linear regressions analysis when the test beds had homogenous soil, stiffer underlying layer support, and constant operation settings. However, on several test beds, correlation values between MVs and conventional results did fall out of the range of significance. Lack of correlation is attributed to several factors, such as sub-lift soil heterogeneity, varying moisture content, narrow range of measurements, transverse heterogeneity, and variation in machine operating parameters. Averaging MVs across the drum width, performing multiple regression analysis on soil properties, and maintaining constant operating parameters can decrease variability and improve correlation of MVs and conventional results. Furthermore, operating rollers at lower amplitude settings, between 0.028 and 0.043 inch, can increase correlation. Correlation increased to levels of significance ($R^2 > 0.5$) between MVs and the results from dry unit weight, California bearing ratio (CBR), LWD, PLT, and modulus ($M_r$) when the effects of moisture content, lift thickness, sub-lift properties, and operation parameters were accounted for by using multiple regression analysis. Table 1 summarizes the correlation values in terms of the coefficient of determination ($R^2$) when adjusted for multiple regression analysis between MVs and dry unit weight ($\gamma_d$), LWD, PLT, and CBR tests [1].

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma_d$</th>
<th>Modulus (LWD &amp; PLT)</th>
<th>CBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-granular subgrade</td>
<td>0.6 - 0.8</td>
<td>0.2 - 0.6</td>
<td>0.3 - 0.7</td>
</tr>
<tr>
<td>Granular subgrade</td>
<td>-</td>
<td>0.5 - 0.7</td>
<td>-</td>
</tr>
<tr>
<td>Granular subbase/base</td>
<td>0.4 - 0.8</td>
<td>0.6 - 0.9</td>
<td>0.4 - 0.8</td>
</tr>
</tbody>
</table>

**REVIEW OF IC IMPLEMENTATION**

The Federal Highway Administration (FHWA) promotes IC via its Every Day Counts initiative. The initiative supports local workshops, demonstration projects, development of standard IC specifications, and additional technical assistance for state and local governments to implement IC. State and local transportation agencies are seen as the catalyst to adoption of IC because they provide contractors with QC/QA specifications for compaction of roadways. This section discusses the types of QC/QA methods for IC that are currently being used and also the current status for QC/QA implementation by state transportation agencies.
Quality Assurance / Quality Control Methods

This section contains QC/QA options and guidelines based on NCHRP Report 676, which have been widely used during IC field demonstrations. The report established six options for QC/QA, which exist within three general methods of testing. The first method involves using point measurements to identify the lowest identified MV locations. The second method involves achieving a percent change in MVs over sequential measurement passes. The third category uses calibration areas to establish target values (TV) for MVs for an evaluation area. Table 2 briefly summarizes each option. Guidelines for the QC/QA options include considerations during a measurement pass for roller operation parameters, evaluation areas, calibration areas, and documentation.

<table>
<thead>
<tr>
<th>QC/QA Option</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>Point measurements on least compacted area based on MVs</td>
</tr>
<tr>
<td>Option 2a</td>
<td>Comparing percent change in mean MV between consecutive passes</td>
</tr>
<tr>
<td>Option 2b</td>
<td>Comparing percent change in MV at a location between consecutive passes and requiring a certain percentage of locations to have a percent change lower than a set target value</td>
</tr>
<tr>
<td>Option 3a</td>
<td>Establishing an acceptable correlation between MV and point measurement on a calibration area to create TVs</td>
</tr>
<tr>
<td>Option 3b</td>
<td>Establishing a TV based on the mean MV when the percent difference in MV consecutive passes on the calibration area is less than or equal to 5% on 90% of the calibration area</td>
</tr>
<tr>
<td>Option 3c</td>
<td>Establishing an acceptable correlation between MVs and lab-determined properties on the calibration area to create TVs</td>
</tr>
</tbody>
</table>

QC/QA Guidelines

Several guidelines are recommended while performing any of the QC/QA options. The guidelines for operating parameters, evaluation area, and calibration area allow for more uniformity while performing measurement passes, allowing for more consistency in the generation of MVs. Documentation of each measurement pass should include pre-established types of information. This information should include the roller MV, three-dimensional position with time stamp and GPS quality, vibration amplitude, vibration frequency, travel speed, driving direction, automatic feedback control setting, indication of jumping, vibration setting, and pass sequence [1].

Operating parameters during measurements passes can have a profound effect on the generation of MVs. The amplitude, frequency, roller speed, and direction are aspects of operation that should be carefully set and monitored during measurement passes. Consistency with these aspects of operation allow for more accurate roller repeatability checks. Amplitude between 0.028 and 0.043 in with a tolerance of ±0.008 in, frequency between 28 and 32 Hz with a tolerance of ±2 Hz, and roller speed between 1.9 and 3.4 mph are recommended. Roller MVs should not be collected during startup, stopping and turning.
IC instruments on the roller must provide sufficient documentation during measurement passes. NCHRP recommends the following parameters be documented during measurement passes: MV, three-dimensional position with time stamp and GPS quality, vibration amplitude, vibration frequency, travel speed, direction, automatic feedback control setting, jumping indication, vibration setting, and pass sequence [1].

QC/QA Option 1

Option 1 utilizes IC to locate the least compacted soil locations. These locations are then tested using point measurement methods. The evaluation area is acceptable if the point measurements meet the required point measurement specifications. This option assumes that a positive correlation exists between soil compaction and MVs. Heterogeneous soils should be examined further to see if this option is appropriate.

QC/QA Option 2a

Option 2a is a comparison of the mean MV from two consecutive roller measurement passes. The percent difference in the mean (\(\%\Delta \mu_{MV_i}\)) between measurement passes is given by Equation 3. The QC/QA is accepted if the recommend percent difference in mean is five percent or less.

\[
\%\Delta \mu_{MV_i} = \left( \frac{\mu_{MV_i} - \mu_{MV_{i-1}}}{\mu_{MV_{i-1}}} \right) \times 100\%
\]  

(3)

QC/QA Option 2b

Option 2b is a comparison of the percent change in MV (\(\%\Delta MV\)) at a location between consecutive passes and requiring a certain percentage of locations to have a percent change lower than a set target value. Equation 4 provides the percent change in MV. It is recommend that between 80 and 95 percent of the locations have a percent change in MV that is less than two times the standard deviation of the percent change in MV. QC/QA using this option requires a process to transform the spatial MV data in to a comparable grid for each measurement pass. This process has not proven to be reliable and requires careful consideration of the methodology if utilized.

\[
\%\Delta MV_i = \left( \frac{MV_i - MV_{i-1}}{MV_{i-1}} \right) \times 100\%
\]  

(4)

QC/QA Option 3a

Option 3a starts with developing a correlation between point measurements and MVs on a calibration area. A minimum of five measurements should be taken for each compaction level—low, medium, and high. Generally, the coefficient of determination between the point measurement and MVs should be greater than or equal to 0.5 (i.e. \(R^2 \geq 0.5\)). When the correlation is established, MV corresponding to the correlated point measurement value is used to create a TV. Typically this can be achieved with a single-variable regression; however, multivariate regression may be necessary to achieve the required coefficient of determination to account for varying soil properties and different measurement depths. Acceptance is met when a
certain percentage of MVs in the evaluation area are equal to or greater than the TV. The recommend acceptance percentage is between 80 and 95 percent.

**QC/QA Option 3b**

Option 3b requires establishing a TV as the mean MV for the evaluation when the percent difference in MV between consecutive passes on the calibration area is less than or equal to 5 percent for 90 percent of the calibration area. Acceptance is met when the TV is achieved on the evaluation area.

**QC/QA Option 3c**

Option 3c is aimed at developing a correlation between laboratory soil property value, such as the resilient modulus ($M_r$) as a function of moisture contents and dry unit weights, and field-measured MVs on a calibration area. This option enables the establishment of a target MV using the laboratory soil test. First, a standard Proctor test is performed to determine the maximum dry unit weight and its optimum moisture content of the soil. Using this soil information, dry unit weight and moisture content ranges should be specified by the reviewing agency. Second, a series of laboratory resilient modulus tests are performed in accordance with the standard protocol used by the state agency at the specified range of moisture contents. A correlation of $M_r$ as a function of soil dry unit weight and moisture content is then established. Third, a relationship is established between field-measured roller MVs and moisture and dry unit weight that are determined using the spot test methods. Forth, a multiple regression model is established between MVs and the field measurements at the respective moisture and dry unit weight. This model would be used to predict the laboratory soil property values based on the field measurements. TVs are established based on the laboratory tests and desired soil properties. Pad foot rollers are not recommended for this QC/QA option [1].

**Implementation of QC/QA Specifications**

A literature review on current state DOT’s draft IC specifications indicates that state and local transportation agencies continue to require conventional compaction testing methods in addition to IC for roadway soil and pavement construction and QC/QA. Currently, 18 states have begun adopting draft QC/QA specifications and special provisions that will be reviewed for adoption into their standard specification manuals. Six states are opting to provide draft specifications for both soils and pavements, while 12 states are drafting specifications for either soils or pavements. More states are expected to begin drafting QC/QA specifications, with as many as 36 states having partaken in a field experiment or workshop regarding IC [5]. Figure 1 displays the types of QC/QA specifications drafted by states. These draft specifications range from special provisions to comprehensive specifications to be used for statewide roadway construction [7].
WYOMING AND NATIONWIDE SURVEYS

Two surveys were developed by the authors to understand the current knowledge of IC among professionals and how IC is being applied. The first survey was conducted in March, 2014 for public and private officials attending the Intelligent Compaction Data Management workshops sponsored by the FHWA in conjunction with the Wyoming Department of Transportation (WYDOT). The second survey, which has been developed but is pending data collection, is aimed toward state transportation officials to understand the role IC plays within different transportation agencies.

Wyoming Survey

The objective of the Wyoming survey was to understand how much knowledge private and public professionals practicing in Wyoming had about IC. The survey was conducted in a paper format and questioned respondents about their familiarity with IC, technical knowledge they may have, perceptions of IC, and their opinion about IC’s future role in Wyoming. There
were 79 total respondents, of which 69 were employed by WYDOT, seven by private firms, and three by local governments.

The survey results revealed that respondents were receptive to the idea of intelligent compaction but had a limited knowledge of and concerns about IC. Fifty-one percent of respondents said that they had heard of IC prior to the workshop. Of those respondents, 51 percent indicated that they had learned about IC from FHWA representatives or publications, 28 percent from their own research, 23 percent from contractors, and the remainder from other sources. However, only one respondent, a local government professional, mentioned that their agency had used an IC roller in the past. Specifically, their IC roller had been applied to landfill compaction rather than roadway compaction.

The lack of experience with IC rollers was indicated by the respondents’ desire to conduct a field demonstration. Fifty-eight percent of respondents indicated that a field demonstration would help them learn about IC, and 79 percent of respondents thought that a field demonstration would help facilitate implementation of IC in Wyoming. Among many concerns with IC, Table 3 indicates that the most notable concern was cost. Concerns about costs could be related to the limited amount of independent research conducted between the costs of IC compared to conventional compaction. Despite the concerns listed in Table 3, 70 percent of respondents thought that IC should be adopted in Wyoming while 26 percent were not sure and 4 percent did not respond.

<table>
<thead>
<tr>
<th>Responses</th>
<th>Percent of Respondents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost</td>
<td>33.3%</td>
</tr>
<tr>
<td>Reliability of data</td>
<td>26.4%</td>
</tr>
<tr>
<td>Not a specified quality control/assurance method</td>
<td>22.2%</td>
</tr>
<tr>
<td>Lack of operator ability and/or time and cost to train operators</td>
<td>22.2%</td>
</tr>
<tr>
<td>Unfamiliar with technology</td>
<td>20.8%</td>
</tr>
<tr>
<td>Reliability and durability of technology</td>
<td>19.4%</td>
</tr>
<tr>
<td>There are no concerns</td>
<td>19.4%</td>
</tr>
</tbody>
</table>

The survey reveals that the next toward implementation is providing an IC field demonstration to public and private sector stakeholders in Wyoming. The field demonstration should incorporate QC/QA methods used by other states and outlined in FHWA specifications to provide a tangible model for implementation of IC. A field demonstration can also be used by researchers to verify the results of other demonstration projects, advance the IC technologies, and explore new applications of IC.

Nationwide Survey

The nationwide survey of state transportation agencies was created to understand the agencies’ experience with IC. The survey is currently pending data collection. Results will be used to understand the steps agencies have taken to integrate IC into their agencies, what technical specifications of IC have been investigated, and what technical specifications may be
implemented in the future. The survey will also help WYDOT officials understand the steps taken by other transportation agencies to incorporate IC into their QC/QA specifications. The information from the survey can assist WYDOT decision-makers in implementing IC.

**ECONOMIC ANALYSIS**

The FHWA has cited reduced construction and maintenance costs as a feature of IC rollers; however, limited benefit-cost data is available to validate this claim [8]. This is echoed by Wyoming professionals, whom indicated that the cost of IC was their largest concern. To address the prominent concern of cost, a framework for an economic analysis was developed. This section contains a methodology to obtain cost data during two specific cost cycles: construction and roadway life. Definitions for each cycle are included below. The summation of the cycle costs are to be compared between two compaction methods: conventional compaction and testing, and IC compaction. For the purpose of this methodology, benefits are monetized as a cost savings or a comparative reduction in cost.

**Definitions**

The definitions below provide an outline for types of costs that would be defined within each cycle and type of compaction method.

*Construction Cycle*

The construction cycle includes the time period that begins with the preparations for conducting roadway compaction. This encompasses the procurement costs for rollers, including the rent or purchase of the roller and incremental roller maintenance cost. Labor costs for preparing the roadway section, GPS calibration, operating the roller, and conducting QC/QA are included. Costs to collect, store, and analyze data are also included. The cost of the equipment to perform QC/QA is also summed. The incremental cost of reusable equipment is calculated based on typical usage amounts in conjunction with depreciation formulas provided by the generally accepted accounting principles (GAAP).

*Roadway Life*

Costs during roadway life include rehabilitation costs over a determined life cycle. Rehabilitation costs include the costs for all types of maintenance and reconstruction of roadways. This does not include safety maintenance unless the maintenance involves a safety hazard caused by a devolved road condition due to inadequate compaction. Costs between compaction methods are to be compared with comparable life cycles.

*Conventional Compaction and Testing*

Conventional compaction means any method of compaction used by contractors to perform roadway compaction and subsequent QC/QA methods that does not use a roller equipped on-board stiffness or density measuring devices. QC/QA data is obtained by in-situ field tests.

*IC Compaction*
IC compaction means the compaction of a roadway section by use of a device, such as one defined in the FHWA QC/QA sample specifications for IC, attached to a roller that allows for the measurement of soil stiffness. Generally, this includes the use of an accelerometer, GPS unit, and on-board computer to aid roller operators in compaction efforts. QC/QA data is obtained from the roller and is analyzed by a QC/QA technician or engineer [9].

**Methodology**

The cost comparison between the two compaction methods is comprised of a summation of the costs from the two cost cycles. The methodology for each cycle is discussed below.

**Construction Cycle**

Based on the costs listed in the definition section, cost analysis of conventional compaction should be conducted based on several data points. These data points can be obtained from three methods: review of final construction expense report to governmental agencies, line-itemed expenses reports from construction companies, and a survey of companies for unit costs for those costs specified in the definition section. The three methods of data collection can be used for comparability. Review of final construction expense report data can be obtained from the contracts division of a governmental transportation agency. The agency archives should contain public records of the bids and total expense report for project contracted within its jurisdiction. Line-itemed expenses reports from construction companies would be obtained by contacting the companies and requesting their expense reports for specific projects. These data may contain more detailed information than their respective governmental contracts and can be used for more in-depth data analysis. Unit costs could be obtained by a phone or electronic survey of construction companies. The unit costs may be less project specific but easier to obtain, especially if construction companies are less willing to share line-itemed expenses.

If projects cost data from the aforementioned methods is selected for data analysis, the projects selected for each type of compaction should share similar design features. This will limit the amount of externalities arising from differences in design characteristics. After a sufficient number of data points are obtained for a certain type of design, different types of projects can be analyzed to understand their effects on costs between compaction methods. The costs for all aspects of a project should be brought to a determined date to calculate the net present value. A construction inflation index should be used in lieu of a consumer price index to obtain more accurate results. The results from this cost cycle would be most pertinent to contractors, whom would be more interested with upfront construction costs. Also, the results would help governmental transportation agencies understand if incentives are need during the construction cycle to promote use of IC, especially if there is increase of cost during the construction cycle that is offset by a savings during the roadway life cycle.

**Roadway Life**

The data points from roadway life must be obtained from projects that included each type of compaction. IC is a relatively new technology with most projects having been performed within the last decade. Due to this, there may be insufficient data to obtain comparable lifecycle costs of an IC roadway section to roadways that were conventionally compacted. In order to obtain comparable lifecycles, pavement condition performance curves can be developed for road
sections compacted with each method. The projected curves for pavement performance can be used to obtain the projected maintenance costs.

A pavement condition index (PCI) that accounts for distresses resulting from compaction quality should be used. The “Distress Identification Manual for the Long-Term Pavement Performance Program” developed by the FHWA should be used to determine the PCI [10]. Deduct values from cracking distresses should be used to calculate the PCI, and the pavement performances curves would be based on the PCI. Costs for road maintenance should be calculated when the remaining service life of a roadway is controlled by cracking or maintenance to mitigate distresses from cracking. The results from this cost cycle would be most pertinent to governmental transportation agencies interested in minimizing long-term maintenance costs. Costs should be calculated during similar roadway life cycles, or interpolated to create similar cost cycles.

Comparison of Compaction Methods

Comparison of the cost data should occur after the costs for each compaction method and cycle is calculated. The summation of the construction and roadway life cycles would be used to compare the difference in costs. Data regarding the mean, range, and deviation of costs can be calculated to understand if IC costs more or less on certain roadway designs or under particular road life conditions. This data can be used by public agencies and private firms to evaluate which circumstances IC may be appropriate.

CONCLUSION

The quality of a roadway is related to the quality of the roadway’s compaction. Conventional compaction and testing methods are insufficient for evaluating the compaction quality of a roadway section due to a lack of QC/QA testing coverage. IC provides a method to increase QC/QA testing coverage to 100 percent while providing more uniform pavement and soil compaction.

IC technology has been refined since the first IC roller was developed in the 1970s. Sufficient correlations between IC stiffness measurements and conventional testing exist to be able to implement IC for QC/QA purposes. Eighteen states, with support from the FHWA, have already begun drafting IC QC/QA specification, and at least 36 states have initiated the process of implementing IC by starting with workshops and field demonstrations.

A survey of Wyoming public and private professionals indicates that there is support for the implementation to IC; however, there are still concerns about cost and IC’s reliability. A majority of respondents indicated that a field demonstration would help facilitate the implementation of IC in Wyoming. A field demonstration is recommended in Wyoming in order to provide a tangible model for IC that can be used in the state. A pending survey of state transportation agencies also has the potential to facilitate implementation of IC in Wyoming by learning from other states’ experiences with IC.

Finally, very limited data about the comparative costs of IC to conventional compaction methods exist. Cost was also indicated as concern by many Wyoming professionals. A methodology for establishing costs comparisons was presented. Future development of cost comparison data can provide stakeholders, notably state transportation agencies and contractors,
with the ability to more objectively evaluate IC’s short-term and long-term cost effectiveness when making decisions on roadway compaction methods.
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Using Time-Domain Reflectometry to Monitor Subsurface Void Propagation

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ABSTRACT

Time-Domain Reflectometry (TDR) was originally used in geoengineering to locate caving heights above longwall coal mines. TDR is a geotechnical instrumentation technique utilizing a coaxial sensor cable grouted in place (usually vertically) in bedrock or soil, designed to deform or break as the surrounding material collapses or moves. The TDR cable is read using a reflectometer, and the resulting waveform can be used to determine the depth and severity of subsurface movement. Its use can also be extended to provide constant, automated data collection in locations where known potential sinkhole hazards exist.

In two case studies, TDR was installed to monitor subsurface movement along roadways where subterranean voids resulting from human activities threatened infrastructure at the surface. In the first case, a highway in Colorado suffered a sinkhole collapse into an abandoned railroad tunnel below. After repair of the sinkhole, TDR was installed to monitor other segments of the roadway that overlie the tunnel and could have similar collapse features. The second case involved an abandoned mining cavern below a developed area in the central United States. Each of these scenarios required a means to monitor the upward propagation of voids, and provide warning to the appropriate personnel in the event of subsurface collapse.

The TDR sensor cables were connected to an Automated Data Acquisition System (ADAS), which read the TDR cables, stored data, analyzed the data, and sent email alerts in the event of sensor movement. Each ADAS was accessible through the internet by an IP-addressable cellular telephone modem.
INTRODUCTION

The presence of subterranean voids caused by human activity presents a unique opportunity for geotechnical instrumentation. The potential hazard is known ahead of time and the scope of the potential collapse is better understood in man-made voids than natural sinkholes. Construction and mining records, previous geologic investigations, borelogs, and historical imagery all help delineate the expected scale of a geologic hazard. This allows the tailoring of a geotechnical instrumentation plan to efficiently monitor the propagation of subterranean voids.

An increasingly common method of monitoring the propagation of subterranean voids is Time-Domain Reflectometry (TDR) used in conjunction with an Automated Data Acquisition System (ADAS). TDR is popular in these systems because the material cost is relatively cheap and the monitoring of TDR after installation requires minimal field work. TDR is also popular because it can easily be scaled up - additional sensor cables take very little time to read manually, or can be read at little additional cost in automated systems through the use of a multiplexer.

This paper presents two case studies in which a man-made subterranean void presents a potential hazard to public safety, with an ADAS utilizing TDR to provide an early warning system. Such systems allow qualified personnel and agencies to monitor the propagation of sinkholes and take appropriate mitigative measures before failure occurs at the surface.

One such system is in place along US Highway 24 in central Colorado. In July of 2012 the roof of an abandoned railroad tunnel collapsed, causing the development of a sinkhole in the highway. The sinkhole was repaired and a system was designed to monitor ground movement at other locations where the tunnel passed beneath the highway. The second case study presented in this paper is at a decommissioned salt mine in the central US. The salt was solution-mined before the implementation of modern practices to minimize the risk of cavern propagation. Nevertheless, the property owner has decided to monitor the bedrock between the cavern and ground surface. The use of an ADAS allows for responsible agencies to make timely and well-informed decisions relating to road closures, evacuations, and mitigative efforts.

DEVELOPMENT OF TDR

In its broadest interpretation, Time-Domain Reflectometry is the measurement of the time it takes for the echo of an energy pulse to return to its source. Using a knowledge of the speed at which that pulse travels and the time it takes for the echo to return, the distance to the reflective surface can be calculated, Figure 1. Radar is an early example of TDR, in which an electromagnetic pulse is emitted, then an antenna listens for the echo of the pulse reflected off some surface, after which the distance to the reflected

![Figure 1. Reflectometer measures sensor cable and detects deformations.](image-url)
surface is calculated based on the elapsed time of transmission (Andrews, 1994).

In a geotechnical context, TDR generally refers to the application of TDR technology as a form of radar along a coaxial cable. (O’Connor, Dowding, 1999) The principle developers and users of TDR were in the telecommunications industry. TDR technology was originally developed in the 1950s to locate faults in power and telecommunication cables, and this remains the primary market for TDR hardware. In the 1980s TDR was first used in geotechnical engineering as a means of monitoring sag above longwall coal mines. Since then, its use has expanded to soil and rock slope monitoring and structural monitoring (smart-piles).

Physics of TDR

In geotechnical applications, TDR is used as “closed circuit radar” along a coaxial cable (Andrews, 1994). A coaxial cable contains an inner conductor and an outer conductor, separated by a dielectric material, Figure 2.

To read a TDR sensor cable, a fast rise time voltage pulse is introduced to and propagates down the cable, creating a current along the conductors. The primary characteristic of a cable that is analyzed in TDR is impedance. Impedance is the resistance of a circuit to current when a voltage is applied. The impedance is controlled by several properties of a coaxial cable, including the capacitance between the two conductors, the inductance of the cable, resistance within the conductors, and conductivity of the dielectric.

A time-domain reflectometer measures the impedance of the cable, which changes as the cable is deformed. The output of a TDR reading from modern reflectometers is a digital record of the reflection coefficient at different times (and, through an understanding of the time of transmission, the distance down the cable). According to O’Connor and Dowding (1999), the reflection coefficient, \( \rho \), is defined as

\[
\rho = \frac{V_r}{V_i}
\]

Where \( \rho \) = the reflection coefficient, \( V_r \) = voltage of the reflected pulse, and \( V_i \) = voltage of the incident step.

Figure 2. Cross-sectional view of coaxial cable. Note the inner conductor, dielectric foam, outer conductor, and sheath.
TDR data is generally represented by a graph of cable location versus reflection coefficient, Figure 3. The waveform is a direct measurement of the electrical properties of the sensor cable, which can be interpreted to tell how the cable (and soil/rock into which it is grouted) has been deformed since installation. When a sensor cable is measured, certain parameters, including the velocity of propagation, length of the cable, and desired point density must be input.

The waveform that results from a TDR reading indicates the reflection coefficient at different points along the cable. When the sensor cable is undamaged, every point along the cable should have the same reflection coefficient. The state of the downhole termination of the cable can be interpreted from the behavior of the end of the waveform. A clean, “open circuit” termination will result in the reflection coefficient approaching 1, while a short circuited cable (in which the inner and outer conductors are touching) will result in the reflection coefficient approaching -1. Figure 3 shows the TDR waveform of a sensor cable with a cleanly terminated downhole end.

There are a variety of reflectometers available for purchase due to the demand created in the telecommunications industry. The Campbell Scientific TDR100 is commonly used in geotechnical applications due to its high data resolution, reliability, and ease of integration into Automated Data Acquisition Systems.
AUTOMATED DATA ACQUISITION SYSTEMS (ADAS)

While TDR sensor cables provide useful data in manually read systems, constant monitoring of sensor cables is feasible due to the low power draw and digital output of modern reflectometers. An ADAS may be used when the site is remote and access by field personnel is limited, or when constant monitoring of sensors is desired. An ADAS requires several hardware components to reach full functionality: a weatherproof enclosure, a power source (usually solar), a datalogger, sensors and their respective interfaces, and a communication system. When properly integrated and programed, an ADAS is capable of reading sensors, analyzing data, storing data, transferring data to an outside database, and generating alerts. Alerts can be in the form of text messages, emails and/or acoustic/light alarm systems. An ADAS is highly customizable due to the variety of hardware and program commands that can be used.

The transfer of data from ADAS units to final storage locations (if the ADAS itself is not intended to be the final storage location) can be done through a variety of communication methods. Licensed and unlicensed radios, satellites, cell-phone modems, telephone lines, and other methods may be used to establish communication, depending on site accessibility.

COLORADO HWY 24 CASE STUDY

US-24 is a well-traveled two-lane highway that runs through the High Rockies in central Colorado. Approximately 9 miles North of Leadville, Colorado, Highway 24 crosses the continental divide at Tennessee Pass. The pass is on the Continental Divide at elevation of 10,424 feet and on the eastern side of the Sawatch Mountain Range. The geology in the vicinity of the pass consists of sheet-like drift from two glaciation periods overlying faulted bedrock that consists of sandstone, shale, conglomerate, quartzite, gneiss, and migmatite. The drift is comprised of clayey sand and gravel with a thickness greater than 100 feet in many locations.

At the top of Tennessee Pass, US-24 passes over two railroad tunnels. Due to the switch back alignment, the tunnels are directly below the highway approximately ¼ mile north of the pass summit in a perpendicular crossing as well as approximately 1,000 feet at the crest of the pass in an approximate parallel alignment.

The first tunnel was an unlined tunnel that was hand-dug in the 1880s by the Denver and Rio Grande Western Railroad. Throughout the operational life of this tunnel, the structure experienced regular issues with instability and seepage. A new concrete lined tunnel was constructed in 1945, approximately 100-feet to the west of the original tunnel, and the original tunnel was abandoned. The railroad still maintains interest in the right-of-way and existing tunnel, although the line is not in operation at this time. Both portals of the original (first) tunnel have collapsed as well as portions of the interior of the tunnel.
July 2012 Sinkhole

In July 2012, a void was observed at the road surface as a result of tunnel collapse approximately 90 feet below the road on the north side of the pass. The sinkhole was first noticed on the early morning of July 9 on the southbound shoulder. The initial size was 35 feet in diameter and 60 feet deep, Figure 5. The original estimated volume of the sinkhole was approximately 1,000 cubic yards. Within a few days, the sinkhole propagated to directly underneath the highway and continued to grow in size. As result the Colorado Department of Transportation (CDOT) elected to close a 4 mile stretch of US-24 for safety, resulting in a detour of greater than 40 miles. To stabilize the roadway, an emergency grouting project with a total cost of over $1.5M was required. The project took 23 days to complete with over 1,500 cubic yards of grout placed in 75 drill holes.

TDR Monitoring Program

At the top of Tennessee Pass, US-24 crosses over the abandoned tunnel again. During the 2012 sinkhole repair on the north side of the pass, borings were completed at the top of the pass to a depth of 175 feet where the original tunnel was intersected. A down hole video and laser imaging inspection was performed and revealed the tunnel was still open. However, evidence of

Figure 5. Sinkhole under US24 West of Tennessee Pass

Figure 6. Proposed TDR sensor cable locations. Yellow line denotes tunnel alignment.
collapse was present. Based on the down hole survey results, an estimated open tunnel volume of over 2,000 cubic yards remains. As a result, Colorado Department of Transportation (CDOT) was concerned future collapse could result in another sinkhole, this time at the top of the pass. CDOT is currently implementing a geotechnical monitoring plan designed to detect subsurface movement and trigger alerts before any voids resulting from the collapsed railroad tunnel can propagate to the surface. TDR sensor cables were installed vertically to a depth of approximately 175-feet (the tunnel is approximately 175-feet below ground surface) where the highway passes over the tunnel. A total of seven TDR locations were completed at the top of the pass and adjacent to the roadway in the summer and fall of 2013. Two baseline readings were taken before work was halted by snowfall. The locations of the boreholes are presented in Figure 6.

The boreholes were drilled by auger and air-rotary methods over a period of several weeks, Figure 7. There is an added challenge when installing TDR to monitor underground voids, in that drilling too deep may penetrate the void and result in lost grout and sensor cable. This occurred during the drilling of one of the boreholes, and a bentonite/grout mix was used to plug the bottom of the hole. Some boreholes also required casing to keep the hole from collapsing while drilling through unconsolidated material.

Figure 7. TDR sensor cable being lowered into a borehole.
ADAS Installation

The TDR sensor cable and ADAS installation is scheduled to resume in the spring, when access is improved. The TDR Sensor Cables will be connected to an Automated Data Acquisition System (ADAS). The ADAS will be programmed to collect data from the cables at regular intervals and communicate with the client’s server to provide real-time access to the TDR data. The ADAS will also be programmed to analyze the TDR data and generate e-mail alert messages in the event of readings indicative of subsurface movement. The ADAS is equipped with its own solar power system and a cellular modem on the GPRS network to allow internet access. A TDR multiplexer will allow a single TDR100 to read up to eight sensor cables in rapid succession.

The datalogger program is capable of analyzing the TDR waveforms to determine if significant deformation has occurred. This is custom-programmed based on each cable’s baseline reading. If the reflection coefficient of any point along the cable deviates significantly from the baseline reflection coefficient, an alert message is generated and sent to the responsible personnel.

Figure 8. Baseline TDR signature and approximate alert thresholds.
SALT MINE CASE STUDY

Introduction and Geology
The Salt Mine Case Study site is located in an active salting mining area in south-central Kansas. This area had solution salt mining activity beginning in the early 20th century and this method of mining is continuing in this area. The abandoned brine production well which is the subject of this case study was actively mined from 1919 until 1931, and was abandoned in 1939.

The geology at the site consists of alluvial sands (with some soil interbeds), which overlie shale, salt, and limestone of the Permian-age Sumner Group. These units are undeformed, with a gentle regional dip of a few degrees to the west.

Unconsolidated Quaternary alluvium, consisting of coarse-grained sand and gravel, with minor amounts of silt and clay, is the upper-most stratigraphic unit present at the site. This sandy alluvium is approximately 80 feet thick at the site. Potable groundwater is present within the alluvium, with the depth to groundwater ranging from ten to 20 feet below ground surface.

Immediately below the alluvium is the Ninnescah Shale Formation. This shale is typically gray or brick-red, with some thin beds of light gray and green shale, and very minor seams of argillaceous limestone and gypsum. The gypsum occurs as thin seams, laminae, and thin beds within the shale. The quality of the shale varies widely, from an extremely weathered, clayey shale to a relatively strong, dense, fissile shale. At the case study site, the Ninnescah Shale formation is approximately 180 feet thick.

The Milan Limestone Member of the Wellington Formation underlies the Ninnescah Shale Formation and consists of three thin dolomite/limestone beds separated by shale interbeds. The Milan Limestone Member is approximately 12 to 16 feet thick, and is more brittle and prone to fracturing than both the overlying and underlying shale units.

Underlying the Milan Limestone is the upper portion of the Wellington Formation. This portion of the Wellington Formation consists of soft, gray, calcareous shale containing gypsum, anhydrite, numerous thick beds of salt, and some thin beds of argillaceous limestone. The Hutchinson Salt Member of the Wellington Formation is the source for the salt that was mined at the case study site. The Hutchinson Salt Member consists predominantly of halite, separated by shale beds which range in thickness from laminae to more than ten feet. At this location, the top of the salt is at a depth of approximately 430 feet below ground surface and approximately 350 feet below the top of the Wellington Formation.

Solution Mining Description
The method used to solution mine is fairly simple. A borehole is advanced through the shallow alluvium and shale to the top of the target salt stratum. This borehole is cased through the alluvium and into the upper portion of the shale, although deeper portions might be left open. Tubing is then placed into the borehole and hot water is pumped down to the salt layer through this tubing. The salt is dissolved, creating a saturated brine, which then circulates back to the ground surface via the cased borehole for processing. Historically, a production bore was used until salt production dropped, which resulted in washing-out a cavern with little control over the resulting geometry. Many of these early caverns did not have salt roofs, but rather washed-out the salt layer to the bottom of the overlying shale. This results in an unstable situation, as the exposure of the shale to potentially undersaturated brine in the cavern could result in deterioration and slaking of the shale, and the growth upward of the cavern.
Modern solution mining methods result in more control over the cavern geometry within the salt layer. Kansas state regulations require that a salt roof be maintained in all solution mined caverns in order to ensure post-mining stability of the void.

**Case Study Site Investigation and Results**

The case study site was one of four locations identified for an intrusive investigation, which included drilling out the abandoned production casing and running a suite of geophysical logs, to include a sonar log if a significant cavern void was encountered within the salt. The drilling was accomplished using sonic water well drilling equipment. Geophysical logs were then run, to include a sonar log which allowed for the determination of the volume and geometry of the cavern void.

Based on the analysis of the sonar log, an upper cavern was identified between a depth of 426 and 493 feet below ground surface. The maximum diameter of this upper cavern was approximately 230 feet, with a maximum void height of 69 feet. The volume of the void space was approximately 22,500 cubic yards. The depth to the top of salt, based on regional mapping, is estimated to occur at a depth of about 430 feet below ground surface at the case study site. This suggests that the cavern void does not have a salt roof at this location and the void extends a few feet up into the overlying shale. These conditions, as previously discussed, could result in deterioration and slaking of the overlying shale. This would allow for the gradual propagation upward of the cavern into the shale, with the possibility that at some future date the cavern might migrate to the top of the shale bedrock.

Once the cavern breaks through, the unsaturated alluvial sand near the surface would flow as a slurry into the cavern, displacing the brine, and a sinkhole could develop at the ground surface. Sinkhole development similar to this have been documented in the general area where the case study site is located.

**Rationale for Installation of TDR Instrumentation**

Infrastructure at the ground surface at the Case Study location included roads and residences, so the ability to monitor this site was necessary. Various methods were evaluated, including both shallow and deep geotechnical sensors. Ultimately, due to the fact that the top of the cavern void was at a significant depth below the ground surface, the decision was made to use TDR in order to monitor the stability of the shale bedrock immediately above the cavern void. In the event of a failure, the client could then make a decision on whether it might be necessary.

![Figure 9. TDR sensor cables exiting borehole, and communication cables exiting conduit, prior to connector installation](image)
to open the cavern to re-evaluate the physical condition and geometry of the cavern void.
Description of Instruments and ADAS

Two TDR sensor cables were installed in a single borehole to a depth of 401-ft below ground surface, Figure 9. Generally, only one TDR sensor cable is installed per borehole, but this was a scenario where redundancy was desired, and increased cost to install two sensor cables instead of one was marginal compared to the drilling cost. Communication cables were installed in a conduit that connected the borehole surface vault to the ADAS enclosure approximately 40-ft away. The ADAS was comprised of a weatherproof enclosure containing a CR8000 datalogger, 7-amp-hr battery, solar panel and regulator, Raven XTV cellular modem, a TDR100 reflectometer, and SDMX-50 TDR multiplexer. The datalogger was programmed to read the TDR sensor cables every five minutes, analyze the signals, and store the reading if the analysis revealed that the signal had a significant deviation from the baseline, Figure 10. The datalogger communicated with the client’s server over the internet through the cellular modem, which was assigned a static IP address.

The ADAS can serve as the final storage location and data processing center for the instrumentation on site, Figure 11. In the salt mine project, the datalogger program was originally designed to act as the final storage location, data processing system, and origin of email alerts. After coordination with other shareholders, the data processing and email alert capabilities of the datalogger program were deactivated. Instead, the datalogger and modem act more as a conduit between the measurement device and a server located off site. The final storage, data processing, and alert generation responsibilities have been assigned to a program located off-site and in constant communication with the ADAS over the internet. This was accomplished by modifying the datalogger program and uploading the program remotely, so no additional field visits were required. The system in place runs automatically and will alert the appropriate personnel in the event significant changes to the sensor cables.

CONCLUSION

TDR is an effective tool to monitor subsurface voids. Ease of automation makes it the ideal instrument for geotechnical monitoring in situations where the cost of manually reading instrumentation such as inclinometers may be prohibitively high. There is also great potential for TDR in systems in which boreholes are drilled for investigations or installation of other instrumentation. In these scenarios, TDR sensor cables may be installed to provide additional data for minimal cost. Each case study presented is a good example of how TDR can be used to economically and reliably monitor collapse features over traveled roadways.
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Review of Bridge Approach Slab Systems in the United States

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Abstract

Approach slabs serve as a transitional system between an approach road and a bridge. Settlement of Bridge Approach Slabs (BAS) and their supporting backfill have been experienced by more than ten Departments of Transportation (DOTs) throughout the United States. According to current Wyoming Department of Transportation (WYDOT) inspection reports, BAS settlements occurred not only on existing bridges but also on newly built bridges that were just opened to traffic. These settlements typically create voids ranging from 6 to 12 in between the base of the approach slab and the geotextile reinforced backfill. The paper presents factors causing the BAS settlements and provides necessary design and construction recommendations. The paper also compares the capability of available retrofitting methods for the BAS settlement. A comprehensive literature review pertaining to approach slabs was performed to examine the research outcomes completed by more than ten states. The current specifications and standards on the BAS of the corresponding state DOTs were also evaluated. A nationwide survey is being conducted to fill in the missing knowledge identified in literature review. The available results of the survey will be presented in this paper. The paper reveals that poor construction practices, inadequate design of the backfill material, weather effects, and height of embankment fill are the most prominent causes of the BAS settlement.
1. Introduction

An approach slab serves as a transitional system between an approach road and a bridge. The prominent functions of the approach slab are 1) diminish the amount of differential settlement between a filled embankment and a bridge abutment, 2) span the void that may develop below the slab, 3) prevent slab deflection, which could result in settlement near the abutment, and 4) provide a better seal against water percolation and erosion of the embankment (Seo et al., 2002).

An approach slab settlement resulting in a "bump at the end of bridge" can lead to many negative effects on the function of roads, impacts on bridge and road structures, and subsequent increase in maintenance costs. Uncomfortable and unsafe driving conditions, road accidents, automobile damage, and damage to sensitive transported goods are few detrimental effects relating to the function of roads. The road bump creates repeated and increased impact loads from the dynamic impact of vehicles to bridge and road structures; thus, reduces the lifetime of the structures (Mahmood I. U., 1990, Kramer and Sajer, 1991, Zaman et al., 1991, Monely and Wu, 1993, Yeh et al., 1995, Hoppe, 1999, Seo et al., 2002, Luna et al., 2004, Marquart, 2004, Abu-Hejleh et al., 2006, Merrit et al., 2007, Helwany, 2007, Islam, 2010, Ma, 2011, Phares et al., 2011).

An extensive literature review conducted by the authors reveals that several factors cause the approach slab settlement (Zaman et al., 1991, Monely and Wu, 1993, Yeh et al., 1995, Seo et al., 2002, Ha et al., 2002, Cai et al., 2005, Mekkawy et al., 2005, Merrit et al., 2007, Helwany, 2007, Mekkawy et al., 2007, Greimann et al., 2008, Thiagarajan and Gopalaratnam, 2010, Phares et al., 2011, Puppala et al., 2011). These factors are categorized as follows:

a) Inadequate design and specifications leading to a high embankment fill, a steep side slope or a steep slab gradient
b) Poor construction practices, such as inadequate backfill compaction
c) Soft natural soil foundation or subgrade, compression of backfill materials or volumetric change of backfill due to an increase in moisture content
d) Weather effect that causes erosion of embankment
e) Other factors, such as high average daily traffic, high traffic load, or broken corbels.

The Wyoming Department of Transportation (WYDOT) has been using a geotextile reinforced backfill for the concrete approach slab for about 20 years. This approach slab system was developed based on a research project completed by Edgar et al. (1989) with the objectives of 1) reducing long-term maintenance costs as a result of excessive embankment settlement; and 2) alleviating the lateral load acting on the bridge abutment walls and lateral deformation causing expansion device closure. For many years, settlements of the concrete approach slab and backfill have been observed by WYDOT engineers and site personnel at new bridges that were just opened to traffic as well as older existing bridges. These settlements create typical voids ranging from 6 to 12 in between the base of the approach slab and the backfill as shown in Figure 1. WYDOT engineers have observed that settlements occurred at the approach roadway end as well as at the bridge end. These voids reduce the bearing support from the backfill material to the approach slab, and in several cases, cause damage to abutment corbels. On the entrance ends, the road bump due to the settlement creates a greater impact load to the bridge, resulting in increase
damage to joints and decks. According to current WYDOT bridge inspection reports, approximately 16% of the bridges have settlement issues, and many have yet to be discovered. Many retrofitting works on these problematic approach slabs have been performed by the WYDOT. The common retrofitting method is a total replacement while other least common methods are overlying of slab surfaces and lifting and realigning of slabs by filling and sealing the void space. Although these retrofitting works are typically carried out in conjunction with bridge rehabilitation projects, they impose a risk on the safety of the public using these infrastructures, continuously create challenges for maintenance crews, and increase the WYDOT road maintenance costs.

![A typical approach slab section](image1)
![6 to 12 inches of void beneath the approach slab](image2)

**Figure 1 – Settlement of a typical approach slab with 6 to 12 inches of void**

The objectives of this paper are to present those causes leading to approach slab settlement and compare suggested retrofitting methods based on past research conducted by more than ten state Departments of Transportation. The paper summarizes the outcomes of the comprehensive literature review conducted by the authors in a state by state basis. A nationwide survey on approach slabs and settlement is being conducted to fill in the missing information, in order that comprehensive recommendations can be developed. The paper presents the framework and methodology in developing this survey while a sample of the survey questionnaires is included in the appendix. The paper presents factors causing the BAS settlements.

2. **Literature Review**

A comprehensive literature has been conducted on 11 states including Wyoming that have experienced settlement in bridge approach slabs. The literature review was conducted based on published research reports and papers as well as relevant state DOT’s specifications on approach slabs.

2.1. **Wyoming**

Beginning in 1984, a research project was led by Edgar et al. (1989) to improve the performance of an approach slab with a reinforced backfill system using a continuous...
polypropylene woven geotextile fabric. A series of laboratory tests were conducted to examine several backfilling methods for the approach slab behind the bridge abutment.

After extensive laboratory studies, a field test program was conducted at the Ozone bridge on Interstate 80, located 25 miles west of Cheyenne, Wyoming, to validate the proposed methods concluded from the laboratory test program. Four embankments were reconstructed using different techniques for reinforcing the embankments and supporting the backfill soil at the abutment.

After instrumentations were installed, repeated initial measurement such as displacement was taken to establish the reference position of the embankments. The results from the field test program concluded that the geotextile reinforced embankment appeared to be an effective technique to control short term deformations. The technique of constructing a void between the reinforced embankment and abutment with a cardboard also appeared to be an easy and effective method of reducing lateral loads on the abutment. The significant findings of this research are summarized as follows:

a) The embankment constructed with the cardboard showed lower lateral earth pressures than the embankment constructed directly against the abutment;
b) The unreinforced embankment showed larger settlement compared to the reinforced embankments;
c) As a result of using geotextile reinforcement and applying the void between the backfill and the abutment, smaller lateral movements occurred under the roadway surface compared to larger lateral movements in the side slopes; and
d) It appeared that heavily reinforced concrete approach slabs were necessary to span the voids caused by differential settlement in the embankments.

Two types of approach slabs are suggested by Wyoming Department of Transportation (WYDOT) depending on the type of road surfaces. Concrete approach slab should be used when the approach roadway has a concrete surface and/or is in conjunction with a sleeper slab. Concrete with asphalt surface approach slab should be used when the approach roadway has an asphalt surface. In case of using asphalt overlay, the depth of asphalt should match that shown on the road plans. Otherwise, it should be one lift of 2 in. It should be considered that the depth of concrete slab plus the asphalt overlay, if required, shall not exceed the depth of the corbel.

The approach slab system is constructed using a backfill material, a geotextile, and an underdrain pipe as shown in Figure 2. Based on the research conducted by Edgar et al. (1989), WYDOT included a void between a bridge backwall and backfill in the bridge approach slab design. This void is used to reduce lateral pressure on the backwall, aid in drainage, and support the approach slab.
According to WYDOT specifications, the approach slab system generally depends on the depth of the abutment backwall. If the depth of backwall below the top of the corbel is 5 ft or less, a shallow configuration is used (Figure 3). If the backwall is greater than 5 ft, a deep configuration is used (Figure 4). The depth of the excavation and backfill shall be 2 ft minimum and 3 ft maximum measured from the bottom of the approach slab or sleeper slab at the back edge of the approach slab or sleeper slab.

Figure 2 – The approach slab system consisting of backfill material, geotextile, and an underdrain pipe (WYDOT 2010)

Figure 3 – Shallow Configuration (WYDOT, 2010)

Figure 4 – Deep Configuration (WYDOT, 2010)
2.2. Missouri

Missouri Department of Transportation (MoDOT) adopted a sleeper beam and approach pavement design in 1993 (Luna et al. 2004), in order to improve the design specifications of the approach slab. Based on this design the approach slab is supported on the abutment and the concrete sleeper beam on two ends.

Petry et al. (2002) conducted a research to identify, document, and prioritize geotechnical problems. The research concluded that settlement of bridge approach slabs was one of the geotechnical problems in Missouri. This research involved conducting a survey in 10 Missouri districts to evaluate the settlement problem of approach slabs. The results of this survey highlighted that high fill embankment; inadequate subgrade compaction, heavy traffic load, and inappropriate drainage system were the primary causes of the bump at the end of the bridge.

A survey conducted by Petry et al. (2002) in 10 Missouri districts found that all districts encountered bridge approach slab problems. This research revealed that the causes of bridge approach slab settlement are due to poor selection of backfill materials, inappropriate foundation soils, drainage problems, embankment erosion, and inadequate compaction of backfill. However, it was concluded that the main cause of bridge approach slab settlements was attributed to compression and consolidation of the embankment soil. In cases where bridge approach slabs were built on cut subgrade, rock cut, and/or have a thin fill height embankment, very little or no differential settlements were observed. On the other hand, those with thick fill height embankments and no sleeper slab drainages experienced differential settlements. Mudjacking was the common retrofitting method used by MoDOT for remediating differentially settled bridge approach slabs.

A subsequent research conducted by Luna et al. (2004) concluded that embankment fill material compression, settlement or creation of voids beneath the sleeper beam and approach slab as well as inadequate compaction of embankment soils were among the mechanisms for approach slab movement. Another research outcome was that the embankment and bridge approach slab was majorly affected by a construction sequence. Delay in the construction of bridge approach slab will avoid the consequence of not meeting a road grade. Consequently, the stiffened embankment becomes less compressible.

Thiagarajan and Gopalarthnam (2010) performed a research on the bridge approach slab with the objective of providing cost-effective structural solutions. This research involved conducting a survey from different states and a numerical modeling of the bridge approach slab. Three types of bridge approach slab (i.e., standard bridge approach slabs, modified bridge approach slab and bridge concrete approach pavement) used by the MoDOT were analyzed and studied. The cast-in-place approach slab, which considers supporting on an elastic soil, cuts the construction cost by 22%. The new recommended design retained the 12-in depth slab with a lesser reinforcement. The construction cost was reduced when the approach slab is assumed continuously supported on soil as well as by eliminating the sleeper beam system.

2.3. Iowa

White et al. (2005) completed a research to reduce bridge approach slab settlement problems by identifying the practices for design, construction and maintenance of bridge in
Iowa. Iowa DOT limits the height of compacted backfill layers to 8 in. The first layer from the bottom should be compacted to 90% of the maximum dry density and the following layers to be compacted to 95% of the maximum dry density. White et al. (2005) presented their findings as follows:

a) Voids developed under the bridge approach within one year from construction indicated the insufficient backfill moisture control/compaction;

b) Materials, such as flexible foam and tire joint fillers, are no proper for sealing the expansion joint;

c) Grouting does not appear to significantly prevent further settlement or loss of backfill material due to erosion;

d) Signs of distress and continued approach slab settlement were observed at several bridge sites with asphalt overlaying on the approach slabs; and

e) Compression of the embankment material or foundation was observed while monitoring the elevation profile of several bridge approaches.

Following these findings White et al. (2005), inspected eight new bridges that were under construction and found that these construction practices did not match with the Iowa DOT specifications (Iowa DOT). Using International Roughness Index (IRI) for evaluating the bridge profiles, White et al. (2005) observed that IRI values were time dependent and increased over time, indicating approach slab settlement, and the maximum IRI values were appeared at a transition between the roadway and the approach slab.

Subsequently, Merrit et al. (2007) conducted a research on a bridge approach slab using a Precast Prestressed Concrete Pavement (PPCP). The performance of the approach slab was improved by post-tensioning the slab in both directions to keep it in compression. This slab panel was acted as a “slab bridge”, which spanned over voids caused by the erosion or settlement of backfill soil. The PPCP system contributed many benefits, such as thinner slab section and rapid construction, to the design, construction and retrofit of approach slabs.

Greimann et al. (2008) involved in the instrumentation and monitoring of two different integral bridge abutment-to-approach slab connections over a year. The research concluded that the integrally connected approach slab-bridge system reduced the bump at the end of the bridge.

2.4. Colorado

Monley and Wu (1993) evaluated the effectiveness of application of tensile reinforcement in reducing approach slab settlement by using a finite element model. The results of the finite element model were verified by two large-scale bridge abutment tests. The research revealed that the approach slab settlements were caused by inadequate backfill compaction, poor foundation, high traffic loads, the development of high shear stresses between abutment wall and approach fill, and sloughing and erosion of the approach backfill. The large-scale test on the unreinforced bridge showed 6.7 to 7 in of settlement while the bridge with geogrid reinforced backfills showed a negligible settlement. The research also presented the following conclusions:

a) Placing a collapsible inclusion between the reinforced approach fill and rigid abutment, led to mobilizing the force to geosynthetics reinforcements; and
b) In cases where small movements of the fill were allowed after construction to mobilize the tensile force to the geosynthetics, small lateral movements observed in backfill.

Yeh and Su (1995) tested the effectiveness of using expanded polystyrene (EPS), flow fill and conventional backfill in reducing the bump at the end of bridge. These methods were tested in actual bridges. The most significant cause of approach slab settlement was attributed to the compression of the embankment and backfill. Among the three backfills, flow fill materials provided the best performance, followed by EPS backfill, while the granular backfill material contributed to the largest settlement.

2.5. Louisiana

Cai et al. (2005) carried out a finite element analysis to investigate the correlation between the deformation and internal force of an approach slab with the approach embankment settlement. This analysis demonstrated that as the differential settlement increases the contact area between the approach slab and embankment soil decreases. Consequently, the sleeper slab receives more portion of the load introduced to the approach slab, which yields in movement of soil toward the sleeper slab. This movement would increase the stress in the contact region, and as a result internal stress in the sleeper slab increases as well. By further settlement of soil, the approach slab loses its contact area with soil, and therefore, the soil beneath the approach slab will not affect the performance of the system. Evaluation of the contemporary approach slab design was another part of this research. Cai et al. (2005) proposed that the approach slab main reinforcement should be increased to #7 steel at 6-in spacing for 20-ft span length to improve the moment capacity of the approach slab.

Bakeer et al. (2005) evaluated a prototype semi-integral bridge and abutment system constructed in 1989 by Louisiana Department of Transportation and Development (LADOTD) to evaluate the performance of this bridge. Semi-integral bridges are modified in the design of integral bridges, in which joints are eliminated. The research highlighted that lateral earth pressure on the integral abutment was increased and permanently accumulated by temperature cycles, which also caused an excessive settlement of the ground surface adjacent to the abutments, leading to settlement of the adjacent pavement and voids under the approach slab. In congruent to the aforementioned problems, LADOTD proposed creating a gap behind the backwall.

LADOTD generally concluded that abutments that were casted integrally with the backwall and geosynthetic reinforced embankments showed satisfactory performances. Furthermore, LADOTD conservatively recommended small height embankments for future approach slab designs. Particularly, LADOTD lists the following design considerations:

a) Casting the approach slab with the bridge deck and backwalls as one integral structure eliminates the potential for abutment movement;

b) Using a geosynthetic-reinforced embankment with an MSE face creates a vertical gap that eliminates lateral pressure transfer to the backwall;

c) Using a sleeper slab and an expansion joint at the end of the approach slab eliminates the effects of seasonal thermal variations on the adjacent roadway pavement; and
d) Using a non-plastic material for embankment construction with the front segments of the top and bottom lifts filled with stone, provides an excellent drainage medium in the gap area behind the backwall.

2.6. Texas

Ha et al. (2002) conducted a research to investigate the settlement of BAS. This research performed in two parts: first part included survey and site investigation of current practice while the second part involved numerical analysis of the current design of approach slab. Parameters affecting the severity of the bump obtained from the survey are summarized in Table 1.

<table>
<thead>
<tr>
<th>Table 1 - Parameters affecting the severity of the bump (Ha et al. 2002)</th>
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<td><strong>More Severity</strong></td>
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<tr>
<td>• High embankment;</td>
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<tr>
<td>• Abutment on pile;</td>
</tr>
<tr>
<td>• High average daily traffic;</td>
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<tr>
<td>• Soft natural soil;</td>
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<tr>
<td>• Intense rain storms;</td>
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<tr>
<td>• Extreme temperature cycles;</td>
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<tr>
<td>• Steep approach gradients.</td>
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Two bridge overpass sites in Houston were chosen as part of the site investigation program to investigate the severity of road bumps based on two developed parameters, Bump Rate (BR) and Bump Index (BI). These BR parameters are simply defined by riding over the bump. The following recommendations are suggested after examining the current practice for planning, design, construction, maintenance and rehabilitation of the approach slab:

a) The specified that Plasticity Index (PI) for the soil within the 150ft of the abutment shall not exceed 15. Also, the relative compaction should be over 95% in this area; and
b) For closed, spill-through and integral abutments the approach must be built first, but for perched abutments the approach slab is constructed after the abutment is built;

Ha et al. (2002) performed a numerical analysis on the current design of approach slab to evaluate the maximum settlement of the pavement as a function of the length of slab. The analysis provided the following useful conclusions:

a) The slope between the abutment wall and the support slab is affected by the stiffness of backfill near the abutment,
b) Abutment wall causes a differential settlement as it prevents the adjoining soil to settle, but the soil further from the wall is not affected,
c) Lengths of sleeper slab and support slab decrease settlement, and the optimum length was determined to be 5 ft, and
d) Settlement of a 10-ft height embankment is 31 percent less than that of a 21-ft height embankment.

A single 20-ft long approach slab spanning from the abutment to the sleeper slab was proposed by Ha et al. (2002) as a result of the site investigation program and numerical analyses.
2.7. Oklahoma

Miller et al. (2011) conducted a research with the main objective of investigating the causes of bridge approach settlement through a literature review, direct field investigation, and laboratory investigations. They concluded that the causes of the bump at the end of the bridge were attributed to the settlement, erosion and height of embankments. The time dependent consolidation of foundation soils not the immediate soil deformation was considered as the primary factor of an approach slab settlement. Wetting-induced collapse due to the increase in the moisture content of the backfill or embankment soil yielded settlement. Furthermore, nondurable materials, collapsed under excessive compressive load and prone to slake in presence of water, caused settlement. Lastly, settlement was caused by poor surface and subsurface drainages. Erosion caused piping and suffusion of granular soils, which led to the creation of void in the embankment soil and resulted, in the settlement of the approach slab.

2.8. North Dakota

Marquart (2004) completed a research to replace backfill materials beneath an approach slab with a better material, since the conventional approach slab design does not shown to be effective. In the new design,

a) 20:1 taper has been carried form the abutment to the back until it intersects the pavement;
b) A void is installed against the abutment and a retaining wall was built against the void;
c) The reinforced backfill is compacted in 1 ft layers and is reinforced with geotextile; and

d) A drainage system is also provided.

The new approach slab design with a reinforced backfill proposed by Marquart (2004) did not show a success in reducing the approach slab settlement. The rate of settlement decreased but the settlement continued to occur. Mudjacking was used as a retrofitting method for the settled approach slab to bring back the slab to its original level.

2.9. Virginia

Hoppe (1999) studied the current practices of different DOT's in use, design and construction of approach slabs by conducting a survey. Some of the most important results of this survey are presented as follows:

a) Approach slab provides smooth ride to the bridge and reduces impact on the backwall.
b) Approach slab has initial high construction cost. Maintenance, settlement problems and difficulties with staged construction were among other disadvantages;
c) Twenty four percent of the respondents were concerned about the excessive settlement of the approach slab;
d) Fifty-seven percent of the DOT's used doweled or tied connection but no mechanical connections were used. However, according to 71% of the DOT's responses, mechanical connections were used in integral bridges;
e) The most common limiting requirement for fill specifications is the percentage of fine particles, in order to reduce the soil plasticity and enhance drainage properties. The passed percentage through No. 200 sieve is limited between 4 to 20% by various states;

f) Most states use an 8-in loose lifts of granular fill, which is compacted to 95% of the standard Proctor maximum dry density;

g) Plastic drainpipes, weep holes in the abutments, and use of granular materials are the most common drainage methods used by DOT’s; and

h) Fifty percent of the respondents revealed that the major problem was to achieve a specified soil compaction close to the abutment.

2.10. Wisconsin

Helwany (2007) completed a research to evaluate and compare the applicability and efficiency of two approach slab settlement mitigation methods used in four bridges in Wisconsin. These two methods were geosynthetic reinforced fill and flowable fill. Two of the four bridges were constructed on incompressible coarse aggregate soils, while the remaining two bridges were constructed over a compressible soil foundation. Helwany (2007) concluded that the granular foundation yielded less settlement. Moreover, side movement of the embankment caused by erosion led to the subsequent movement of backfill material. The research concluded that the mitigation methods are performing far better on uncompressible foundation. The flowable backfill was not suggested for small projects as it was not proven to be cost effective.

Oliva and Rajek (2011) conducted a research with the objective of improving the performance of approach slab. They investigated the effect of foundation and abutment settlements on approach slab rotation. Cracking, rotation and other problems related to approach slab were quantified in this study. They concluded that abutment height affected the approach slab settlement significantly. The strains within the approach slab and the recorded settlement were higher in high abutments.

2.11. Ohio

Islam (2010) an analytical simulation with the purpose of reducing bumps at pavement-bridge interface. Analytical simulations showed that a great approach slab deflection occurred when the soil beneath the approach slab moved away from it. The movement of soil reduced the bearing capacity of the slab. As another important step of the research, Islam (2010) calculated the bearing capacity and the introduced moment to the slab. It was concluded that the current Ohio Department of Transportation (ODOT) specification of the approach slab should be improved to avoid under designed approach slabs.

Phares et al. (2011) completed a research with the main objective to assist ODOT in developing better pre and post-construction strategies to avoid or minimize the approach slab settlement. Besides reviewing state DOTs as well as ODOT current design specifications, Phares et al. (2011) investigated the behavior and condition of bridges currently in service. Conclusions derived from this study are as follows:

a) ODOT has different definition and design for integral and semi-integral abutments. The design does not allow a complete connection between the abutment and
superstructure. Integral abutment system has shown to be more reliable by many researchers, because this system does not rotate when subjecting to live loads;

b) Improper design of the backfill material can lead to void creation under the approach slab. In the ODOT design the superstructure and approach slab are connected, leading to a translational movement of the superstructure to the approach slab;

c) One of the causes of collapsing backfill material is the existence of bulking moisture content, which should be unexceptionally avoided in all cases. Additionally, insufficient effort in compaction of the tested materials in case studies led to poor compaction results within 5 ft of the abutment; and

d) Soil erosion of embankment and approach slab, settlement/compression of embankment or abutment, differential vertical movements and differential horizontal movements are among the possible causes of bridge bump problems.

3. Nationwide Survey

The literature review summarized in Section 2 highlights a wide range of causes to the approach slab settlement, and it is difficult to rank them while different states use slightly different approach slab systems and have different traffic and environment conditions as well as construction practices. Furthermore, information about current DOTs specifications on the design and construction of approach slabs is not readily available online for review. To identify gaps in the body knowledge and populate relevant knowledge on approach slab from various states, a nationwide survey was formulated with 22 questionnaires as given in the appendix. The survey was developed using commercial online software, SurveyMonkey®. The survey is being distributed through the American Association of State Highway and Transportation Officials (AASHTO) to all DOTs in the United States. These questions were designed to gather important information on specific topics. The survey is presently conducted, and thus, results of the survey are not readily shared in this paper. The background and purpose of these questions are described as follows:

a) Question (2): Some states have conducted research on approach slab; it would be beneficial to build up a complete reference list of these research works, in order to avoid redundancy and to perform comprehensive literature review.

b) Question (3): Not all constructed bridges are using an approach slab system. It is asked to determine the usage of approach slab in their bridges.

c) Question (4): Integral and non-integral abutment design considerations are found to be one of the major differences in DOT's design specifications. It is known to be one of the influencing factors of the approach slab settlement.

d) Question (5): It is asked to confirm the type of approach slab systems used in each state or specified in their respective design specifications.

e) Question (6): It is an important question to determine the percentage of the approach slabs with settlement problems.

f) Question (7): One of the major ambiguous issues found in the literature was about the causes of approach slab settlement. It is important to identify these causes with respect to their design/construction practices as well as local environmental and traffic conditions.

g) Question (8): It is believed that different types of settlement will improve our understanding of its causes and potentially lead to better solutions for remediation and retrofitting.

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h) Questions (9) to (15): A detailed understanding of backfill specifications will enable us to compare and contrast different approach slab systems used by other states with that recommended by Wyoming DOT. Therefore, a good portion of the survey is dedicated to this matter.

i) Question (16) to (18): We hope that these questions will shed light on structural slab specifications, retrofitting methods, cost of retrofitting methods and current design specification used by each state that are beneficial to solve the settlement problem of approach slabs in Wyoming.

j) Questions (19) and (20): Since some state DOTs current specifications are not readily available online, these questions will allow us to obtain the current specifications and typical design of the approach slab system for completing the literature review.

k) Question (21): This question rates the satisfaction of the DOT’s on the current performance of their approach slab systems.

4. Conclusions and Recommendations

A comprehensive literature review on approach slabs has been conducted on 11 states to understand the causes of approach slab settlement and identify potential solutions for remediation and retrofitting. A nationwide survey pertaining to this problem is being conducted with the objective of filling in missing knowledge to further improve the understanding the problem and facilitate the development of remedial solutions. The following conclusions and recommendations are described as follows:

a) According to the reviewed literature many states have experienced similar approach slab settlement problems.

b) There is a wide range of actual causes of the approach slab settlement. Particularly, the paper reveals that poor construction practices, inadequate design of the backfill material, weather effects, and height of embankment fill are the most prominent causes of the BAS settlement.

c) There is no unique design specification for the approach slab in the United States.

d) Most approach slabs are built based on typical standard details, which are not designed for a specific individual bridge.

e) Backfill material and method of construction affect the settlement of the approach slab;

f) It is recommended that more comprehensive true-scale experimental studies to be conducted to confirm the research outcomes and field observations.

g) Although some numerical studies had been performed by researchers, they were not capable of simulating the actual outcome observed in the field.

h) The literature review reveals that poor construction practice is one of the main causes of approach slab settlement. It is recommended that the design and construction specifications be revised to address the challenge with quality control and assurance of field compaction of approach slab backfill, especially areas near and adjacent to a bridge abutment.

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- Journal

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  3. Cai C. S., Voyiadjis G. Z., Shi X. Determination of Interaction Between Bridge Concrete Approach Slab and Embankment Settlement. FHWA/LA.05/403. Louisiana Department of Transportation, 2005

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  1. Ma S. Bridge approach slab analysis and design incorporating elastic soil support. University of Missouri-Columbia, 2011.
- Website
5. APPENDIX

The 22 questionnaires in the nationwide survey are listed as follows:

(1) What is the name of your agency?
(2) Has your state conducted research on approach slabs? If possible, please provide the reference.
(3) What percentage of the bridges use an approach slab system?
(4) What percentage of bridges with approach slab system use an Integral abutment?
(5) Which approach slab system(s) are currently used in your state? (Choose all that apply)
(6) What percentage of bridges have approach slab settlements?
(7) What are the causes to approach slab settlement?
(8) What type(s) of settlements have you experienced?
(9) What type(s) of approach slab backfill are currently used in your state?
(10) Is select backfill material used beneath the approach slab? If yes, please indicate the section(s) describing the gradation in your specifications requested in Question 19?
(11) What is the typical Geometry Specification (Average Depth) of your backfill?
(12) Is a drainage system used beneath the approach slab?
(13) Is a positive separation between subgrade and backfill provided?
(14) Is in-situ density test performed on compacted backfill? If yes, what is/are the in-situ test methods?
(15) Are spacers being used between the backfill and the abutment wall to minimize the lateral load on abutment? If yes, what is the specification of this spacer?
(16) What is the typical thickness of the structural approach slab?
(17) What is the typical span length of the approach slab?
(18) What retrofitting method(s) are used for approach slab settlement? What is the average cost of each method per bridge?
(19) What is the current specification used for design and construction of approach slabs? (If possible please attach the electronic version or provide the online URL link)
(20) If you are using typical template drawings for constructing the approach slabs, please upload them into the following boxes?
(21) Are you satisfied with your current design or are you planning on improving it? Please provide any useful comments in this manner, in the specified box.
(22) Please provide your contact information if you wish to receive results of this survey.
The Anatomy of A Challenging Construction Project,
Extension of the Roadway Across a Celery Bog, West Lafayette, Indiana

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ABSTRACT

Rapid population growth of West Lafayette, IN, adjacent to Purdue University, required the city to widen the road across the Celery Bog from two lanes to four. Celery Bog is a marsh containing thick deposits of peat and marl that fill a large kettle lake of Wisconsin age. Distance across the bog was about 800 feet. An embankment was required to raise the elevation of the road and to extend it from two lanes to four. The embankment portion across the bog failed as a classic rotational slump during construction. Some concerns have arisen in retrospect regarding the thoroughness of the exploration program, laboratory testing prior to construction, earthwork placement, and construction supervision.

Next, to stabilize the bog crossing, grout columns were placed through the soft soils into the glacial till below. However, the subgrade mat below the pavement failed to transfer loads onto the piles, placed at six-foot centers, causing the road to develop a washboard effect. In an attempt to add stiffness to the subgrade the city repaved the road with an asphalt overlay but the washboard effect returned. As a last resort a bridge was constructed over the bog, which nearly doubled the cost of the road-widening project. Steel H-piles approximately 100 feet in length, were driven into the glacial till and gravel layer to support the bridge. The original cost of about $3 million was increased to about $12 million for the total road project extending about 6350 feet in length. Litigation between the contractor and the state highway department was conducted in an attempt to determine the cause of failures and the level of responsibility.

INTRODUCTION

Rapidly increasing population growth of West Lafayette, IN adjacent to Purdue University has required the city to keep pace with the expansion of roadways and other infrastructure. One such expansion of an existing road over a marsh containing thick deposits of peat and marl has become an intriguing engineering problem since its completion. An embankment was required to raise the elevation of the road and to extend it from two lanes to four. The embankment failed as a classic rotational slump during construction.

Some concerns, in retrospect, have arisen as to the thoroughness of the exploration program, laboratory testing prior to construction and the earthwork placement. Next, to stabilize the road, grout columns were placed through the soft soils into glacial till below. However, the subgrade mat below the pavement failed to transfer loads onto the piles causing the road to develop a washboard effect. Analyses of the failures related to road construction were conducted at Purdue University (Fairfax, 2007), but travel continued over the marginal quality road until a bridge over the bog was completed in 2011. A description of the construction problems prior to bridge construction is presented first in this discussion. Location maps are presented as Figures 1 and 2.
Figure 2.

Lindberg Road
West Lafayette, Indiana
Vicinity and
Topographic Map

Source: USGS 7.5 Minute Series Map, Lafayette West, Ind. Quadrangle.
SITE DESCRIPTION

Details for the site in West Lafayette, Tippecanoe County, northwest Indiana are presented in outline form below.

West Lafayette, Indiana (Tippecanoe County)
- Located on Wabash River upland
- Tipton Till Plain
- Kettle hole topography

“Celery Bog”
- 100 acres of marshland
- Highly compressible, organic peat and marl soils from 5 to 25 feet deep (Houghton Muck)

Lindberg Road
- Increased development required road improvement from 2 lanes to 4 lanes over a distance of ~6,350 feet from Northwestern Avenue to McCormick Road
- Crosses bog for a distance of ~800 feet

PROJECT DESCRIPTION

- Proposed to raise road grade ~3 feet and widen from two lanes to four yielding a width of 70 feet
- Six soil borings made initially ranging in depth from 15 to 45 feet in depth
  - Crushed stone base (1.5 feet)
  - Silty clay loam (4.5 feet)
  - Fibrous peat (12 feet) (density=9 lb/cf and w=564%)
  - Marl (6 feet) (density=21lb/cf and w=240%)
  - Glacial till below 24 feet
  - Sand and gravel layer found in one boring from 19-26 feet below surface

Based on these borings and on lab tests, several alternatives were suggested in order to minimize settlement of the new roadbed. These five alternatives are presented as Table 1 on the following page. Figure 3 shows the cross section and six boring logs taken where Lindberg Road crosses the bog. These borings were obtained at the outset of the project.
Table 1. Summary of embankment support alternatives.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Complete Removal</td>
<td>The most positive method of minimizing future settlement would be to entirely remove the peat and marl and replace it with compacted “B” borrow backfill. However, based on the depth of the compressible soils and the need to remove these soils below the water table, this alternative will not likely be economically feasible. In addition, this alternative would affect a larger portion of the wetlands during construction.</td>
</tr>
<tr>
<td>Surcharge</td>
<td>This alternative would consist of removal of the existing asphalt pavement and placement of engineered fill above the existing fill to a height suitable to achieve adequate consolidation of the compressible soils. While the embankment is compressing the underlying soils, the roadway could be reopened to traffic with a temporary pavement section.</td>
</tr>
<tr>
<td>Lightweight Fill</td>
<td>Various types of lightweight fill are available with unit weights as low as 480 kg/cu m. Some concerns that will require additional consideration for this option include: 1) the material weighs less than water, therefore, the backfill should be designed to resist the buoyancy effect; 2) fill placed outside of the existing roadway will likely settle differentially since no overburden is present at this location; and 3) a geogrid may be required to reduce the likelihood of differential settlement.</td>
</tr>
<tr>
<td>Stabilizing Columns</td>
<td>Drilled shafts filled with stone or grout could be drilled through the compressible soils to support the embankment. However, the spacing of the columns would likely be relatively close to obtain a bridging effect, which may make this alternative uneconomical. In addition, the certainty of execution of this alternative is relatively low since the confinement of the columns would be minimal.</td>
</tr>
<tr>
<td>Bridge</td>
<td>A bridge supported on deep foundations could be constructed over the compressible soils. However, due to the length of the bridge required, it may be cost prohibitive. Any compressible soils beyond the length of the bridge should be removed, so that long term settlement of the approach embankments does not cause an abrupt grade change.</td>
</tr>
</tbody>
</table>
PROJECT TIMELINE

- **September 1993:** Environmental Assessment and Preliminary Geotechnical Reports prepared by Project Manager number 1 and subsurface exploration by Drilling Contractor and Engineer number one.
- **January 1997:** Geotechnical evaluation provided by Drilling Contractor and Engineer number one
- **July 2001:** Earthwork begins under primary contractor. Surcharge method selected.
- **December 3, 2001:** Rotational slump failure occurs on north side of Lindberg Road through thickest section of the marl layer. Likely caused by rapidly placed or overly thick soil fill (power line poles).
  - **April 29, 2002:** Second failure occurs on south side of Lindberg Road through marl layer
- **April 2002:** Project Manager number two hired as replacement.
- **August 2002:** Geotechnical Consultant hired to obtain additional data and evaluate failure.
  - Findings indicated that failure occurred in thickest section of marl (12.8').
  - Concluded that Auger Cast Concrete Pile Program with a geogrid below base course suggested by Engineer number one would be satisfactory to stabilize area.
    - Piles should extend at least 9.8’ (3m) into silty clay soil.
    - Soils in vicinity were marginally stable and no soil filling should be accomplished without pile support.

**Additional Borings**

- **August 2002:** A national construction company was recommended to perform auger-cast concrete pile work by local experts.
  - **September 2002:** The national construction company makes proposal for Vibro-Concrete Column construction (VCC).
  - **September 2002:** The Prime Contractor requests proposal from a local construction company to accomplish column support.
    - **October 2002:** Local Contractor makes proposal for Augered Pressure Grouted Displacement (APGD) piles.
  - **December 2002:** Geotechnical Consultant number two reviews the VCC and APGD proposals, but both procedures are acceptable.
    - Concludes preference to national construction company’s proposal
- **Spring 2003:** Prime contractor selects Local contractor’s proposal using APGD; West Lafayette and INDOT approve. The APGD grout column procedure is provided in Figure 4 in profile view and in plan in Figure 5.
- **Spring 2004:** Project completed. Surface settlement begins almost immediately. Road resembles washboard.
  - Base course fails to transfer pavement loads to augered piles.
  - Pile system not extended for full length of bog yielding sizeable settlement on east side of construction project (50 feet short).
Figure 4. Layout of auger-grouted columns at 5 and 6 foot centers in the bog area.
Figure 5. Cross section showing Augered Pressure Grouted Displacement Pile.
- **Fall 2005**: City re-paves roadway in attempt to smooth surface. Hoped added weight would provide greater stiffness and accomplish load transfer from base course to columns.
- **Fall 2006**: Settlement continues. City monitoring settlement and will drill borings through road to determine grid material and composition of base course.

Figures 6 and 7 show the washboard effect of the road over the Celery Bog, and Figure 8 shows the top of an auger pile after the pavement and base material are removed. Note the lack of a geogrid and crushed rock base overlying the pile.

**LINDBERG ROAD COSTS**

<table>
<thead>
<tr>
<th>Original Cost</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Widen to 4 lanes</td>
<td>$3.3 million</td>
</tr>
<tr>
<td>(surcharge method)</td>
<td></td>
</tr>
<tr>
<td>Stabilizing columns</td>
<td>$2.9 million</td>
</tr>
<tr>
<td>(after soil failure)</td>
<td>6.2 million</td>
</tr>
<tr>
<td>Costs to repave, plus further study</td>
<td>$0.266 million</td>
</tr>
<tr>
<td>Total Cost, Dec. 2008</td>
<td>$7.47 million</td>
</tr>
<tr>
<td>Bridge cost (proposed)</td>
<td>$6.5 million</td>
</tr>
</tbody>
</table>

Anticipated Total Cost ~$14 million for job first estimated at $3.3 million.
Figure 6. Washboard effect on Lindberg Road, looking eastward.

Figure 7. Washboard effect on Lindberg Road, looking eastward, showing drop in pavement where grout columns end.
Figure 8. Top of auger pile with lack of specified geogrid and crushed limestone rock.
LESSONS LEARNED

- More extensive preliminary site exploration was needed
- Insufficient laboratory testing on soils prior to construction
- Stage construction was poorly done
  - Fill placed too rapidly (time pressures) and inappropriately (between poles)
  - Insufficient inspection of work in progress (City, INDOT, contractors)
  - Post-failure residual strength of soil made surcharge method impossible
- Project supervisor was poorly qualified for geotechnical work involved
- Local contractor was ill-prepared for such a difficult project
  - Was this cost effective?
- Loads did not transfer properly to pier system. Why?
  - Unsure about the nature of the fill or the geogrid material and unsure how fill was placed.
  - Pier system was not carried far enough to the east (50’ more needed)

From the fall of 2006 until 2009 travel on the Lindberg Road crossing of the Celery Bog was conducted over the wavy pavement that developed after the surface had settled over the muck and marl areas between the auger cast, grout columns. A dispute over final pavement for construction developed between the prime contractor and the Indiana Department of Transportation (INDOT). Although project construction had been supervised by the City of West Lafayette, because the contract was funded by a 90/10 ratio of state to local money, INDOT had prepared and administered the contract. The specific details about the outcome of this dispute have not been made public; it is generally believed that the prime contractor was fully compensated for work conducted on the project.

A solution to the poorly performing road across the bog came by way of Tippecanoe County, where the road is located. The county has a budget for bridge construction and because the road was no longer controlled by the State of Indiana, the Tippecanoe County government made funds available to cross the bog by way of a bridge. So, the outcome was to elect the final option of those procedures listed in Table 1: to build the road previously indicated, and construct a bridge over the bog.

BRIDGE CONSTRUCTION

In March 2009, the bridge construction option for crossing the Celery Bog was initiated. A geotechnical engineering consultant from Indianapolis was engaged to conduct a subsurface investigation for the site and to recommend the foundation support for the bridge. Subsequently six shallow borings (30 foot deep) were completed; three at each end of the bridge span. Following that work, nine deep borings were obtained at two hundred foot intervals across the bog between these two points. These borings were all drilled 120 feet, with the exception of one boring that extended to only 105 feet. Split spoon samples were taken at 2.5-foot intervals for the shallow borings and for the deep borings. Sampling was accomplished at 2.5-foot intervals until 20 feet, followed by 5-foot intervals to the bottom of the boring. Most of the borings encountered
fill below the asphalt pavement and granular base. The fill consisted of various soil types, including clay, silty clay loam, sandy gravel, clay loam sand and gravel and crushed limestone, which extended to depths of 13 feet. Except for the shallow borings drilled at the ends of the proposed bridge and the two deep borings adjacent to the ends, most of the borings encountered peat to depths of 15.5 to 22.0 feet below the pavement. Moisture contents in the peat varied from 150 to 500 percent, with organic contents between 65 and 90 percent.

Except for the borings at the two ends of the proposed structure, the majority of the borings showed very soft silty clay loam (locally known as marl), which contained organic material and extended to depths near the center of the project of about 37.0 feet. Below these materials, the typical stratigraphy consisted of generally stiff to hard loam (glacial till) with interbedded layers of dense to very dense sand and gravel.

Laboratory testing included: soil classification according to the FHWA classification, grain size distributions, Atterberg Limits, density, moisture contents, unconfined compression tests and organic content (loss on ignition). 204 samples were processed. Moisture contents were run on all the samples, 39 unconfined compression and density tests on till samples at depths ranging from 18 to 85 feet, four Atterberg Limit samples on cohesive soils ranging from 3.5 to 100 feet deep, with PIs ranging from 10 to 38, and organic contents on 16 peat samples. Dry densities ranged from 122 to 133 lbs per cubic foot and unconfined compression strengths from 3450 to 14590 lbs per square foot. Most strengths were in the 4,000 to 8000 lbs per square foot range.

Steel H-piles were recommended to support the end bents and interior piers for the proposed bridge structure. It was necessary to locate the new piles to avoid interference with the existing auger-cast piles placed through the peat bog in a previous attempt to stabilize the road. If any new pile encounters refusal on an existing auger-pile an additional pile will be required at an offset location. Pile tip elevations are to be determined by calculations and verified by pile capacity testing. HP 12x53 piles are recommended and are to be placed at least 3 pile widths apart in the pile cap. Hard driving conditions should be anticipated due to the hard glacial till (containing cobbles and boulders) and the dense to very dense sandy gravel or sand and gravel (also containing cobbles and gravel) located above the estimated tip elevations. All steel H-piles shall be fitted with pile shoes to facilitate driving the piles to proper bearing depths. The actual pile tip elevations are to be determined based on results of PDA testing. At least one PDA test shall be performed within each of the three different sections (end bent, intermediate and middle).

PILE DRIVING OPERATION

Pile driving was conducted in accordance with the recommendations provided by the geotechnical consultant. Thirty one pile caps were constructed to yield the two end bents and the interior 29 piers, each containing from 5 to 8 piles each. Pile lengths for all piers except for the bent locations on the east and west ends, ranged from 81 to 115 feet in length. Pile lengths for the bents were from 55 to 65 feet in length. A total of 12,475.2 linear feet of H-piles were driven on the project. The tip elevation for the piles ranged from 593 to 617 feet in elevation. The original ground elevation ranged from 683 to 688 feet and the bridge deck elevation is about 693
feet in elevation. In general, the pile tips were founded on the dense to very dense sand and gravel layer that had N-values ranging from 20 to 60 with some values above 40 for a drive of 2 to 4 inches. The piers are spaced at 48-foot intervals yielding a bridge length of 1344 feet. The total length of the reconstruction project was 1970 feet. The Lindberg bridge and road reconstruction was completed in the spring of 2012 and opened to traffic. The first attempt to widen the two-lane-wide Lindberg Road was begun in July 2001 and after several attempts to build the road over the Celery Bog was completed 11 years later. The cost to build the bridge totaled $4.5 million, less than the $6.5 million estimated in 2006. Figure 9 is a photograph of the completed bridge structure, looking to the southwest, showing both the north and south portions of the bog.

Figure 9. Photograph of completed bridge structure.
CONCLUSIONS

The expansion of Lindberg Road over the Celery Bog from a two lane, poor quality road to four lanes, including a bike and pedestrian pathway, developed into a challenging construction problem due to a series of complications. A soil embankment over the bog could have been successful if the fill had been properly placed to allow pore pressures to dissipate properly. A slump failure lowered the strength of the soil precluding any soil embankment work and made deep support for the fill necessary. Grout columns founded on hard glacial till would have worked well if the pavement loads had been properly transmitted to the columns from a stiffer base course. A thicker geogrid and crushed stone base would have accomplished this transfer, but an inferior substitute of a thin geomembrane and cohesive soil was employed instead. Poor construction oversight and lack of experience in this type of roadwork contributed to this failure. Finally, high quality exploration, design and construction procedures to build a bridge were established under the direction of the County Engineer’s office and a high quality roadway was finally completed. The cost of the project was greatly impacted from an initial cost of $3.3 million to $12 million to cross 800 feet of highly compressible soil and provide 6350 feet of total roadway.

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The Full Scale Mechanics of Slope Stabilization

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ABSTRACT

Slope stabilization systems that use flexible facings in combination with grouted anchors have been widely used to stabilize steep soil and weathered rock slopes for more than a decade. These systems have proven to be a very cost-effective solution, and have seen widespread acceptance. The tools used to design these systems have been based on small scale modeling and testing of individual components. Empirical evidence has shown that these design models are providing solutions that are technically sound.

The absence of scientific, full scale testing, however, has prevented full validation of the design tools. An extensive series of tests has now been performed to provide an in-depth look at the full scale mechanics of slope stabilization. The test series was based on the use of a simulated slope consisting of a large scale box that could be tilted to simulate a full range of slope angles up to 85°. Multiple soil types were tested in conjunction with a variety of different flexible facing materials. Instrumentation on the test box provided load information, and laser scanning of the slope surface provided detailed data regarding deformations of the soil.

This paper will discuss how this full scale test series has provided validation of the system dimensioning concept and the importance of load transfer within the system. It will also introduce new types of mesh and spike plates that offer solutions for a broader range of slope conditions.
INTRODUCTION

Within the overall context of geotechnical issues that designers must address in transportation corridors, slope stability problems are one of the most common. Stability problems can occur in all types of slopes (soil, rock, mixed soil and rock), and on slopes ranging from very steep to relatively shallow angles. These problems can result from a wide variety of conditions, including: water saturation, unfavorable rock discontinuity orientation, changes in slope geometry, earthquakes, failure of existing stabilization measures, and many others. Figures 1 and 2 below show examples of two different types of slope failures on highways.

Figure 1 – Shallow failure in soil slope

Figure 2 – Rock slope failure due to unfavorable stratification

SLOPE STABILIZATION OPTIONS

Just as there are many causes for slope stability problems, there are also many potential solutions for these problems. One common solution is to install a pattern of anchors in the slope and attach a facing material to those anchors. The anchors and facing work together to hold the slope
in place. The facing retains the surficial layer of the slope, and the anchors address any global stability issues while also transferring loads from the facing into the stable subsoil or rock. When designing an anchored system such as this, there are a number of options for facing materials.

**Hard Facing**

A hard facing typically consists of shotcrete or concrete. There are situations where a hard facing is an effective solution, but they do have limitations:

- **Rigidity** -- They are very stiff which can lead to issues such as: failure due to hydrostatic water pressure, failure due to dynamic loading (i.e. earthquakes), and cracking due to their inability to adapt to movements that may occur within the slope.
- **Slow installation** – Installation of a hard facing can be slow due to the need to coordinate concrete trucks on the job site, the time required for curing, and work stoppages due to weather issues (i.e. rain, high winds, etc.).
- **Poor visual appearance** – Appearance issues can be helped by the use of a sculpted face, but this drives costs higher.

Figure 3 below shows an example of a failure that occurred with a hard facing system. As shown in the picture, problems can result due to punching failure where the anchors are connected to the facing.

![Figure 3 – Shotcrete facing failure due to build-up of hydrostatic pressure](image-url)
Soft Facing

Soft facings are generally geotextiles, and are mainly for erosion control. Despite the fact that they are typically installed with the intention of preventing erosion they may in some cases be acting in a stabilization role. This can be as a result of unexpected slope instability, or an improper application of the material. Figure 4 illustrates an instance where a soft facing failed to provide stability to the slope when movement occurred. Failure occurred as a result of the fact that the material properties of the soft facing do not allow it to provide significant transfer of forces to the anchors. As a result, movements in the slope result in rupture of the facing at the anchor locations; and very little stabilizing effect is applied to the slope.

Flexible Facing

Historically, flexible facings have consisted of a variety of materials, including: lightweight mild steel mesh (i.e. double twist mesh or chain-link mesh of different wire diameters), mild steel mesh reinforced with wire ropes, and woven cable nets. Slope stabilization systems with flexible facings can be very effective and cost-efficient mitigation measures if they are properly dimensioned. It is important to bear in mind, however, that not all flexible facings are created equally.
The anchor spacing for these systems must be designed such that the facing is able to transmit the forces that develop in the surficial layer between the anchors. Inability of the facing to successfully transfer these forces to the anchors will result in a failure of the facing. As a result, systems that use facing materials with low force transfer capabilities must have a close anchor spacing in order to reduce the magnitude of the forces that will be placed on the facing.

The installations pictured in Figures 5 and 6 both used flexible facing materials that can be effective as part of a stabilization system. Both failed, however, as a result of improper dimensioning. The anchor spacing was not properly adapted to the force transfer capabilities of the facing. Movement in the slope progressively loaded the facings until they failed at the anchor connection points. Success could have been achieved in both cases had the system used either a facing with higher force transfer ability or a closer anchor spacing.

Regardless of whether a slope stabilization system uses a hard, soft, or flexible facing, success or failure of the solution is dependent upon one main characteristic – the ability of the system to successfully transfer forces from the slope to the facing to the anchors to the stable subsoil or rock.

STABILIZATION SYSTEMS USING HIGH TENSILE STEEL WIRE MESH

Beginning in 2000, a flexible facing consisting of a steel wire mesh made with high tensile strength wire became available. This high tensile mesh was specifically developed to stabilize slopes as part of a complete system. In order to eliminate weak points in the system, the connecting elements (Figure 7) were designed to provide a joint with the same strength as the mesh. Rather than use an off-the-shelf, steel plate, a spike plate (Figure 8) was designed to optimize the way forces transfer from the mesh to the anchors. The high tensile strength mesh
(Figure 9) was woven with a 0.118” (3mm) diameter wire with a tensile strength of 256 ksi (1,770 N/mm²).

In addition to development of the facing materials, a design tool was also developed. The design tool allows the surficial layer of the slope to be modeled, and calculates the anticipated forces that the slope imparts to the facing and anchors. This tool enabled designers to apply engineering principles and factors of safety to the design of the facing element of stabilization systems. The design tool was developed through testing of the material properties of the system components, and with laboratory-scaled slope simulations using an inclinable sand table. Successful field experience provided validation of the design tool; but full scale, scientific testing was not available.

The high tensile mesh flexible facing system has been very successful and widely adopted over the last 14 years. More than 6 million ft² have been installed throughout North America; and it has been used by many DOTs, FHWA, railroads, commercial and residential properties, and many others. Designers and end customers frequently choose it because it offers a number of clear advantages, including:

- Very cost effective in comparison with alternatives
- Allows vegetation to grow so the slope can have a natural appearance
- Allows freedom of anchor placement based on the slope conditions
- Provides effective static load transfer, and therefore high reliability

With millions of square feet installed and some installations more than 14 years old, this would appear to be a very mature system. Three primary challenges remained though:

1) Full scale validation of the design model
2) Comparison to other flexible facing materials to establish guidelines for the capabilities of a variety of potential facing materials
3) Expansion of the system to cover a broader range of slope conditions (see Figure 10)

![Figure 7 – Connecting elements](image1)
![Figure 8 – Spike plate](image2)
![Figure 9 – High tensile strength mesh](image3)

![Figure 10 – Range of slope conditions](image4)
FULL SCALE AND COMPARISON TESTING

A year-long research and development program was undertaken in cooperation with the Bern University of Applied Sciences (Bern, Switzerland). The goal of the testing was to measure and observe the behavior of different flexible facing materials using a variety of different slope materials and at variable slope angles. The challenge of creating a full scale test slope that would allow the desired frequency and flexibility of testing was solved through the use of a large steel box that could be lifted to different slope angles by a crane (Figure 11).

Figure 11 – Full scale slope mechanics test set-up

Details of the test set-up were as follows:

- Box dimensions – 40’ (12m) x 33’ (10m) x 4’ (1.2m)
- Instrumented anchor bars
- Load cells in boundary ropes
- Multiple soil types – rounded gravel ($\Phi = 33^\circ$), crushed gravel ($\Phi = 38^\circ$)
- Variable slope angle from 0° to 85°
- Multiple meshes and anchor plates
- Laser scanning to measure slope movement
In addition to several types of mild steel mesh, the test program considered high tensile steel meshes with a variety of wire diameters (all with tensile strength of 256 ksi [1,770 n/mm²]). The testing also included different sizes of spike plates in order to consider the enhanced stabilizing effect that could result from a larger spike plate.

Analysis of Real Sliding Mechanisms

The test protocol was to raise each trial slope in increments, and then perform a laser scan of the surface at each slope inclination. Analysis and comparison of the laser scan results allowed for detailed evaluation of the slope movements that occurred underneath the facing, and also provided a means with which to compare the performance of different meshes. Figures 11 and 12 show the laser scan results of the same slope (angle = 60°, Φ = 33°, anchor spacing 11.5’ [3.5m]), but with different mesh types. The coloring of the images represents the amount of displacement that occurred, with red indicating the largest amount of movement and yellow showing limited movement. The test shown in Figure 12 used a mesh with a larger diameter high tensile wire to provide a mesh with higher load transfer ability. Comparison of the two laser scan images shows that there is less movement of the slope in Figure 12, and therefore the heavier duty mesh is providing a higher stabilizing factor to the slope. Figures 12 and 13 use the same mesh, but are retaining material with different friction angles. Comparison of the images illustrates that there is less displacement of the higher friction angle material.
Validation of Design Model

The design model that was developed 14 years ago for the high tensile mesh stabilization system included a reasonable assumption regarding the development of zones of influence at the anchor locations, and the lateral influence that would be provided by those cones. This assumption has been borne out by field experience, but observation had never been possible in a controlled, full scale setting. This test program provided an opportunity to analyze the effect of pressure cones.

The installation method of the system calls for installing the spike plates at each anchor location with a small pre-tensioning force. The drawing in the upper part of Figure 14 below illustrates conceptually how this pre-tensioning creates a stabilizing force that radiates out a short distance from each spike plate. The design model assumes that this body of material being acted upon by the stabilizing force is stable, and does not need to be accounted for in the loads placed upon the system.

The full scale testing validates this concept as shown by the lower part of Figure 14. The laser scan shows that the body of material surrounding each spike plate exhibits very little deformation, and is therefore stable as predicted and assumed in the design model. Additionally, it shows how this stabilizing force reduces as it radiates from the spike plate – as is illustrated in the drawing by the force vectors getting smaller with increased distance from the anchor.

Figure 14 – Drawing and laser scan illustrating lateral influence of pressure cones
Further validation of the design model was provided by applying it to a number of test slope scenarios. Comparison of the design model output to the observed and measured behavior of the test slope showed a strong correlation. The correlation applied both to the effectiveness of the ability of the safety factors in the design model to provide a design that allowed only limited deformation in the flexible facing, and the ability of the design model to predict the actual failure point of the mesh by removing the safety factors.

**Analysis of Load Transfer Within the System**

Another valuable result of the laser scan data was to provide a detailed view of how force transfer occurs within the system. As mentioned previously, the success of any stabilization system is a result of its ability to successfully transfer forces from the slope to the facing to the anchors to the stable subsoil or rock. Analysis of the laser scan data provided:

- A real world view of how this force transfer occurs within the system
- A detailed look at where forces move through the system, and the critical locations where they are transferred from the facing to the anchors
- Analysis of the failure mechanisms of a variety of flexible facing materials

Figure 15 shows a graphical and real world, laser scan representation of the two critical points at which forces are transferred from the mesh to the anchors. The drawing on the right illustrates a typical slope stability calculation looking at a cross section of the slope. It shows two possible failure mechanisms of the mesh – a punching failure where the mesh meets the lower spike plate, and a tensile failure where the mesh transfers force to the upper spike plate. The laser scan on the left provides validation of the critical nature of this force transfer pattern. Further, the laser scan illustrates that there is no need for a left-to-right (laterally across the slope) calculation as forces clearly move up and down the slope.

**Figure 15 – Laser scan and drawing showing critical mesh-to-anchor force transfer locations**
Figure 15 illustrates the importance of understanding the key performance measures of the facing material. As is shown in both the drawing and the laser scan, the key characteristic of the mesh is its ability to transfer forces specifically in the area where it interacts with the spike plate. Overall longitudinal strength (the strength of the mesh when it is pulled lengthwise with a distributed force) is not relevant in the actual stabilization load case. Force transfer occurs strictly at the mesh-spike plate contact area, and is therefore a function of the interaction between the spike plate and the individual wires of the mesh with which it is in contact. This points to the primacy of individual mesh wire strength as the deciding performance factor of the overall system. The cumulative strength of the mesh wires that are in contact with the spike plate determines the ultimate ability of the system to successfully transfer the necessary forces. Factors that are external to the individual strength of the wires do not affect facing performance in this critical force transfer area.

Figure 16 illustrates how forces transfer within the system on a macro level. The natural upward radiation of forces follows along both the diamond shaped pattern of the individual meshes of the high tensile mesh, and also along the diamond shaped pattern of the anchor spacing. This is indicated in the laser scan image of Figure 16 by the lines between each anchor. These areas exhibit less slope deformation as a result of the higher forces being transferred through these zones.

Similarly, analysis of the pattern of mesh deformation shown in Figure 17 provides direct visual evidence of the pattern of force transfer within the facing on a micro level. Forces are not transmitted directly up and down the slope, but rather they radiate upward from each anchor to the two anchors located above it. The same force transfer occurs within the mesh itself. Each mesh diamond transfers force to the two diamonds above it to the 4 above those to the 8 above those and this continues up to the next row of anchors. Therefore, the facing material must be able to distribute forces not just up and down the slope, but rather along the diamond shaped pattern between adjacent anchor points. As a result, the diamond shape of the high tensile mesh opening is well adapted to distribute and transfer forces along the actual vectors that develop in the stabilization load case.
A MORE VERSATILE SYSTEM

In addition to several mild steel meshes, the test program included three different types of mesh made with high tensile strength wire:

- 0.078” (2mm) diameter wire (wire breaking strength – 1,200 lbs.)
- 0.118” (3mm) diameter wire (wire breaking strength – 2,800 lbs.)
- 0.157” (4mm) diameter wire (wire breaking strength – 4,900 lbs.)

The wire in all three of these mesh types had a tensile strength of 256 ksi (1,770 N/mm²), and all meshes had an opening size of 2.56” (65mm).

Also incorporated into the testing were two different sizes of spike plate:

- Small plate – 13” (330mm) wide
- Large plate – 26” (660mm) wide

Table 1 below summarizes the high strength mesh test results. In order to allow direct comparison of the mesh-plate combinations, this data is all based on testing performed on the same slope material (rounded gravel, Φ = 33°), and the same anchor spacing (11.5’ [3.5m]). The second row of data listed in the table is the design level inclination. This is not the slope angle at which the system failed, but rather a conservative design angle well before failure.

The final row of data (Stabilized Angle) represents the difference between the design level inclination and the friction angle of the test material. The specific value of the Stabilized Angle will vary depending upon the characteristics of the retained material and the anchor spacing, but the table below allows comparison of the performance of the different mesh-plate combinations.

<table>
<thead>
<tr>
<th></th>
<th>Light Duty</th>
<th>Standard Duty</th>
<th>Heavy Duty</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2mm mesh, small plate</td>
<td>3mm mesh, small plate</td>
<td>4mm mesh, small plate</td>
</tr>
<tr>
<td>Friction angle [°]</td>
<td>33</td>
<td>33</td>
<td>33</td>
</tr>
<tr>
<td>Design level inclination [°]</td>
<td>42</td>
<td>51</td>
<td>55</td>
</tr>
<tr>
<td>Stabilized angle [°]</td>
<td>9</td>
<td>18</td>
<td>22</td>
</tr>
</tbody>
</table>

Table 1 – Stabilized angle of different high tensile mesh and spike plate combinations

One of the goals of the test program was to expand the capabilities of the anchored mesh system to allow it to be a reliable and cost-effective solution for a broader range of slope conditions.
This includes both slopes that are more severely unstable (steeper slope angle, less stable slope material, larger unstable rock sizes, etc.), and also slopes with less unstable conditions (shallower slope angle, more stable material, smaller rock sizes, heavy duty erosion control, etc.). Figure 18 below illustrates a range of slope conditions spanning from stable to severe. Also listed are the associated mesh options now available for the range of conditions: Light Duty, Standard Duty, and Heavy Duty. The expanded range of mesh and spike plate options now allows designers to apply the benefits of the anchored high strength mesh system to a much more diverse range of slope conditions.

![Figure 18 – Applicable slope condition chart for the new, expanded stabilization system](image)

**SUMMARY**

The goal of the full scale test series was to analyze the behavior of a variety of flexible facing materials to gain a deeper understanding of how slopes interact with flexible facings on a general level, and to validate the key performance factors of facings that determine their ultimate success as part of an overall stabilization system.

Analysis of the resulting test data provided significant insight into these questions. The laser scan data provided detailed information about the way in which the slope materials behave and interact with the stabilizing system at different slope angles. The information gained in the test series included details about how the stabilization system behaves on both a macro and a micro level. The paramount importance of the force transfer ability of the facing material rather than longitudinal mesh strength was clearly established. In addition to defining the way in which forces radiate upward along the lines of the diamond anchor spacing, the importance of the
facing’s ability to transfer forces from the individual mesh wires to the spike plates was clearly shown.

The test series also provided full scale validation of the system design model, and has led to a system that is now applicable to a broadened range of slopes. Possible areas for further testing on the full scale level include: performance under dynamic load conditions (such as earthquake) and testing of other types of slope materials.
Evaluation and Stabilization of the Bret Landslide

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ABSTRACT

The Bret Landslide is located at approximate mile post 29.1 on United States Highway 14 in Big Horn County, Wyoming. The Landslide is an active portion of a much larger landslide which extends for over one thousand feet along the highway. This large landslide has been marginally stable since movement initiated in the spring of 1965 during construction to widen the highway along the route of an old wagon road. The Bret Landslide is approximately 600 feet wide and extends approximately 250 vertical feet from the highway down to Shell Creek at an average slope angle of 40 degrees. Several tension cracks have been mapped on the slope between the highway and the creek. The wet winter and spring of 2011 contributed to high runoff in Shell Creek which eroded the toe of the landslide resulting in an approximately 30-foot high over-steepened slope. After continued highway damage from landslide movement and several emergency attempts to stabilize the landslide, the Wyoming Department of Transportation (WYDOT) decided to explore options for permanent landslide mitigation. Numerous mitigation options were considered for geotechnical feasibility and constructability. Six options were further evaluated and presented to WYDOT for selection of the most feasible option. The preferred mitigation option was chosen and consists of micropiles and ground anchor tiebacks with highway reconstruction using a geogrid reinforced subgrade. The mitigation was designed to stabilize the highway considering the difficult site conditions. Construction of the mitigation began in April of 2014.
INTRODUCTION

The Bret Landslide is located along United States (U.S.) Highway 14 at approximate mile post 29.1 approximately 29 miles east of the town of Greybull in Big Horn County, Wyoming (Figure 1). Landslide movement at this location has been observed by the Wyoming Department of Transportation (WYDOT) since at least 1965 and numerous landslide investigations have been performed since that time. In the area of the landslide, the highway follows Shell Canyon in the Bighorn National Forest.

Figure 1 – Bret Landslide Location Map
Site History

The modern U.S. 14 generally follows a wagon road route used by settlers to travel from Fort McKenzie north of Sheridan to the Bighorn Basin near Greybull. The wagon road was widened to an auto road in the 1920’s. In the 1950’s and 1960’s further roadway improvements were completed. These improvements included significant cuts and fills to widen the highway, reduce curves, and flatten grades. During construction in 1965, at least eight major landslides occurred in a three mile section of highway (1).

The Bret Landslide is an active portion of one of these major landslides (Figure 2 and Figure 3). At the time of failure of the major landslide, movement was estimated at a maximum of approximately 3 feet per minute over a short period. A section of the partially completed highway was displaced approximately 200 feet downslope. Offset at the headscarp was reported as approximately 60 vertical feet. Relocation of the highway was deemed impossible. Due to the size of the landslide and the presence of surface and ground water, it was concluded that control of water was the most feasible solution to stabilize the landslide.

Figure 2 – Photograph of the 1965 Landslide
A single permanent dewatering well was designed and installed in 1971. A 20-inch casing was drilled through colluvial material, sandstone, and granite to a depth of approximately 212 feet. The dewatering well pumped approximately 400,000 gallons per day for three days after which 40,000 to 72,000 gallons per day were pumped to Shell Creek. The dewatering well is still in use today and appears to provide stability to the large landslide mass.

Regional Geology

The Bret Landslide is located within the Bighorn Mountain Range, an elongate dome about 100 miles in length and 30 to 35 miles in width. The axis of the range trends NNW to SSE. The basement rock is Pre-Cambrian granite that is exposed in the relatively flat topped area along the crest of the range to the east of the landslide.

Overlying the granite are the Cambrian Flathead Formation, Gros Ventre Shale, Gallatin Limestone, and the Ordovician Bighorn Dolomite. The Cambrian age units have previously been referred to as the Deadwood Formation. The Flathead Formation consists of a basal bluff to yellow-orange sandstone and conglomerate, a middle member of gray to green shale and interbedded thin limestone and sandstone, and an upper member of interbedded white to brown sandstone and gray to maroon shale named the Wolf Creek Member (2).

The Flathead Formation is overlain by the Gros Ventre Shale which is a gray to green shale with interbedded thin sandstone and limestone. Overlying the Gros Ventre Shale is the...
Gallatin Limestone and the Bighorn Dolomite. The Gallatin Limestone is a gray to tan limestone and the Bighorn Dolomite is a bluff dolomite and limestone.

Regional uplift approximately 5 million years ago caused the bedrock strata to dip to the west along the western side of the range at the location of the landslide. On the western flank of the range, at the mouth of Shell Canyon, strata dip steeply to the west into the Bighorn Basin.

Site Geology

The Pre-Cambrian granite is exposed in the canyon below the landslide due to erosion and down cutting by Shell Creek. The lower members of the Flathead Formation are present beneath the landslide and extending above the highway. The basal sandstone and conglomerate were identified during site investigation and contain boulders of Pre-Cambrian granite. The Gros Ventre Shale is present above the highway and this section of the canyon is characterized by extensive landslide topography. The Gallatin Limestone and Bighorn Dolomite are exposed in prominent cliffs at the top of canyon at an approximate elevation of 8,800 feet. Surficial soils extend from the dolomite cliffs to Shell Creek varying from a few feet thick to over 100 feet thick. The strata in the vicinity of the landslide strike approximately 140 degrees (azimuth) and dip approximate four degrees to the southwest, toward the highway.

GEOTECHNICAL INFORMATION

Site Observations

The first major investigation of the Bret Landslide was performed in 1996 and 1997 by the WYDOT Geology Program. This investigation consisted of mapping the extent of the landslide and performing a subsurface investigation. Several distinct tension cracks were mapped on the slope between Shell Creek and the highway pavement. Tension cracks were mapped along the highway over a distance of approximately 400 feet. Based on the investigation, installation of H-piles was recommended along the outside edge of the guardrail. Over 100 55-foot long size HP 12x74 piles were installed on approximately 3 foot center to center spacing. The pile driving summary suggests that approximately only 30 percent of the piles were driven to refusal. Pavement damage from landslide movement appeared to decrease after the pile installation.

During the wet spring of 2011 with runoff from a higher than average snowpack, Shell Creek further eroded into the toe of the landslide leaving an over-steepened slope approximately 30 feet high (Figure 4). Additional landslide movement caused tension cracks to again form in the highway pavement with cracking extending approximately 600 feet along the highway (Figure 5). Over 160 size HP 14x117 piles were driven along the guardrail at approximately 3 foot center to center spacing (Figure 6) to provide short-term mitigation to allow time for long-term landslide mitigation options to be evaluated.
Figure 4 – Over-steepened Slope at the Toe of the Landslide

Figure 5 – Pavement Distress at the Bret Landslide
Subsurface Information

WYDOT Geology has drilled at least 17 borings at various times between 1968 and 2012 in the vicinity of the Bret Landslide. Three inclinometers were installed in 2011 in the landslide. The subsurface material in the landslide consists of 15 to 100 feet of soft, silty clay with gravel, cobbles, and boulders overlying interbedded sandstone and shale. Underlying the sandstone and shale is granite at depths in excess of 90 feet below the ground surface. Soil overburden generally classifies as A-4 and A-6 material using the AASHTO classification system. SPT blow counts in the soil ranged from 4 to 100, however the high end of the range is likely from striking cobbles and boulders. Groundwater levels measured beneath the highway in 2011 were approximately 10 feet above the top of the sandstone. Figure 7 is a generalized cross section illustrating the subsurface conditions of the landslide.

Inclinometer readings from 2011 to 2013 indicate that the landslide is moving at a depth of 43 feet to 55 feet below the highway which generally corresponds with the depth of the top of the sandstone and shale. The rate of landslide movement has been relatively constant since the installation of the inclinometers indicating that seasonal variations in site conditions do not significantly affect the landslide in the vicinity of the highway.
ANALYSIS

Landslide Failure Mechanism

Based on the previous landslide damage, surface characteristics, and subsurface information, it appears that the landslide fails sequentially in 100 to 200 foot long increments below the road. The failures appear to be initiated by erosion of the landslide toe by Shell Creek, which destabilizes the lower slope section. Loss of confinement resulting from failure of the lower slope results in propagation of the failure surface up the slope to form a mid and upper scarp. Eventually the landslide terminates at the highway. Larger failures have occurred above the highway in the past, but these failures appeared be less active than the Bret Landslide at the time of the investigation.

Groundwater and seepage may also contribute to the instability of the lower section of the landslide. Photographs and mapping of the lower section of the landslide suggest that groundwater occasionally seeps from the slope near scarps. Groundwater does not appear to significantly affect the stability of the upper portion of the landslide as measurements indicate the level is at or below the failure plane directly below the highway.

Global Stability Analyses of Existing Conditions

Global stability analyses were performed to verify the landslide failure mechanism. Back analysis of the lower slope adjacent to Shell Creek relied on surface observations and estimated soil strength parameters to model a factor of safety (FS) of 1.0. The next failing segment of the
landslide was modeled by assuming loss of confinement provided by the lower slope. This analysis process was repeated at each section of the landslide until the tension cracks at the highway could be modeled and back analyzed to a FS=1.0.

MITIGATION OPTIONS

Based on site reconnaissance, geotechnical data provided by WYDOT, constructability, and global stability modeling, potential mitigation options were identified and evaluated. Most options were evaluated to provide a FS greater than 1.30; however, options with FS less than 1.30 were also evaluated. The options were composed of various combinations of the following elements:

- Geogrid subgrade reinforcement beneath the highway pavement
- Rockery toe buttress
- Lightweight fill consisting of expanded polystyrene (EPS)
- Ground anchor tiebacks with panels
- Soldier pile wall with ground anchor tiebacks
- Micropile and ground anchor tieback system

Additional options such as groundwater control and realignment of the highway into the cut slope were also considered but were not evaluated in detail. Groundwater control was not considered further due to the low groundwater levels measured directly beneath the highway and the difficulty in removing a sufficient amount of water to improve the stability of the lower section of the landslide. Also, the existing dewatering well is removing a significant volume of water in the general vicinity of the Bret Landslide but this does not appear to improve the stability of the landslide. Realignment of the highway into the cut slope was not further considered because of the history of landslide movement above the highway and the support required to maintain a FS similar to the existing condition. Maintaining the existing condition of the landslide and repairing the road was also considered; however, U.S. 14 is the primary route over the Bighorn range so there are minimal options for detouring traffic around any section of the highway that may fail catastrophically.

In addition to global stability modeling and constructability considerations, advantages and disadvantages to each potential option were evaluated along with the estimated cost of mitigation for each option based on similar projects and bid items. Table 1 summarizes the advantages and disadvantages of the elements previously identified.
### Table 1 – Summary of Mitigation Option Advantages and Disadvantages

<table>
<thead>
<tr>
<th>Mitigation Element</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geogrid Subgrade Reinforcement</td>
<td>- Relatively straightforward construction sequence</td>
<td>- Slight improvement in FS</td>
</tr>
<tr>
<td></td>
<td>- Entirely within WYDOT ROW</td>
<td>- Traffic impacts during all stages of construction</td>
</tr>
<tr>
<td>Rockery Toe Buttress</td>
<td>- Improve stability at toe of landslide</td>
<td>- Outside of WYDOT ROW</td>
</tr>
<tr>
<td></td>
<td>- Relatively straightforward construction sequence</td>
<td>- Requires access road on difficult terrain</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Potential for failure above buttress</td>
</tr>
<tr>
<td>EPS Fill</td>
<td>- Reduced driving forces on upper portion of the landslide</td>
<td>- Potential to destabilize slope above the highway</td>
</tr>
<tr>
<td></td>
<td>- Entirely within WYDOT ROW</td>
<td>- Slight improvement in FS</td>
</tr>
<tr>
<td>Ground Anchor Tiebacks with Panels</td>
<td>- Stabilize highway</td>
<td>- Requires access road to construct system</td>
</tr>
<tr>
<td></td>
<td>- Potential to lose ground from beneath panels</td>
<td>- Ground anchors require performance and proof testing</td>
</tr>
<tr>
<td>Soldier Pile Wall with Ground Anchor Tiebacks</td>
<td>- Stabilize highway</td>
<td>- Traffic impacts during all stages of construction</td>
</tr>
<tr>
<td></td>
<td>- At least one lane open to traffic during construction</td>
<td>- Requires relatively large drilled shafts socketed in granite bedrock</td>
</tr>
<tr>
<td>Micropile and Ground Anchor Tieback System</td>
<td>- Stabilize highway</td>
<td>- Requires access road to construct system</td>
</tr>
<tr>
<td></td>
<td>- At least one lane open to traffic during construction</td>
<td>- Ground anchors require performance and proof testing</td>
</tr>
</tbody>
</table>

After review of the project constraints, available geotechnical information, analysis, constructability, and cost, WYDOT selected a combination option consisting of the micropile and ground anchor tieback system, ground anchor tiebacks with panels, and geogrid subgrade reinforcement of the highway for the permanent landslide mitigation.

### MITIGATION DESIGN

After selection of the preferred mitigation option, detailed analyses were performed to design the micropiles and ground anchor tiebacks for the difficult site conditions. A typical section of the landslide mitigation is illustrated in Figure 8. The design process included consideration of the type and size of equipment that would be required to drill the various mitigation elements. It was essential to be able to construct the mitigation systems with relatively small equipment to avoid making large destabilizing cuts for access and a working bench on the landslide and steep slopes. The location of the micropile and ground anchor system was chosen to utilize a narrow natural bench on the slope to reduce the required size of cuts.

The micropile and ground anchor tieback system was designed to act similar to a soldier pile wall in that this system would stabilize the highway even if movement were to continue.
below the system. Stabilization of the landslide below the micropile system was evaluated; however this was not carried forward due to the difficulty of accessing the toe of the landslide. Thus, the micropile system was designed to stabilize the highway assuming that the lower portion of the landslide would continue to move toward Shell Creek. The load applied to ground anchors in the micropile system was optimized to provide resisting force to stabilize the highway without over-stressing the micropiles.

To further improve the stability of the highway, a second row of ground anchors was designed above the micropile system. Because the micropile system would provide support and confinement to soil in the area of the ground anchors, it was not necessary to include micropiles in this portion of the design. The ground anchors were located to avoid conflicts with the previously installed H-piles. A continuous facing consisting of shotcrete and steel reinforcement was designed to distribute the ground anchor loads to the landslide. A continuous facing was selected to provide a connection between the individual ground anchors and to reduce the potential for settlement that would cause a loss in tension and load in the ground anchors. A shotcrete facing was selected to reduce point loading behind the facing that is common with precast panels and because of limited access and difficulty of placing precast panels on the steep slope.

Movement of the highway is anticipated to continue after construction until the landslide fully engages the mitigation systems. To reduce the damage to the pavement during this time, a geogrid reinforced subgrade was designed. The reinforced subgrade extends approximately 4 feet below the top of the pavement and contains 5 layers of high strength geogrid separated by subbase material.
MITIGATION CONSTRUCTION

The construction contract was awarded for $6.6 million. The contract includes a total of 188 ground anchors and 347 micropiles. Work began in April of 2014 and is anticipated to last into September of 2014.

REFERENCES

Friction Losses in Tieback Anchors Used for Landslide Stabilization

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ABSTRACT

Tieback anchors are now used routinely to stabilize landslides affecting transportation infrastructure in the USA and abroad. It is generally understood by design engineers that friction loss over the unbonded length of a tieback anchor is inherent to the system, and such loss is permitted as long as it does not exceed a certain threshold, typically 20 percent of the post-tensioning load. However, recent experience from projects utilizing tieback anchors with unbonded lengths in excess of 150 feet has demonstrated that this criterion may be difficult to achieve, particularly in certain ground conditions. Furthermore, anchor friction loss, even if below the 20-percent threshold, has important implications for the stability of the slope that are often ignored in practice. This paper presents the results of a study in which friction loss is quantified from load test data for projects in a variety of ground conditions by means of a “wobble coefficient.” Factors that may influence the wobble coefficient are examined, and recommendations for addressing this issue in practice are presented. A recent instrumented load-test program for a landslide stabilization project in Southern California is examined, demonstrating that with appropriate monitoring controls, uncertainty with regard to friction loss can be minimized.
INTRODUCTION

Steel-strand tieback anchors have been used by the geotechnical community to support shoring systems for many decades, and more recently as a means to stabilize landslides, particularly adjacent to transportation corridors. For strand encased in grease-filled plastic sheathing, which is intended to allow the strand to elongate freely, some load can still be transferred by friction between the strand and the grease and between the grease and sheathing. The sheathing then transfers load to the surrounding grout and, in the case of ground anchors, to the surrounding soil or rock. The resulting decrease in load over the length of the unbonded zone is known as “friction loss.”

A widely used criterion for ground anchor acceptance based on load testing is that the measured anchor elongation must achieve a minimum value of 80 percent of the theoretical elastic elongation, or in other words the actual elongation can be up to 20 percent less than the theoretical value. This ensures that the majority of the load applied at the anchor head is reaching the bond zone behind the assumed failure surface, since load shed within the failure mass does not contribute to stability. This criterion acknowledges that friction loss is an inherent feature of the anchor system which cannot be eliminated completely, only minimized.

For shoring projects, the governing performance criteria is usually that the wall horizontal deflection must not exceed some threshold value, and the 80-percent criterion was developed as a “rule of thumb” that was found to satisfy this performance goal for most anchored shoring systems designed using apparent earth pressure diagrams to determine anchor loads. In most applications, a properly manufactured and properly installed ground anchor will easily meet the 80-percent elongation criterion. However, there have been several cases in recent years in which anchors being used for landslide stabilization and having unbonded lengths in the range of 80 to more than 200 feet have shown an unusually high failing rate with respect to the 80-percent criterion. This raises the question of whether the 80-percent criterion is applicable to all ground anchors or if it should be limited to anchors within a certain range of unbonded length. In particular, the question is whether friction loss increases with unbonded length to a point at which the anchor elongation can be expected to fall below 80 percent of theoretical elongation even if good construction practices are followed. If that is the case, acceptance criteria for anchors with long unbonded lengths, such as are now being used in landslide stabilization applications, may require modification from the 80-percent criterion.

In this study, tieback load test data from a variety of shoring and landslide-stabilization projects are analyzed to assess the influence of various parameters on friction loss. A case study is presented in which engineering controls are implemented to directly measure friction loss. The importance of considering friction loss in slope stability calculations is also examined.

ANALYSIS OF FRICTION LOSS

The following relationship is often cited (e.g., [1]) as an approximation of load \( P \) as a function of distance \( x \) from the anchor head:

\[
P(x) = P_0 e^{-(\alpha x + \kappa x)}
\]
Where $P(x)$ is the load in the anchor at distance $x$ from the anchor head, $P_0$ is the load at the anchor head (at $x = 0$), $\alpha$ is the cumulative change in angle of the strands (radians) from the stressing point to distance $x$, $\mu$ is the coefficient of angular friction, $K$ is the wobble coefficient of friction (radians per unit of length), and $x$ is the distance from the stressing point.

The first term in the exponent, $\alpha \mu$, represents the effect of cumulative change in angle over the length of the strand and is useful for calculating friction loss in precast-prestressing applications where the strand is supported at known locations in the structure (e.g., the strand is draped over supports in the concrete form). For such applications in which the angular change is intentional, $\alpha \mu$ can easily be evaluated incrementally over each support interval. For ground anchors the strand is intended to be perfectly straight and this term is not applicable. Equation 1 reduces to:

$$P(x) = P_0 e^{-Kx}$$

(2)

Unintentional curvature of the strand is expressed in terms of the “wobble coefficient,” $K$, which is the product of angle change per unit of length (e.g., radians/ft) and the coefficient of angular friction (unitless) between the sheathed strand and grease. The normalized load at a distance $x$ from the anchor head is given by:

$$\frac{p(x)}{P_0} = e^{-Kx}$$

(3)

Under a tension load, the theoretical elastic elongation assuming no friction loss is evaluated according to Hooke’s Law as:

$$\Delta_{\text{elastic}} = \frac{P_0 L}{AE}$$

(4)

Since $P$ varies over the unbonded length ($L_u$) due to friction loss, anchor elongation must be expressed in its integral form:

$$\Delta = \frac{P_0}{AE} \int_0^{L_u} e^{-Kx} dx$$

(5)

Solving the integral, the expected elongation with friction loss taken into account is given by:

$$\Delta_{\text{friction}} = \frac{P_0}{AE} \left[ -\frac{e^{-Kx}}{K} \right]_0^{L_u}$$

(6)

The “normalized elongation,” or ratio of elongation with friction loss taken into account (Equation 6) to the theoretical elastic elongation without friction loss (Equation 3), can be expressed by:

$$\frac{\Delta_{\text{friction}}}{\Delta_{\text{elastic}}} = \frac{\frac{P_0}{AE} \left[ -\frac{e^{-Kx}}{K} \right]_{x=0}^{L_u}}{\frac{P_0 L_u}{AE}} = \frac{\left[ -e^{-Kx}\right]_{0}^{L_u}}{KL_u} = 1 - e^{-KL_u}$$

(7)
Or expressed as percent elongation (EL):

\[
EL = 100\% \times \left[ \frac{1-e^{-KL_u}}{KL_u} \right]
\]  

Equation 8 provides a tool to evaluate the expected elongation, with friction taken into account, as a percentage of theoretical elastic elongation. The criterion for ground anchors under proof or performance load testing in the field is usually that \( EL \geq 80\% \) based on the recommendations of the Post-Tensioning Institute (2).

In Figure 1: Effect of wobble coefficient, \( K \), on the percentage elongation achieved during load testing for a range of unbonded lengths.

The percent elongation as calculated by Equation 8 is plotted as a function of anchor free-stressing length. (For the remainder of this paper, unbonded length is assumed to include the jack length during testing, i.e. free stressing length is equal to unbonded length). Three curves are shown, each representing a different value of \( K \) ranging from 0.001 to 0.005 radians per foot. The 80-percent criterion is shown for reference as the blue dashed line. Note that even for small values of \( K \), indicating a very straight hole and low friction loss, there is some maximum length beyond which the 80-percent criterion cannot possibly be satisfied.

![Figure 1: Effect of wobble coefficient, \( K \), on the percentage elongation achieved during load testing for a range of unbonded lengths.](image)

Theoretically, \( K \) is a function of two physical interactions: (1) friction between the grease and the steel strand, and between the grease and the sheathing, which are material properties, and
(2) the amount of curvature along the length of the strand. A theoretical or experimental evaluation of these interactions aimed at establishing numerical values of $K$ is not likely to produce a practical result. An empirical approach that could provide useful insight is to establish back-calculated values of $K$ from a database of field load tests on ground anchors. $K$ can be back-calculated from a proof or performance load test using Equation 8 if the unbonded length and percent elongation are known from the test. Because $K$ appears in the exponential term in the numerator and also in the denominator, there is not a closed-form analytical solution to Equation 8, and $K$ must therefore be established through an iterative solution approach. The author has developed a spreadsheet for this purpose that determines $K$ for a given set of results from an anchor load test.

The objective is to back-calculate $K$ from field load test data to establish (1) a range of “typical” wobble coefficients for routine projects and (2) the influence of various parameters on the wobble coefficient, such as anchor inclination, anchor manufacturer, soil/rock conditions, etc. The author has compiled a database of tieback anchor load tests used for shoring and landslide stabilization projects and back-calculated wobble coefficients for each test, which are summarized in Table 1. The full database of tests and calculations will be included in a forthcoming report to be published by the Deep Foundations Institute (DFI) Tiebacks and Soil Nailing Committee.

<table>
<thead>
<tr>
<th>Project #</th>
<th>Name</th>
<th># Tests</th>
<th>Avg. Free Stressing Length (ft)</th>
<th>Avg. Elongation</th>
<th>Min/Max</th>
<th>Avg.</th>
<th>COV K (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Togwotee Pass</td>
<td>20</td>
<td>149</td>
<td>83%</td>
<td>0.0005/0.0040</td>
<td>0.0026</td>
<td>39</td>
</tr>
<tr>
<td>2</td>
<td>Cuesta Grade</td>
<td>108</td>
<td>129</td>
<td>79%</td>
<td>0.0005/0.0111</td>
<td>0.0038</td>
<td>43</td>
</tr>
<tr>
<td>3</td>
<td>Anonymous</td>
<td>6</td>
<td>128</td>
<td>91%</td>
<td>0.0011/0.0022</td>
<td>0.0016</td>
<td>26</td>
</tr>
<tr>
<td>4</td>
<td>Anonymous</td>
<td>2</td>
<td>115</td>
<td>94%</td>
<td>0.0006/0.0013</td>
<td>0.0009</td>
<td>53</td>
</tr>
<tr>
<td>5</td>
<td>Anonymous</td>
<td>35</td>
<td>108</td>
<td>85%</td>
<td>0.0009/0.0057</td>
<td>0.003</td>
<td>34</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>171</strong></td>
<td><strong>127</strong></td>
<td><strong>82%</strong></td>
<td><strong>0.0005/0.0111</strong></td>
<td><strong>0.00294</strong></td>
<td><strong>46</strong></td>
</tr>
</tbody>
</table>

Table 1: Summary of tieback anchor load test database
RESULTS

Back-calculated values of $K$ from the project database are presented in Table 1 and plotted versus unbonded length in Figure 2. A general trend of increasing wobble coefficient with increasing unbonded length is observed. That is to say that the longer anchors in the project database were more likely to have greater total friction loss, but also greater friction loss per unit length over the unbonded zone. The author speculates that the primary factor influencing this trend is that the drill stem becomes more flexible as it is advanced further into the ground, making it more prone to deviating from a straight course when an irregularity is encountered. The chance that such an irregularity will be encountered are inherently greater in certain ground conditions, such as boulders in a soil matrix, bedding planes or foliation that are sub-parallel to the drilling direction, contacts between fill and native soil or rock, or any subsurface feature that leads to borehole deviation and therefore greater curvature over the anchor unbonded length.

The large scatter in the data set plotted in Figure 2 does not provide the chance to develop a statistically meaningful relationship between friction loss and unbonded length, which is not surprising given the inherent variability of the subsurface. Note that several anchors with unbonded lengths between 100 and 200 feet did satisfy the 80-percent criterion. Furthermore, the
limited size of the database does not allow quantification of the relative influence of parameters such as anchor inclination or ground conditions with statistical significance. Nonetheless, designers and contractors should keep in mind that several parameters are likely to influence the wobble coefficient even if the results presented here cannot quantify the corresponding amount of friction loss.

For example, placing the anchor in a relatively long hole that is inclined at a shallow angle will be difficult. Anecdotal reports from two state DOT landslide stabilization projects, Togwotee Pass in Wyoming and Cuesta Grade in California, indicated that under such conditions the contractor resorted to forcing the anchor into the hole by “brute force.” This procedure is likely to induce curvature in the anchor that becomes permanent when the anchor is grouted, leading to greater friction loss. A potential solution to this problem is two-stage grouting in which the bond zone is grouted and a small alignment load is applied to straighten the unbonded zone before it is grouted. Although simple in concept, however, this procedure is time-consuming and difficult to perform, and hence not often utilized in the USA.

IMPLICATIONS FOR SLOPE STABILITY ANALYSIS

Limit-equilibrium slope stability analysis is often conducted to estimate the factor of safety of a landslide or potentially unstable slope and to assess the increased factor of safety that would be obtained by installing tieback anchors. In such analyses, the engineer specifies a design load in each anchor, and the computer program determines the corresponding resisting force at the point of intersection of the anchor with the failure plane and the increased normal stress acting on the soil or rock at this point. The magnitude of the load in the tieback anchor has a direct impact on the factor of safety computed by the program.

When these anchors are constructed in the field, if the average friction loss over the unbonded zone is 20 percent of the design load, this means that only 80 percent of the load that was assumed to reach behind the failure plane in the analysis is actually doing so, and according to typical project specifications this is acceptable. What is often ignored in practice is that the factor of safety computed during the slope stability analysis is not actually being achieved in the field when this is the case. Project owners are likely unaware that the geotechnical report specifies a factor of safety for the slope, while the specifications effectively allow the project to be constructed with a lower factor of safety. To determine the actual factor of safety, the engineer would need to re-run the slope stability analysis using the actual load in the anchor determined by load testing.

A more practical approach would be to specify a design load that achieves the target factor of safety with friction loss over the unbonded zone explicitly considered. For example, if a 100 kip anchor load is needed to achieve the target factor of safety, the engineer should specify a design load for construction of 100 kips/0.8 = 125 kips. There is no rational basis for specifying a target factor of safety and then not enforcing construction specifications that achieve this factor of safety in the real system.

For steeply-inclined failure surfaces, such as those often encountered when a landslide headscarp intersects a highway or other transportation alignment in steep terrain, anchor loads
derived from an apparent earth pressure diagram are often greater than those needed to achieve a global stability target factor of safety and thus govern the design. In this case, the comparison should be made between the apparent earth pressure anchor loads and the anchor loads needed to achieve the global stability target factor of safety with explicit consideration of friction losses as discussed in the previous paragraph. In certain cases, global stability may become the governing design case when friction losses are considered whereas the apparent earth pressure case would have governed otherwise.

MINIMIZING FRICTION LOSS: A CASE STUDY

For critical life safety projects or routine projects for which the designer feels that the ground conditions or other factors may lead to unacceptably large friction loss, engineering controls can be implemented to reduce uncertainty with regards to the wobble coefficient. An example is described here.

As part of a recent slope stabilization project for an adversely-dipping, weak sedimentary rock slope in Southern California, two tieback anchors with unbonded lengths of 130 feet were performance tested to 133 percent of design load (pictured in Figure 3). Each anchor was instrumented with DYNA Force® elasto-magnetic sensors, manufactured by DYWIDAG-Systems International, mounted directly to the strands over the unbonded and bonded zones to measure the load-transfer between the strands and the ground and to determine where along the length of the anchor the load transfer was occurring. Furthermore, the equipment used to drill the anchor holes was equipped with a gyroscopic device that measures the deviation of the drill string from perfectly straight, and anecdotal reports indicated that the device measured less than one-percent deviation along the length of the drilled holes. The anchors were inclined at 45 degrees in order to intersect the potential failure plane, which was assumed to be inclined at a relatively shallow angle of about five to ten degrees. A crane-supported dolly was used to mechanically feed the anchors into the drilled holes, and the contractor did not report any difficulty or blockages during the placement.

Given the reported ease of installation and “straightness” of the drilled holes along with the relatively steep anchor inclination, it is expected that the 80-percent criterion would be easily satisfied for these anchors, and the instrumentation verified that this was the case.
Figure 3: Installation and load testing of tieback anchors for mitigation of a recent landslide in Southern California. Photographs by B. Turner 2011—2014.
CONCLUSIONS

For this study, the author reviewed data from several projects in which tieback anchors were used for either landslide stabilization or shoring. This study utilizes the "wobble coefficient" as a proxy to relate the amount of unintentional deviation from straightness in the anchor to the amount of friction loss over the unbonded length.

The data processed for the project show a general trend of increasing wobble coefficient with increasing unbonded length, meaning that longer anchors are more likely to fail the 80-percent minimum elongation criterion. While the author speculates that shallow anchor inclination and certain geologic conditions may lead to higher wobble, the amount of scatter in the current project dataset does not allow specific conclusions to be drawn about these effects. Nonetheless, designers and contractors should be aware that the probability of not meeting the 80-percent minimum elongation criteria goes up with increasing anchor unbonded length. In this
regard, the project geologist plays an important role in identifying conditions that may be conducive to high wobble during the subsurface characterization phase so that potential problems can be addressed during design instead of by litigation during construction.

Methods for directly measuring load transfer between the anchor and the ground are available, such as strain gages or direct-force measuring devices that can be mounted to the anchor strands along its length. Incorporating these measurements into the performance test(s) for critical projects or projects with a large number of anchors can (1) reduce uncertainty about where friction loss is occurring, which provides an opportunity to address the problem if excessive loss is occurring in a particular zone, and (2) allows for a direct measurement of the load transfer in the bond zone, which can be used to optimize the lengths of the remaining anchors.

Regardless of whether anchors pass or fail the 80-percent criterion, slope stability analyses that include the resisting force of the anchor should only consider the magnitude of load that is expected and specified to reach the bond zone. If the designer requires more resisting force to reach a target factor of safety, larger anchors or a higher minimum elongation criteria must be specified. Likewise, anchors that fail the 80-percent criterion do not necessarily have to be rejected outright if additional slope stability analysis considering the actual magnitude of load reaching the bond zone can be shown to still satisfy the target factor of safety when supplemental anchors are added. This recommendation is consistent with the PTI specifications (2) although it is not typically included in project specifications based on the author’s experience.

Since many anchors from the project database with unbonded lengths in the range of 100 to 200 feet satisfied the 80-percent criterion, the author does not recommend a modified minimum elongation acceptance criterion for long anchors at this time. More data will potentially allow provide the opportunity to draw quantitative conclusions about the influence of specific parameters such as anchor hole inclination and the manufacturing processes used by various anchor manufacturers. The author would be very grateful if practitioners with access to performance load test data that meet the needs of this project would submit the data for inclusion in the project database.

REFERENCES:


Change We Can Count On?

Evaluating the Merits of Traditional and Modern Methods in Conducting Preliminary Assessments of Geohazards

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ABSTRACT

A generation ago, preliminary geohazard potential assessments were performed by studying hard-copy geologic references available within consultant and government agency offices, bookstores, public libraries, and academic institutions. Limitations in the type and variety of geologic reference materials readily available to consultants and public officials meant the preliminary assessments were constrained to information contained in those references. Typically, aerial photography was obtained specifically for an individual project when warranted, and cost and schedule permitting. When available, the photography was viewed stereoscopically to substantiate potential geohazards identified by review of the geologic reference materials. Most importantly, the literature review and aerial photographic interpretation was field-verified by site reconnaissance.

Today, use of readily accessible images that are publically available through Google Earth, Bing, etc., combined with electronically archived publications, mapping, manuals, historical texts and photographs, and other aerial and satellite imagery, has expanded and expedited our approach to geohazard assessments in beginning project stages.

This paper starts with an overview of traditional methods in conducting preliminary geohazard assessments and discusses how those methods evolved into modern practices. The paper seeks to describe how various sources of information have become increasingly utilized while less emphasis is placed on more traditional methods. Using current technology, subsequent investigations such as site reconnaissances, subsurface investigations, and laboratory testing programs can become more focused, leading to more efficient and cost effective geotechnical studies. The merits and drawbacks of modern methodologies and information sources used for preliminary geohazard assessments are reviewed in comparison to traditional methods and sources.
INTRODUCTION

A number of variations on the definition of geologic hazard, or geohazard, are currently in use. For instance, The International Centre for Geohazards offers the following definition of a geohazard: ‘Geohazards are events due to geological features and processes that may pose threats to humans, property and the natural and built environment’ (I). For the purposes of this paper, a geologic hazard may be defined as an adverse geologic state or event capable of causing damage or loss of property and/or life. A plethora of diverse natural phenomena all fit the definition of geohazard, including but not limited to: landslides, volcanic eruptions, stream-bank instability, acid-bearing rock, migration of sand dunes, and soft, yielding soils. The topic of geohazards is therefore a very broad subject in and of itself. The methods of identifying and characterizing geohazards are equally abundant and diverse, due to the diversity of the geohazards being investigated and the limitations of available information and technology.

A primary function of the Engineering Geologist working in the planning and design (preliminary) stages of a project is to identify and characterize (assess) those geohazards within project limits that could have a potentially adverse impact on the design, construction, and/or performance of an engineered facility. This is primarily performed in order to reduce the risk associated with those geohazards. As part of this function, the Engineering Geologist is obligated to have a working knowledge of the most effective methods available for identifying and characterizing the geohazard of interest. A part of this knowledge should come from an understanding of how preliminary geohazard assessments have evolved into the current practice.

Geohazards and the methods employed in identifying and characterizing them constitute a vast array of subject matter that is beyond the scope of this paper to fully describe. Rather, an exploration of the topic is undertaken to provide an understanding of the manner in which preliminary geohazard assessments have evolved over time to current practice through examination of select preliminary geohazard assessment subject matter derived over the past 40 years. While some methods of conducting preliminary geohazard assessments have remained essentially the same over the years, others have either evolved or fallen completely out of practical use, relegated to brief mention in text books, field manuals, or symposia literature. An examination of the merits and drawbacks to these changes is presented.

CASE EXAMPLES OF TRADITIONAL GEOHAZARD ASSESSMENT METHODOLOGIES

Landslide Geohazard Susceptibility Study (1976)

In the mid-1970’s, a landslide susceptibility study was conducted for two thirty-mile sections of in-service cable utility corridor; one in southern Ohio and the other in the west-central part of West Virginia (2). As both areas were known from prior experience to be susceptible to landslides, particularly when disturbed by earthwork activities, it was determined to be cost preferential to conduct a landslide susceptibility study along the corridors to evaluate a more pro-active approach to maintenance by identifying landslide susceptible areas along the corridors to permit addressing of those areas before any slope failures could occur of a magnitude sufficient to take the utility out of commission.
**Preliminary Assessment**

As was customary at that time, a literature review was conducted prior to initiation of the project work using available geologic and geomorphic references to establish an understanding of the project geologic setting and geomorphic characteristics. In-house hardcopy references were reviewed to locate and evaluate the various soil and rock types in the study areas. The literature reviews established the geologic setting of the Ohio corridor area as an unglaciated terrain comprised of hillsides formed from the weathering of predominantly shale sequences, with colluvial deposits formed by the gravity transportation downgrade of residual soils. The structural geology of the Ohio corridor was revealed to not have a major influence on the landslide susceptibility along the corridor as the shale strata were generally flat-lying.

Similar to the Ohio corridor, the West Virginia corridor was found to be unglaciated and comprised of hillsides formed by weathering of shale sequences along with ‘red bed’ claystone sequences, all overlain by colluvial soils concentrated near the base of the slopes. However, the structural geology of the West Virginia corridor was found to potentially play a role in the landslide susceptibility due to an anticline intersecting the corridor roughly perpendicular to the route, introducing the opportunity for dipping sedimentary sequences unfavorably intersecting the corridor and any future cut earthwork.

The literature review for both the Ohio and West Virginia corridors was augmented by examination of United States Geological Survey (USGS) 7.5 minute quadrangle topographic maps covering the area traversed by the corridor routes. Based upon the review of the topographic maps for geomorphic evidence of past landsliding and landslide susceptibility along the corridors, preliminary landslide-prone areas were annotated on strip maps adapted from the USGS quadrangle maps (Figure 1).

![Figure 1 – Preliminary Landslide Susceptibility Map](image)

Color and color-infrared aerial photography was flown specifically for both thirty-mile corridor sections. After completion of the literature review and examination of the topographic
quadrangle mapping, an initial review of the color aerial photography was performed to confirm the results of the geologic literature review and topographic map examination and locate additional areas of potential landslide susceptibility. The results of this initial review were also annotated on the USGS quadrangle maps.

**Detailed Assessment**

It was determined that it would be time and cost prohibitive to perform a detailed analysis of the full thirty-mile section of each corridor route. Therefore, based upon the results of the literature review, topographic map study, field reports provided by the client’s field personnel, and initial review of the color aerial photography, a prioritized ten-mile section of each corridor route was selected for detailed landslide susceptibility assessment based upon the section of each corridor determined to encompass the most landslide susceptible slopes. Selection of each prioritized ten-mile corridor section was assisted primarily by review of the annotated USGS quadrangle maps.

The color and color-infrared aerial photography in each prioritized ten-mile corridor section was studied stereoscopically to further assess the presence of existing historical landslides and other contributing factors to landslide susceptibility. The study focused on landforms, drainage patterns, erosion characteristics, vegetation, culture and land-use, in addition to imagery characteristics such as color, tone, and textural variations. Photomosaic base maps were developed for each ten-mile corridor section and subsequently annotated with the results of the detailed stereoscopic examination. These results were used to classify the entire length of each ten-mile corridor section according to landslide susceptibility ranging from negligible through extreme.

**Field Reconnaissance**

After the various segments of each ten-mile corridor section in West Virginia and Ohio were given a landslide susceptibility designation, a field reconnaissance, or ‘ground truth’ was undertaken as a final stage in the landslide geohazard assessment. Importantly, the landslide geohazard field reconnaissance was performed by those same Engineering Geologists involved in the previous study activities ranging from the literature review to the detailed stereoscopic aerial photo examination. The field personnel were also trained in the identification of slope geologic/geomorphic characteristics indicative of ancient, now inactive slope movement along with other characteristic factors contributing to landslide susceptibility, including but not limited to hummocky/uneven terrain, colluvial soil lobes, disturbed or erratic stream courses, unusually or excessively wet ground, bent/deformed trees, seepage, weak and yielding soils, slope benches, vegetation favoring wet ground conditions, and slopes constituted of weathered claystone/clayshale bedrock.

**Landslide Geohazard Susceptibility Evaluation**

As a result of the field reconnaissance, the landslide susceptibility designations previously assigned to both ten-mile corridor sections were revised and either maintained, upgraded to a higher susceptibility designation, or downgraded to a lower designation. The final
mapping work product that was generated included the photomosaic base maps of each corridor section annotated with the results of the detailed stereoscopic examination, pertinent features observed during the field reconnaissance, and a final landslide susceptibility designation for the various segments of each corridor as informed by the results of the field reconnaissance. Practical recommendations for operation and maintenance of both utility corridors could also be presented. An excerpt from one of these photomosaic base maps is included as Figure 2.

Figure 2 – Photomosaic Map of Landslide Susceptibility and Contributing Features

Table 1 below shows the manner in which the various stages of the geohazard assessment effectively eliminated large sections of both utility corridors from further consideration. As seen in the table, the preliminary geohazard assessment was effective in eliminating large tracts of the utility corridors from the need for further geohazard study with further refinement occurring later on, particularly during the field reconnaissance.

<table>
<thead>
<tr>
<th>State</th>
<th>Preliminary Analysis</th>
<th>Detailed Analysis</th>
<th>Final Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Virginia</td>
<td>44.0%</td>
<td>19.9%</td>
<td>16.5%</td>
</tr>
<tr>
<td>Ohio</td>
<td>49.5%</td>
<td>19.8%</td>
<td>16.6%</td>
</tr>
</tbody>
</table>

The principal conclusion that can be drawn from the 1976 case study presented herein is that traditional geohazard assessment methodologies from a generation ago were indeed effective in identifying the geohazards of concern over significant stretches of terrain. However, part of the effort involved the time and cost associated with conducting aerial photography,
obtaining the results in the form of notoriously-sensitive film rolls, and the condensing and analyzing of the data. This level of effort to obtain aerial imagery is very often unfeasible from a cost and schedule standpoint, and requires the consultant to persuade the client to buy into conducting project-specific aerial photography such as that discussed herein. Furthermore, unless the film obtained for the project study is stored and archived properly, the imagery obtained for the study will eventually fall into decay and disrepair, precluding future utilization.

Urban Highway Corridor Rockfall Geohazard Susceptibility and Land-Use Study (1997)

An aerial reconnaissance was conducted in 1997 in support of preliminary geotechnical engineering to evaluate design alternatives for rehabilitation and upgrading of an urban highway corridor in the city of Pittsburgh, Pennsylvania (3). The urban highway corridor was characterized by dense residential, commercial, and industrial development dating back to the early 1800’s, an active railroad corridor, and various public utilities located on the river floodplain, all compressed between the Allegheny River and a moderate to steep or vertical slope reaching heights of approximately 100 to 150 feet above the river level, with additional residential/commercial development on the terrain above the slope (Figure 3).

Figure 3 – Circa 1988 Aerial Photograph of Urban Highway Corridor

Preliminary Geologic and Geohazard Assessment

Literature review performed for the preliminary geotechnical engineering effort revealed the project setting to be in the Pittsburgh Low Plateau portion of the Appalachian Plateau physiographic province, an extensive mature plateau of moderate relief. The project soils and geologic setting was characterized by urban fill soils underlain by Allegheny River alluvial deposits in the river floodplain area. Alluvial deposits were interlayered near the toe of the slope with colluvial deposits translated off of the slope during the Wisconsinan episode of the Pleistocene Epoch. Illinoisan episode terrace deposits formed part of the upper portion of the slope, locally referred to as the ‘Parker Strath.’ Bedrock geology consisted of relatively flat-
lying, Pennsylvanian-aged sedimentary sequences consisting of shales, claystones, and sandstones with lesser amounts of limestone.

The geologic history of the project area through which the urban highway corridor passed contributed to several geohazards identified as a concern during preliminary design of the highway project. Downcutting of the Allegheny River by high discharge of glacial meltwater during the last late-Wisconsinan glacial retreat resulted in formation of valley stress-relief joints in the slope bedrock parallel to the river valley axis. The morphology of these valley stress-relief joints is such that water accumulation in the joints results in lateral water pressure being exerted outward towards the river valley. This phenomenon, when combined with undercutting of competent bedrock (sandstone, limestone, sandy shale) units by differential weathering of weaker bedrock (clayshale, claystone, coal seam) units underneath, results in the creation of rock overhangs on the slope and incipient rockfall and topple scenarios. Colluvial soil materials formed by weathering of sedimentary bedrock units constituting the slope, followed by downward migration of those soil materials under force of gravity, presented landslide susceptibility geohazards above the urban highway corridor, exacerbated by uncontrolled runoff originating from the residential areas above the slope, accumulation of groundwater into the colluvial soil mass, and undercutting of the toe of the slope as a result of historical highway widening activities.

**Project Oblique Aerial Photography and Videography**

In April, 1997, a series of oblique, black and white photographs, color photographs, 35 mm color transparencies, and VHS videotape were obtained via helicopter by Graule Studios of Rochester, Pennsylvania. The photographs and videography taken were of the geohazard-prone slope beneath which the urban highway passed along with the surrounding immediate environs. A qualified Engineering Geologist, fully-apprised of the project geologic setting and suspected geohazards identified during previous investigations, including literature review, was on-board the helicopter during the reconnaissance mission. The principal set of oblique photographs were obtained in an overlapping fashion to permit splicing of adjacent photograph prints to create longer, continuous mosaics of specific slope segments and obtain a more comprehensive view of the slope. The individual pictures were enlarged and used to prepare photo mosaics included as accompanying documentation of the slope conditions in the study report. Select examples of the oblique project imagery obtained are presented as Figures 4 and 5 below.

It was found that the participation of the Engineering Geologist on-board the helicopter not only permitted the optimum oblique photograph angles to be obtained by the on-board photographer in coordination with the helicopter pilot, but also allowed for a bird’s-eye view of the subject slope to be observed and pertinent geologic and geomorphic features noted. Where specific project features of interest were observed during the flight, the Engineering Geologist could direct the helicopter pilot to return to those areas of interest to permit the photographer to obtain supplemental photographs. A more comprehensive and effective documentation of the slope conditions resulted, eliminating the need for costly subsequent flights to obtain necessary imagery omitted during the initial flight.
Figure 4 – Oblique, Black & White Aerial Photograph Depicting Rockfall Geohazards

Figure 5 – Oblique, Black & White Aerial Photograph Depicting Debris Slide Geohazard
Subsequent geotechnical studies benefitted from the aerial geologic reconnaissance and the oblique aerial photography derived from it. Study of the aerial imagery allowed for later field reconnaissances to be more focused on specific geohazard-prone slope segments, eliminating the need for field personnel to traverse portions of the slope. Given the oftentimes steep and/or heavily vegetated nature of the slope, the safety of the field personnel benefitted as well. Improvement and optimization of the planning and execution of drilling and testing investigations was realized, further benefitting the project cost/schedule and safety of field personnel.

MODERN PRELIMINARY GEOHAZARD ASSESSMENT RESOURCES

Advances in technology with the increased amount and ease of accessibility of aerial, satellite, and photographic imagery, in addition to geologic literature and mapping, have changed how preliminary geohazard potential assessments may be conducted. What follows is a discussion of select examples of modern resources available to support preliminary geohazard assessments.

Bing Maps Bird’s-Eye View

Bing Maps (4) is a Microsoft web mapping service currently available at www.bing.com/maps. Bing Maps is publicly accessible and provides any user the ability to access bird’s eye-type, color oblique aerial imagery principally from metropolitan areas as well as suburban and rural areas in the United States. The imagery can be viewed from different cardinal directions, permitting the user to view features from the perspective most suited to reveal features of interest. This imagery is similar in nature to that which might be obtained from low-flying aircraft such as the helicopter used in the aforementioned preliminary geohazard study. Examination of Figure 6 below reveals the obvious potential utility of web features such as Bing Maps in conducting preliminary geohazard assessments, where oblique aerial imagery is available for viewing from the PC in an office setting or in the field from a mobile device.

The principal benefit derived from use of a resource such as Bing Maps is the time and cost savings that can be realized by reducing or eliminating the need to conduct project-specific oblique aerial photography. Provided the web imagery reveals sufficient detail and depth perception to identify the geohazard(s) of interest for a project study, the web-available imagery may be used in lieu of or as an augmentation of project specific imagery. Alternatively, the web-available imagery could be used to aid and inform the planning of a traditional aerial reconnaissance/photography event. For areas where the web-available oblique aerial imagery is unavailable, preliminary geohazard assessment would still require a traditional aerial reconnaissance/photography event to obtain sufficient coverage of project area imagery from the desired perspective.

Caution should be exercised when accessing Bing Maps imagery. Usage of outdated imagery has the potential to misrepresent recently initiated or modified geohazards, leading to inaccurate characterization of geohazard potential and features. Web-available aerial photography cannot replace the trained eyes of field personnel in identifying and characterizing geohazard features observed on the ground in the field or from low-flying aircraft (as in the
Figure 6 – Example of Traditional Oblique Aerial Photography (Above) and Bird’s Eye View Aerial Imagery from Microsoft’s Bing Maps (4) (Below)
example cited above). Proprietary issues may also exist where imagery extracted from the Bing Maps platform is used as part of a for-profit consultant study or other purposes.

**Google Earth**

Google Earth (5) is a proprietary product of Google, Inc. currently available for download at [www.google.com/earth](http://www.google.com/earth). The publically-accessible freeware version of the product permits the user to view geo-referenced, superimposed satellite and aerial imagery from the interface of a virtual globe. The software allows the user to manipulate the imagery to be viewed from a variety of perspectives and directions, including oblique terrain viewing, and can be operated from the PC in an office setting or from a mobile device anywhere in the field. The satellite imagery and terrain morphology viewing capability of Google Earth renders it a useful tool for preliminary geohazard assessments, as illustrated in the project examples below.

**Rural Landslide Susceptibility Study Aided by Google Earth**

Google Earth was employed as a tool in conducting a preliminary landslide geohazard assessment for a confidential client in rural Southwest Pennsylvania in 2013 (6). Literature research from electronically available publications obtained from the web quickly revealed the project site to be located in mature, un-glaciated, dissected terrain formed into Permian-aged, mudrock and sandstone sedimentary sequences. Landslide mapping studies carried out by the United States Geological Survey identified recently active and ancient, inactive landslides at the site. Structural geologic mapping sources, also obtained in electronic format online, indicated structural bedrock dip outwards from the slope, yielding a potential condition for regional groundwater flow along secondary bedrock jointing to infiltrate and saturate the colluvial soil envelope.

A field reconnaissance was determined to be appropriate to identify specific problem areas worthy of further investigation by drilling, sampling, and testing. Prior to conducting the field reconnaissance, Google Earth imagery was viewed to identify portions of the project site where additional emphasis was to be placed during the field reconnaissance. As seen in Figure 7, the slope morphology and oblique depth perspective offered by viewing the Google Earth imagery allowed for identification of potential active and inactive slide areas, potential high seepage areas and drainage features, land-use, vegetation cover, and portions of the project site less prone to slide activity. Utilizing the information obtained from study of the Google Earth imagery, the goals of the geohazard field reconnaissance could be prioritized, allowing for time to be saved in the field traversing and viewing portions of the project site lacking those factors known to contribute to landslide susceptibility. Usage of the Google Earth imagery to corroborate historical landslide mapping, verified by field reconnaissance (Figure 8), allowed for various portions of the project site to be prioritized, optimizing the drilling, sampling, and testing investigation that was carried out at a later date. In addition to cost and schedule savings, reduced impact and footprint to the project site were realized as an end result of the more focused subsurface sampling and testing investigation.
Figure 7 – Google Earth (5) Terrain Image for Rural Landslide Susceptibility Study

Figure 8 – Annotated Field Reconnaissance Plan of Rural Landslide Susceptibility Study Area (6)
Additional Google Earth Functionality and Discussion

In addition to its primary functionality, Google Earth offers other features that are pertinent to preliminary geohazard assessments. For instance, Google Earth Street View (Figure 9) permits the user to view panoramic photographic imagery taken at ground-level from most major roadways as well as smaller roadway designations. With substantial coverage in the United States, as well as partial coverage elsewhere around the globe, this imagery application may be applied to conduct preliminary geohazard assessments, provided the features of interest are visible and in sufficient detail from the perspective of the roadway. The ability to essentially visit a site of interest remotely allows early identification of features of interest that can be documented and reported directly or investigated in greater detail during later in-person field reconnaissance. However, the existing roadway perspective offered by Google Earth Street View would be of limited utility for roadway projects being planned off of the existing alignment. Historical satellite imagery available from Google Earth can aid in the investigation of former land-use, giving the user the ability to document temporal changes in land-use over the time period covered by the imagery available for a particular area. This rapid ability to view and document temporal changes in land-use and road network development, along with faster-paced geologic processes such as stream bank migration, landslide activity, sinkhole development, shoreline erosion, etc. is of particular use in preliminary geohazard assessments.
As with Bing Maps imagery, caution should be exercised when accessing Google Earth imagery for preliminary geohazard assessments, and for many of the same reasons. Outdated imagery, even one to two years in age, will not depict recently initiated or modified geohazards or pertinent land-use and road network modifications. The ability of an application like Google Earth Street View to provide the user with an ‘in-person’ experience from the convenience of the office introduces the temptation to diminish or eliminate in-person field reconnaissance activities. Critical details that would be obtained by a trained Engineering Geologist or other qualified field personnel might be de-emphasized or overlooked completely. Lastly, and as with Bing Maps imagery, proprietary issues may also exist where imagery extracted from Google Earth or any of the additional features offered within the software are used as part of a for-profit consultant study or for other purposes. Google Earth Pro offers imagery and additional functionality to the user wishing to utilize Google Earth for commercial business activities.

**Additional Preliminary Geohazard Assessment Resources**

Aside from Google Earth, Bing Maps, and other imagery tools, a prolific variety and quantity of web resources are available and at the disposal of the preliminary geohazard assessment practitioner. A select few of these from government and university sources are explored below as a brief representation. While web-available literature and mapping resources, particularly from government and academic institutions, continue to provide practical information for preliminary geohazard assessments, the government-based resources are susceptible to government funding fluctuations, leaving the geohazard assessment practitioner vulnerable to circumstances wherein traditional sources of information (i.e. paper and hard-copy references) must be more heavily relied upon.

*Pennsylvania Mine Map Atlas*

The Pennsylvania Mine Map Atlas (7), currently residing at [www.paminemaps.psu.edu](http://www.paminemaps.psu.edu), provides the geohazard assessment practitioner with the ability to view, print, and save georeferenced maps of mine workings from the interface of a map of Pennsylvania that can be scrolled and zoomed in and out to display the area of interest. The web application saves the practitioner the time and expense involved in travelling to a mine map repository location, greatly expediting the process of identifying underground mine working geohazards. Similar web applications exist in other states afflicted with abandoned underground mine workings and the geohazards associated with them.

*Web Soil Survey*

The Web Soil Survey provides soil data and information produced by the National Cooperative Soil Survey (8) and is operated by the United States Department of Agriculture (USDA) Natural Resources Conservation Service. The Web Soil Survey currently resides at [websoilsurvey.nrcs.usda.gov](http://websoilsurvey.nrcs.usda.gov) and allows the user to zoom in to their particular area of interest in the United States and generate a customized soil map and report depicting the areal distribution of the soil units found within the selected area of interest and listing the characteristics of each depicted soil unit. The Web Soil Survey replaces the hard copy soil survey references published previously by the USDA, allowing for practitioners not in possession of the hard-copy soil
survey references of a particular area to identify objectionable soil types during a preliminary geohazard assessment.

**KML Geologic Maps - United States Geological Survey**

Amongst the large amount of data electronically available from the United States Geological Survey (USGS) website, state geologic maps in Google’s Keyhole Markup Language (KML) format for viewing in Google Earth (9) are currently available for download at [tin.er.usgs.gov/geology/state/kml](http://tin.er.usgs.gov/geology/state/kml). Not only does this resource eliminate the need for paper-copies of basic geologic maps for the various states, but also permits the Google Earth user to superimpose a georeferenced geologic map over the satellite imagery interface. The polygons constituting the discrete geologic units each have a full, textual geologic description associated with them, which can be viewed simply by clicking the cursor within the polygon. In this manner, the preliminary geohazard assessment practitioner can rapidly access basic bedrock geologic data for any location within the United States, regardless of the practitioner’s proximity to that location.

**Unmanned Aerial Vehicles**

High quality, project-specific imagery at a reasonable cost for preliminary geohazard assessments might soon be acquired through the use of unmanned aerial vehicles, or drones. Although currently not in common practice for non-military applications and with a nebulous legal disposition, the unmanned aerial vehicle, equipped with photographic or other imagery-gathering equipment and guided or programmed by a trained Engineering Geologist or other qualified practitioner, might provide a valuable tool for assessing geohazard potentials in a wide variety of scenarios where project budgets prohibit the use of more traditional aerial photography. The technology would also be useful where geohazard site features of interest reside in locations that are difficult and/or dangerous for traditional, manned aircraft to fly. Geohazard assessment practitioners might be well served by looking into adding this technology to their tool belts as the legal standing for unmanned aerial vehicle use for civil applications clarifies and the technology evolves to where it becomes more widespread and cost-feasible.
CONCLUSIONS

The change that has occurred in the manner in which preliminary geohazard assessments are conducted, principally through the use of information that is electronically available on the web and online aerial and satellite imagery resources, brings practical advantage. Preliminary geohazard assessments can be conducted more rapidly and with greater precision and effect, with a subsequent reduction in labor, cost, and elapsed time to generate the information and apply it for additional geologic/geotechnical actions. Follow-on investigations such as site reconnaissances, subsurface drilling and sampling investigations, and laboratory testing programs can become more focused and somewhat surgical, as well as more sustainable through reduction in footprint at the project site via prioritization of geohazard susceptible areas of interest.

With the proliferation and continuing evolution of web-available geologic mapping and literature, along with satellite and aerial imagery, the challenge to Engineering Geologists or other practitioners in geohazard characterization becomes keeping abreast with the latest, up-to-date resources and web utilities. The practitioner failing to meet the challenge of maintaining a command over the breadth of extant materials, and those becoming newly available daily, is in jeopardy of not only becoming obsolete in comparison to their peers, but most importantly of overlooking critical information and details of a particular geohazard characterization project simply out of ignorance of the information readily available.

With great power comes great responsibility. Caution must be exercised on the part of the practitioner engaging in preliminary geohazard characterization with regard to understanding the responsible use and limits of the resources. Limitations of outdated or proprietary imagery, obsolete or erroneous literature, and photographic imagery applications offering ‘in-person’ site experience, must be understood and carefully considered when utilizing web-available resources. The proliferation of web-available imagery must be met with caution to avoid overreliance, especially where the web-available imagery is inadequate, and project-specific aerial photography or other imagery means are more appropriate for a preliminary geohazard assessment. The temptation of cost-savings by substitution of ‘boots on the ground’ field reconnaissance with examination of ground-level photography from applications such as Google Earth Street View should be avoided. Without exception, thorough field reconnaissance by trained personnel should always be a fundamental component of every preliminary geohazard assessment.
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Double Nickel Micropile Landslide Stabilization

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ABSTRACT

In July 2011, the Wyoming Department of Transportation (WYDOT) desired to stabilize a recurring landslide located on Highway 28 between Farson and Lander, Wyoming in Fremont County. The landslide referred to as “Double Nickel” is located at Milepost 55.5 and had affected approximately 1,500 lineal feet of roadway section. The slope had experienced repeated events of movement with the most recent occurring in the spring of 2010 following a heavy snowfall and rapid melt. Several repairs and various roadway alignments had been tried in the past dating back to 1992 in an attempt to mitigate the slide. Using a consultant design and an accelerated design schedule, WYDOT completed their subsurface investigation and landslide stabilization design documents within five months. After evaluation of several landslide mitigation alternatives, over 500 large diameter (12”) micropile “shear piles” were selected as the most technically and economically feasible design and construction solution. Bid documents were then finalized for a January 2012 bid letting with construction anticipated in the spring of 2012.

This paper will present and discuss descriptions of the historical landslide movements and repairs, environmental and geotechnical assessments for design and construction, slope stability analysis, landslide mitigation alternatives, micropile shear pile design features, site challenges, micropile installation, and monitoring and performance (to date) of the landslide stabilization system.
INTRODUCTION AND BACKGROUND

The Wyoming Department of Transportation (WYDOT) desired to stabilize a recurring landslide located on Highway 28 between Farson and Lander, Wyoming in Fremont County. Refer to Figure 1 for a site vicinity map. The landslide is referred to as “Double Nickel” and is located at Milepost 55.5. The slope had experienced repeated events of movement with the most recent occurring in the spring of 2010 following a heavy snowfall and rapid melt. Several repairs and various roadway alignments had been tried in an attempt to mitigate the slide. WYDOT contracted with HNTB Corporation (HNTB) to evaluate and design the landslide stabilization on an accelerated schedule with intentions to let the project by January 2012 with substantial completion by Fall 2012. Design requirements established by WYDOT included the following:

- The selected stabilization option shall maintain two-way, two-lane traffic during construction.
- Improvements shall be constructed from within the existing right-of-way limits.
- The landslide stabilization shall be designed to a static factor of safety equal to 1.3 with a target factor of safety during a seismic event of 1.0+.

LANDSLIDE HISTORY

Prior to 1985, Highway 28 was positioned further upslope from the current alignment. In order to improve geometrics, the roadway was realigned and “straightened”. As part of the highway improvement design, WYDOT performed a typical roadway investigation consisting of numerous shallow borings drilled along the proposed alignment in 1981 and 1982. A significant landslide was documented to the west of the project site, occurring along a bentonitic clay layer at a depth of 12 to 14 ft. A spring was noted within the project limits about 15 ft right of the proposed centerline. Roadway design included an underdrain to discharge the spring flow. Within the project limits, a significant amount of fill (40 ft) was required to bring the proposed roadway to grade.
Movement was observed in the summer of 1987 and was originally attributed to excessive settlement of the fill. Several inclinometers were installed to monitor the movement. Further disturbance was noted in the summer of 1988 and additional inclinometers were installed. Continued movement indicated the development of a landslide rather than settlement of the fill. Additional investigations were conducted in August 1991 and July/August 1992.

Three toe trench drains were installed in 1992 immediately down slope of the spring location. The design length of the trenches was 120 ft with a maximum footprint width of 50 ft at the top. A drainage trench was installed at the base of the toe trenches. As-built documentation indicates that the slide plane was intercepted in the back slope of the trenches at the right-of-way limit. Backfill material consisted of large diameter, iron ore rock from a nearby quarry.

Continued movement after the installation of the toe trench drains in 1992 necessitated a more extensive landslide repair. In 1994, WYDOT designed a landslide repair including realignment of the roadway to the north (i.e. upslope), placement of lightweight tire shred fill, and installation of four additional rock-filled toe berms. Furthermore, a three-sided box culvert was constructed to collect and discharge the spring water. Refer to Figure 2 for a plan view illustrating the various roadway alignments and location of previous improvements.

Movement of the slope requiring frequent maintenance was on-going. After the 1994 improvements, three inclinometers were installed in December 1995. Two more inclinometers
were installed in April 2000. Heavy snowfall followed by a rapid snowmelt in the spring of 2010 aggravated the slide mass with new tension cracks forming 250 ft east of the previously defined active landslide. In response, WYDOT installed additional inclinometers in June/July 2010. WYDOT conducted a more extensive investigation in the spring of 2011 including drilling 16 borings and installing open standpipe piezometers and inclinometers.

**SUBSURFACE INVESTIGATIONS**

As discussed above, WYDOT completed numerous subsurface investigations directed primarily at installing instrumentation to monitor the site. Hence, limited samples of the soil and bedrock were collected and drilling methods were selected to achieve target depths for instrumentation installation. Characterization of the subsurface materials was based primarily on evaluation of cuttings and drill rig response. WYDOT installed casing for displacement monitoring in selected boreholes using Slope Indicator equipment. Several open standpipe piezometers were installed to monitor groundwater levels.

As part of the landslide stabilization design, HNTB and their sub-consultant Wyllie & Norrish Rock Engineers, Inc. (W&N) developed a subsurface exploration program in cooperation with WYDOT to achieve the following objectives:

- Investigate and characterize the engineering and geologic subsurface conditions
- Install multiple-level vibrating wire piezometers to characterize pore water pressure
- Identify potential shear planes
- Characterize the structural integrity of the bedrock using an acoustic and optical televiewer instrument.

Boring locations were selected by WYDOT, HNTB, and W&N and staked in the field during a site reconnaissance on July 14, 2011. Four borings were located along the inferred axis of the landslide. A fifth boring location was selected to fill in an apparent gap in the existing boring information. Prior to drilling, WYDOT surveyed the location and elevation of the previously installed instrumentation (inclinometers and piezometers) and the proposed borings. The subsurface investigation was completed by WYDOT forces between August 2 and 12, 2011.

**PROJECT GEOLOGY AND EXISTING SUBSURFACE CONDITIONS**

The project site is underlain by soil overburden on top of the Tertiary age White River Formation. The White River consists of calcareous conglomerate and tuffaceous sandstone. An erosional unconformity is present between the younger White River Formation and the adjacent Pennsylvanian-aged Tensleep Sandstone and Amsden Formation to the north. Bedrock generally dips toward the northeast at 10 to 12 degrees. Significant structural features are not noted in the area.

Overburden materials encountered during drilling consist of fill, colluvium, and residual soils. Shredded tire fill from the 1994 repair was noted in several borings as was iron ore rock fill associated with the toe trench construction. The colluvium and residual materials vary considerably in composition but are generally described as stiff to hard, brown to reddish brown, gravelly clay and medium dense to dense, brown to gray, clayey gravel with sand, cobbles, and
boulders. Moisture content of the overburden generally increases with depth. The fine-grained fraction is generally plastic.

Underlying the overburden is relatively soft, fine-grained bedrock consisting of shale, siltstone, and claystone. Harder sandstone and limestone bedrock is typically present in thin nodular seams. Due to the soft bedrock initially encountered in the borings at the site, the top of bedrock contact is not well defined and gradational as the weathering intensity decreases with increasing depth. The top of bedrock plan indicates a depression in the vicinity of Sta. 735+50 that corresponds to an erosional draw identified in a 1982 aerial photograph that was subsequently filled in for the 1985 roadway realignment. Overburden thickness ranges between approximately 10 and 75 ft based on the available boring information. As indicated by the top of bedrock contours, the thickest overburden section is located near the axis of the landslide.

Groundwater was encountered during the subsurface investigation, and water levels were also measured after completion of drilling. Delayed groundwater levels were generally higher than the noted level of groundwater entry, indicating an upward hydraulic gradient (artesian conditions).

**GEOLOGIC INTERPRETATION AND SLIDE GENESIS**

In general, the suspected slide plane (determined from inclinometer readings and direct observation of disturbed material) is positioned at or near the top of bedrock. The maximum depth of the slide plane below the ground surface is approximately 80 ft at the axis of the landslide at approximately Sta. 735+50 (Figure 3). The depth to the slide plane decreases to about 60 ft on the side flanks of the slide.

![Figure 3. Cross-Section at Slide Axis (Sta. 735+50).](image)
The slide is most influenced by the White River Formation, consisting of Tertiary-aged, slightly indurated, tuffaceous sandstone and shale. The White River is described in the literature as being ash rich. Thus, it is believed to contain numerous bentonitic layers and consist of an overall weak material. The Tertiary period (30 million years before present (ybp)) is characterized by the Laramide Orogeny and filling of the structural basins by sediments eroded from the mountains. The White River Formation is underlain and flanked by stronger Paleozoic-aged (300 to 400 million ybp) bedrock of limestone, shale, and sandstone. The Tensleep Formation is described as a cliff forming sandstone, is geometrically parallel to the strike, and may have formed detached blocks. These blocks may have detached laterally from the formation and moved down slope prior to Tertiary time. These blocks are not believed to be the cause of or contributing to the failure; however, the slide itself may be made up of en echelon blocks.

Groundwater is likely transmitted down dip with considerable head from southwest to northeast, downward from the continental divide to the site. When examined from along the strike, the Madison Formation could be directly contributing water laterally in the elevation range of 7250 to 7300 ft, further weakening the White River materials and providing a mechanism for slide movement.

In summary, it is believed the slide is occurring in overall weak Tertiary materials aided by bentonitic zones and weathering of the shale layers to fat clay. The axis of the slide appears to be positioned within a bedrock depression from a past erosional event. The mechanism is triggered by the addition of groundwater from the Madison Formation as well as along the contact between the Tertiary and Paleozoic formations. Surface water is also contributed near the present head scarp of the slide.

**SLOPE STABILITY EVALUATION**

An assessment of the landslide was performed to evaluate potential landslide movement under measured or inferred engineering parameters for the site. These parameters include slide geometry, measured groundwater levels, and material properties. Stability model(s) were
developed to evaluate the ratio of resisting forces to driving forces expressed as a factor of safety (FS). A calculated FS slightly less than unity would be consistent with landslide movement. Slope stability evaluations were conducted using Slide6.0 (version 6.012) software developed by RocScience.

Two geotechnical models for stability analyses were developed representing a cross section through the axis of the observed slide and another section on the side flank. Initially, a simple two-layer model was analyzed consisting of soil overburden and bedrock. A path search for non-circular surfaces was conducted and the critical slip surface originates at the toe of the slide with a calculated FS of 1.25.

Based on direct observation of disturbed materials along the inferred slide plane, depths of recorded inclinometer movements, and laboratory testing, a “pre-sheared” layer was incorporated into the model. The residual friction angle of this material was varied until the global FS was near unity. The bedrock was assigned an anisotropic strength function to incorporate weakness along the bentonitic bedding planes dipping into the hillside. Overburden material above the pre-sheared layer that may contain relict structure was also assigned the anisotropic strength function to account for potential bentonitic planes of weakness. Finally, the toe trenches present at the base of the slope (1992 and 1994 improvements) were included in the model with composite material properties from the overburden and rock fill. The following figure illustrates the anisotropic strength functions used in the analysis.

![Figure 5. Anisotropic Strength Function.](image)

In order to achieve a FS of unity (i.e. on the verge of failure), the residual friction angle of the pre-sheared layer and along potential bedding planes in the bedrock was lowered to 14 degrees. This is in good agreement with available residual direct shear test results on samples at or near the inferred slide plane and WYDOT’s experience. A residual friction angle of 14 degrees also correlates well with published relationships based on Plasticity Index (PI). The same material parameters were then used in the stability model along the side flank of the slide.
CONCEPTUAL STABILIZATION ALTERNATIVES

General methods for landslide stabilization include slope grading, groundwater control, and structural reinforcement such as micropiles, drilled shafts, ground anchors, and driven piles. For the Double Nickel site, slope grading would likely have required the acquisition of additional right-of-way. Groundwater control through the installation of horizontal drains would also have required additional right-of-way and would be difficult to install due to the length of the drains and the subsurface conditions. Other active dewatering options such as a pump station would require utility improvements to the site and on-going maintenance. In order to meet the design criteria and schedule established by WYDOT, the preferred stabilization technique was structural reinforcement.

Based on the existing subsurface conditions and the high potential to encounter cobbles and boulders, driven piles and drilled shafts were not desirable. Driven piles would require a high percentage of pre-drilling to install the piles to the minimum tip elevation. Drilled shafts are rigid structural elements that would require fabrication of long reinforcing steel cages. Use of temporary or permanent casing would likely be required. Ground anchors require construction of structural concrete bearing blocks to distribute the load over the ground surface. Due to the low strength of the bedrock present at the site, numerous long anchors would likely be required. Rigorous testing is required to verify load carrying capacity. In addition, the most efficient location of ground anchors would be within the slope beneath the existing roadway level. Drilling angled holes for the insertion of the ground anchors may be difficult due to the presence of cobbles, boulders, and potentially shredded tire fill. Preliminary cost estimates indicated that the ground anchor option was cost prohibitive.

The preferred stabilization method was shear piles (micropiles) consisting of small diameter flexible elements installed across the slide plane to provide passive shear capacity, disrupt the slide plane, and reinforce the soil mass. Due to the depth of the slide plane below the ground surface and to reduce potential drilling difficulties, the shear piles were assumed to be installed vertically. By limiting the drill hole diameter to approximately 12 inches, it was assumed that the holes could be advanced utilizing rotary drilling methods that advance casing with the bit (i.e. dual rotary drilling). The structural elements can be readily installed down slope of the existing roadway, primarily from the bench located at approximately elevation 7330 ft, in order to minimize impacts to traffic.

MICROPILE (SHEAR PILE) ANALYSIS

The stability model at Sta. 735+50 (design section) was used to evaluate the resisting force needed to improve the existing factor of safety from unity to the minimum required of 1.3. Using Slide 6.0, the required resisting force for a single row of piles to increase the factor of safety to 1.3 was 300 kips/lf of slope. In order to reduce the resisting force per structural element, a total of four rows of piles were analyzed. Several iterations were conducted to optimize the spacing and to address up slope and down slope stability. The final configuration consisted of one row of piles near the crest of the lower toe slope and three rows on the bench.
below the existing roadway. Due to inefficiencies associated with spreading the rows out from the center of the slide mass, the required resisting force per row increased slightly to 76 kips/lf.

![Figure 6. Slope Stability Model with Shear Piles (Micropiles).](image)

The subsurface conditions along the section at Sta. 735+50 (design section) were used to develop the lateral loading parameters. LPile 6.0 software by Ensoft Inc. was used to model the piles under lateral load. Various methods to analyze piles for stabilizing a moving soil mass are presented in literature. Per FHWA’s Drilled Shaft Manual, the lateral load applied to the structural element is the lesser of the maximum passive earth pressure of the moving soil or the resisting force necessary for overall stability of the slide mass. Using the soil parameters entered into LPile and the associated p-y curves, the ultimate passive earth pressure corresponding to approximately 2.5 inches of displacement is 1,005 kips per pile. Based on a 6-ft pile spacing for four rows, the resisting force needed per pile is 456 kips. Thus, the piles were designed to resist a force of 456 kips corresponding to less displacement. The 456-kip force was applied as a triangular pressure distribution against the pile above the slide surface for a pile weak in bending. Due to the depth of the slide plane and overburden confinement, “flow” of the soil around the piles at the 6-ft spacing was not anticipated. In addition, the material on the down slope side of the pile was assumed to remain in-place and provide resistance along the length of the pile (i.e. the piles were considered a soil reinforcing element). To account for variability of the soil mass and potential disturbance during installation of the shear piles, a reduction factor was applied to the lateral resistance of the soil mass on the down slope side of the pile.

Based on the loading conditions presented, LPile was then used to evaluate the structural response of the pile including shear force, bending moment, and deflection. The structural capacity at the threaded joints was calculated based on guidance provided in FHWA’s Micropile Design and Construction Manual and was approximately one-half of the values calculated for the
11.875-in diameter casing with a wall thickness of 0.582 inches. Due to the depth of the slide plane, bending moment controls for a pile loaded with a triangular pressure distribution. Comparing the maximum calculated bending moment and shear force obtained from LPile and the structural capacity of the shear piles, the casing ($F_y = 80$ ksi) and joints were adequate in both bending and shear based on a triangular pressure distribution.

Where the slide plane is well defined at the center of the slide mass, a check was made considering the structural capacity of the shear pile if the 456-kip load was applied as a concentrated force at the slide plane. In this case, the shear capacity controls and the joints did not provide sufficient capacity. Therefore, restrictions on joint locations were included in the contract plans where the slide plane was well defined along the axis of the slide.

**FINAL DESIGN RECOMMENDATIONS**

Based on the analysis presented above, the following graphics summarize the design recommendations for the shear piles.

![Figure 7. Shear Pile Stabilization Plan Layout.](image-url)
As previously discussed, various methods of analyzing and designing shear piles are presented in the literature. Reasonable assumptions were made with respect to the soil-pile interaction in order to minimize the structural element required and provide a cost-effective repair. Based on the assumptions made during the analysis of the shear piles, additional instrumentation including inclinometers and strain gauges were required within the shear pile casing at select locations to monitor the response of the shear piles.

CONTRACT AWARD

Bids for the Double Nickel Slide were opened on January 12, 2012. A total of seven bids were submitted and ranged from $5.884 million to $6.983 million with an engineer’s estimate of $6.177 million. The contract was awarded to Donald B. Murphy Contractors, Inc. of Federal Way, Washington.

CONSTRUCTION

Planning and Preparation

Drill Rigs

During the planning process several factors dictated the choice of equipment. Since the shear pile design required the 11.875-in OD permanent casing to be advanced as the hole was drilled, it
was decided to equip the drill rigs with two rotary heads. The upper rotary was set up to turn the inner drill rod, and the lower one was used to rotate the permanent casing.

Additionally, the specifications required that over 35% of the shear piles be installed with no casing joints between elevation 7240 and 7260 ft. This restriction, along with the relatively large diameter of the casing, necessitated the use of large rigs with high torque and long stroke capability. Two ABI 18/22 rigs were selected, and were outfitted with dual rotary drives.

**Permanent Casing**

The plans specified a minimum tip elevation for each shear pile, with the top of pile required to be between two feet and five feet below the existing ground elevation. This resulted in pile lengths ranging from a minimum of 78.2 ft to a maximum length of 101.6 ft. The average pile length was 91.3 ft.

One of the challenges in procuring the permanent casing was working out combinations of lengths of casing that would meet the minimum shear pile lengths specified in the contract drawings, while minimizing excess length, which would increase material costs and drilling time, without additional compensation from WYDOT. In addition, use of the dual rotary drilling method required that the lengths of the permanent casing pieces be matched with the lengths of the inner drill rod sections. A significant effort went into optimizing this geometry and minimizing the number of different length pieces of casing.

Additional challenges in casing procurement resulted from the quantity of material (48,000 LF) required for the project. Typically, casing used in the U.S. micropile industry is API oilfield pipe that does not meet the rigid manufacturing specifications established by API for oilfield work.
The price for this “mill secondary” casing is substantially less since it is not usable for its primary purpose.

Initial Drill Tooling Selection

Based on the information in the geotechnical report, the initial drill tooling selected for the project consisted of carbide-tipped “J” teeth welded to the lead end of the 11.875-in OD permanent casing for the outer string, along with an aggressive claw-type bit with carbide-tipped “bullet” teeth coupled onto the 7.625-in OD drill rod for the inner string. In addition, due to the known presence of the rock fill in the toe drains for some of the shear piles located in Row 4, an 8-in downhole hammer and button bit set-up was also selected. It was anticipated that this percussive set-up would need to be used in selective locations to deal with the toe drain rock fill, as well as some of the hard sandstone seams identified during the geotechnical investigation.

Support Equipment

The primary flushing medium selected for the drilling was air, with the intention to add water (and possibly clay inhibitor) as necessary to maintain adequate flush. The geotechnical investigation identified overburden consisting primarily of silts and clays which have a tendency to “collar off” the annulus between the inner rod and the casing, thereby hampering advancement of the drill hole. With this in mind it was anticipated that the amount of water added to the air flush would need to be adjusted by crews in the field to maintain productivity.

An additional consideration in selecting the air compressor configuration was the elevation at the site. The project site is located at elevation 7300 ft. At this elevation, the rated volume of an air compressor is significantly reduced. In order to maintain adequate uphole velocity of the drill cuttings between the inner drill rod and the permanent casing, the job was set up with three each 900 CFM / 350 PSI portable air compressors. All three units were attached to a single steel header pipe for distribution to the two drill rigs.

The length and weight of the drill rod and casing also required special handling equipment. Casing was handled with a service crane and special rigging. The drill rods were handled with hydraulic excavators equipped with pipe clamps for loading the pipe into the casing during
installation, and for removing the drill rod as it was “tripped out” after completion of drilling each hole.

Figure 12. Casing/Drill Rod Handling.

**Grouting**

The contract required the 11.875-in OD permanent casing be backfilled with cement grout. With the large quantity of piles for this project, an automated grout plant was selected to efficiently mix and pump the neat cement grout. A Scheltzke MPS 510 grout plant was chosen and was set up with twin silos to allow use of bulk cement.

**Site Preparation**

**Access and Benching**

The contract drawings required the top of each shear pile to be installed to an elevation between two feet and five feet below existing ground. With the sloping topography at the jobsite, and with the size of the drill rigs required to install the permanent casing, a significant amount of grading work was required to create stable and relatively level access to each of the 526 shear pile locations.

During this grading work, numerous boulders were encountered near the surface. One of these boulders had a 2-in diameter hole drilled through it, indicating it was “shot rock”, or rubble created from a blasting operation.
Spoils Control

It was anticipated that there would be a significant volume of dirty water and wet spoils generated during 48,000 LF of shear pile installation. The site was set up with a settling pond at the low end of the site where waste water could be channeled and contained. In this same area, a location for spoils disposal was selected.

Shear Pile Installation

Shear pile installation started on the lower Row 4 piles as required by the sequence in the contract documents. The presence of the rock fill in the toe drains in this area was known, and it was anticipated that this condition would present challenges during installation of these piles. This was certainly the case. In two instances, the casing could not be advanced to the required tip elevation. Initially, these challenges were attributed in part to the learning curve with the new equipment and drilling system, as well as the known ground conditions at Row 4. Various modifications were made to the drill tooling, including changes to the bit configuration and the type and quantity of cutting teeth on the permanent casing. In addition, the button bit and downhole hammer set-up was used on several shear piles during this period of the project. While production improved somewhat with these adjustments, it was still not adequate to meet the schedule requirements for the project.

Based on the experience gained during this initial production period, a new tooling system was selected. This system consisted of a 10-in diameter downhole hammer paired with an underreamer bit with a maximum diameter of 12.5 inches. This system required the permanent
casing to have a “blank” end with no cutting teeth. This system resulted in a much more reliable production rate than was achieved previously.

Figure 14. Down-The-Hole Hammer and Underreamer.

However, this particular system was available in limited supply. With the remaining quantity of drilling left on the project, it was necessary to develop an additional tooling configuration that was available in adequate supply to allow the project to be completed on schedule. This additional system consisted of a 10-in diameter downhole hammer paired with a 10-in diameter button bit. The lead section of casing was modified with thicker carbide “J” teeth than originally selected. Various modifications were made to the quantity of the cutting teeth on each lead section of casing during the course of pile installation.

Figure 15. More Aggressive Casing Leads.
These two different systems were used for the remainder of the project. As a result of implementing these percussive systems, additional compressed air volume was required. A fourth compressor was added to the system so each drill rig would be supported by a pair of 900/350 air compressors.

During the shear pile installation with both of these systems, it was noteworthy that the downhole hammer would fire consistently through the clay and silt layers in addition to the rock formations indicating the continuous presence of harder materials. This was the case with either of the two systems (underreamer or button bit) used during the production phase of the work.

**Schedule and Production Summary**

Installation of the 526 shear piles was completed in 15 weeks. The first three weeks and the final two weeks included one drill rig working a single shift. The other 10 weeks of production were performed with two drill rigs working a single shift, or one rig on a two shift basis. The shift structure varied as a result of rig maintenance and/or tooling availability.

Once 75% of the piles were installed, reconstruction of the roadway began. Re-grading of the drill benches and final site restoration activities followed immediately after shear pile installation was complete. Despite the challenges encountered early in the project, the job was finished within WYDOT’s budget and construction schedule specified in the contract.

![Figure 16. Shear Pile Installation.](image-url)
INSTRUMENTATION AND MONITORING

As required by the construction contract, instrumentation consisting of sister-bar strain meters and inclinometer casing was installed within six shear piles across the site. The sister-bar strain meters were installed as upslope and downslope pairs at specific elevations above, at, and below the suspected slide plane. Inclinometer casing was installed full-depth within the shear pile. The table below summarizes the instrumented shear piles.

<table>
<thead>
<tr>
<th>Shear Pile No.</th>
<th>Strain Meter Pair Elevations (ft)</th>
<th>Inclinometer Casing</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1-30</td>
<td>7310, 7300</td>
<td>Full-Depth</td>
</tr>
<tr>
<td>SP1-109</td>
<td>7270, 7260, 7255, 7250</td>
<td>Full-Depth</td>
</tr>
<tr>
<td>SP1-150</td>
<td>7270, 7260</td>
<td>Full-Depth</td>
</tr>
<tr>
<td>SP2-130</td>
<td>7280, 7275, 7270</td>
<td>Full-Depth</td>
</tr>
<tr>
<td>SP3-10</td>
<td>7285, 7280, 7275</td>
<td>Full-Depth</td>
</tr>
<tr>
<td>SP4-40</td>
<td>7265, 7255, 7250, 7245</td>
<td>Full-Depth</td>
</tr>
</tbody>
</table>

Instrumented shear pile SP1-109 and SP4-40 are along the inferred axis of the slide plane. SP1-150 and 2-130 are on the right flank near the spring location while SP1-30 and SP3-10 are positioned on the left flank of the slide.

The strain meters are connected to data loggers and are set to take readings on a daily basis. Acquisition of strain meter readings began within a few days after installation with the baseline “zero” reading established approximately 28 to 30 days after installation to account for curing of the grout within the shear pile casing. The inclinometers are manually read by the Department. Initial baseline readings of the inclinometers were obtained in October 2012. Subsequent readings of the inclinometers were obtained in May 2013 and October 2013 along with downloading of the strain meter data loggers. Key observations to date include the following:

- The maximum calculated strain for the shear piles is approximately 5,000 με. The maximum recorded strain at any strain meter is less than 300 με (less than 6%).
- The maximum movement indicated by the inclinometer readings is approximately 0.1 in.
- Negative strain readings indicate compression while positive strain readings indicate tension. The strain meter data at SP1-109 illustrates the anticipated shear pile response with the upslope strain meter at elevation 7270 ft indicating tension and the corresponding downslope strain meter indicating compression. Strain meters at lower elevations indicate essentially no differential loading between the upslope and downslope pairs. The inclinometer data at SP1-109 indicates approximately 0.1 in of movement in the A₀ direction at elevation 7270, supporting the strain meter data.

CONCLUSIONS

The designed landslide stabilization met the aggressive schedule established by WYDOT and necessitated by the typical weather conditions at the site. The innovative use of vertically installed micropiles to provide passive shear resistance, disrupt the slide plane and reinforce the
soil mass permitted rapid installation at a remote location in a short construction season. Initial production rates were slowed by the variable subsurface materials and delivery of materials and equipment to the site. Acquisition of different drill bits and modification to the cutting teeth on the casing improved production. Daily production rates increased in mid-June and installation of the 526 shear piles, over 48,000 ft, was complete by the end of August 2012. Instrumentation designed and installed within the production micropiles will provide invaluable insight into the response of the system. The combination of inclinometers and previously-installed piezometers will monitor the geotechnical parameters and pinpoint the depth of slide movement while the strain meters will monitor the structural response of the vertically-installed micropiles. The measured strain of the micropiles can be converted into load to better understand the mechanics of load transfer between the moving slide mass and the structural elements. To date, the instrumentation indicates stable conditions with no significant movement since the shear piles were installed.
Rock Bolting in Sensitive Environments:

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Disclaimer

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ABSTRACT

Rock bolts are routinely used throughout the United States to improve the stability of natural rock slopes and cut slopes in rock adjacent to roadways, including on low volume roads in sensitive environments such as Federal Lands Highway (FLH) projects. Several existing manuals address the design and installation of rock bolts. However, these manuals are generally geared toward high volume roads where public safety and cost are the primary project objectives. While FLH projects recognize the importance of safety and cost, the overall goals of the projects are unique because of the major concerns with aesthetics, construction impacts, and maintenance. In addition, the level of acceptable risk for low volume roads may vary from those for a high volume divided highway. To address these concerns, the Central Federal Lands Division of the FHWA has commissioned development of a rock bolting manual. Specifically, it will address project planning, design, construction, inspection, testing, and maintenance in a format that considers the unique FLH project goals. While the manual is being prepared specifically for use on projects administered by Federal Lands Management Agencies (ex. National Parks and Forests), we anticipate that it can be used by other agencies that own, build, and maintain low volume roads, such as state and local park agencies. The manual will be available in late 2014 or early 2015, and this presentation will provide an overview of the manual while highlighting methods of dealing with the unique challenges of rock slope stabilization in sensitive environments.
BACKGROUND

Rock bolts are structural elements that are installed to improve the stability of slopes and tunnels. Rock bolts were first used in the United States in 1927 for underground coal mining and their use expanded significantly into the mining and civil tunneling industry during the 1940s and 1950s. The confidence in rock bolt design and construction is supported by the fact that over 100 million bolts were used in the United States mining industry in 1999 (1).

In recent years, rock bolts have been routinely used by transportation and railroad agencies for design of new cut slopes, for rehabilitation and stability improvements on existing cut slopes, and for tunnel improvements. The use of rock bolts has extended to Federal Lands Highway (FLH) facilities that include roadways through national forests and parks. The common applications of rock bolts on FLH projects are stabilization of masses to improve the safety of natural and cut slopes along the roadways. However, some of the projects have resulted in a significant detrimental visual impact on the rock. For many FLH roads, providing safe access to federal lands and preserving the context of the surroundings are the primary concerns, and significant alteration of the aesthetic qualities is not acceptable. Therefore, prior aesthetically unpleasing projects have resulted in hesitation or resistance by stakeholders to use rock bolts. The situations leading to the unacceptable end product are varied, but include poor communication between the designer and the contractor, inadequate inspection, and improper resolution of condition changes in the field.

Purpose and Need

The Central Federal Lands Highway Division of FHWA recognized the challenges associated with educating stakeholders, contractors, and engineers regarding the unique challenges and expectations when installing rock bolts on FLH projects. They commissioned Shannon & Wilson, Inc. to prepare a guidance manual that will address all aspects of rock bolting on Federal Land Highway (FLH) projects in an effort to standardize and provide a more consistent product. Specifically, the manual will address project planning, design, construction, inspection, testing, and maintenance in a format that considers the unique FLH delivery goals. It is anticipated that the primary manual users will be federal land management agencies (ex. National Park and Forest administrators), designers, construction inspectors, and contractors. However, it is anticipated that the guidance manual will be of value on projects in other locales where aesthetics and low impact rock bolt construction are important, such as along scenic byways and within state or local parks.

Several existing manuals address the design and installation of rock bolts, including the FHWA Rock Slopes Reference Manual (2), Chapter 6 of the FHWA Technical Manual for Design and Construction of Road Tunnels – Civil Elements (3), and the Recommendations for Prestressed Soil and Rock Anchors by the Post-Tensioning Institute (4). Each of these documents provides valuable rock bolting information and has been successfully utilized on many rock bolting projects, but none address the specific needs and challenges present for FLH rock bolting projects. Much of the information provided in the manual will be gleaned from these existing documents and framed in the context of the unique FLH project goals, including minimizing disturbance to the natural environment and providing a finished product that is
visually non-intrusive. Therefore, the manual does not propose new methods of design and installation, but instead provides a new, comprehensive approach to planning and executing projects involving rock bolting on federal lands.

**Rock Bolts Covered in Manual**

The guidance manual will discuss a variety of rock bolt types, varying from mechanical and/or friction bolts, cement and resin grouted bolts, tensioned and untensioned installations, and a variety of material types and hardware assemblages that are used for rock bolting applications. Advantages and disadvantages for the different types of rock bolts will be discussed, in addition to common applications and construction methods for each type. However, the emphasis in the manual will be focused on permanent installations to improve the stability of rock cuts for highway applications. Regardless of the application or type of rock bolt being used, the basis for design is similar in that the type, number, spacing, and length of rock reinforcement elements are dependent on the size of the feature being stabilized, the geometry and characteristics of discontinuities controlling the failure mechanism, the shear strength and engineering properties of the materials, and groundwater conditions.

**MANUAL OBJECTIVES**

**Rock Bolting in Sensitive Environments**

The common objectives for most roadway projects, including those where rock support is required, are public safety and cost (initial and life cycle). While FLH roadway projects recognize the importance of safety and cost, the overall goals of the projects are unique because of the sensitive environments in which they are constructed. Aesthetic concerns, construction impacts, and maintenance considerations have high importance, and the level of acceptable risk for the low volume FLH roads are often different than those for a high volume divided highway. The objective of the guidance manual is to provide a framework for balancing these project requirements to provide a finished product that meets the needs and expectations of FLH and other stakeholders. Specifically, the primary objectives of the manual are to alleviate stakeholder concerns by providing examples of projects that meet FLH expectations, educate contractors on the expectations of FLH projects, and provide a protocol for communication and problem solving throughout the project to proactively address stakeholder concerns. The following sections provide an overview of the project goals and concerns that were considered in preparing the manual.

**Safety on Low Volume Roads**

Safety of the travelling public on FLH roads is a primary concern. However, because of the purpose and low volume of the roadways, safety can be approached in a different manner than typically used for higher volume state and federal roads. Typical design methods for rock bolts utilize a factor of safety or probability of failure to assess the safety of a reinforced rock mass. Rock mass failures may result in rock fall that impacts the road where vehicles that can be present in high volumes and/or travelling at high speeds. Because the consequence of accidents associated with the rock fall event that adversely affects the travelling public is high, a relatively
high factor of safety or relatively low probability of failure is used in design. For the majority of FLH roadways, the number of vehicles and the speed of individual vehicles can be significantly less than an interstate highway. Therefore, if a rock mass failure results in rock fall that impacts the road, it is less likely that a vehicle will be directly impacted and more likely that the slow-moving vehicles can avoid obstructions. Larger slope failures that prohibit safe passage, along the road, at any speed are still undesirable because detours on FLH roads are often long or impractical, but the tolerance for smaller failures may be greater. In addition, driver expectations are different on FLH roads compared to interstate highways. FLH roads are often in relatively remote, undeveloped areas and drivers are typically using the roadway to either access federal recreation areas or as a scenic byway. Therefore, the perception of the users is that the travelling surface is also less developed and obstructions may be encountered. To account for the factors, discussed above, the design factor of safety may be lowered (or probability of failure increased) in some situations.

Aesthetic Concerns

For many FLH roads, providing safe access to federal lands and preserving the beauty of the surroundings are the primary concerns, and significant alteration of the aesthetic qualities of the roadway setting is not acceptable. Disturbance to the natural topography must be kept to a minimum, and the ideal rock bolt is one that cannot be seen by the travelling public. Many materials and installation technologies are available that can meet the aesthetic goals of a project. However, to fully achieve the aesthetic goals, project planning and management are essential. The project owners and stakeholders must first clearly define project aesthetic expectations. The designers and contractors must then develop a system that can meet or exceed the stakeholder expectations. Finally, inspectors must be equipped with the necessary information and tools to make field decisions that will not adversely affect the aesthetic qualities of the project.

A rock stabilization project completed along the main loop drive in Colorado National Monument provides an example of a completed project that met or exceeded aesthetic expectations. The project design consisted of stabilization using polyurethane resin (PUR) injection and fully-grouted, untensioned rock bolts. Figure 1 shows installation of a rock bolt within the stabilization zone, and Figure 2 shows the completed project. Without the photo annotation, the rock bolt locations are nearly invisible. Face plates were not utilized in the design resulting in a smaller area at the face of the rock to camouflage.
A rock stabilization project completed in Glacier National Park provides an example of a completed project that did not meet aesthetic expectations. The project design consisted of stabilization using fully-grouted, untensioned rock bolts. Figure 3 provides an example of grout staining along the rock face as a result of improper control of fluid grout during installation.
Figure 4 shows rock bolts that were installed without face plates, similar to the Colorado National Monument project. However, the grout used near the rock face in Figure 4 was not properly tinted to match the color of the rock, resulting in conspicuous rock bolt locations.
Construction Impacts

In conjunction with providing a finished product on a rock bolting project that meets the aesthetic expectations of the stakeholders, it is beneficial to limit impacts during construction. First, construction means and methods should be selected such that surrounding areas are avoided or minimally disturbed. Unlike typical roadway projects, it may not be possible to restore surrounding areas in a sensitive environment because of the natural or historical significance. Rock bolts may be beneficial on projects where minimization of disturbance is important, because the common alternative to rock stabilization is often flattening of slopes that result in an increase in volumes of excavated material and increased disturbance. Second, it is often beneficial to limit the duration of construction. Access to the roadway and areas adjacent to the roadway may be limited to specific seasons because of weather and/or biological restrictions. In addition, FLH roadways that provide access to popular federal land recreation areas typically have busy seasons where road closures are highly discouraged or not possible. Finally, many FLH roads are relatively narrow and in difficult access areas where lane shifts and detours are not possible.

Maintenance Considerations

Because of difficult access associated with many of the rock bolt projects on FLH roadways, it is important to provide a low maintenance product. For rock bolting applications that require periodic inspection and/or maintenance, design decisions and products should be chosen that consider management of the features over the life cycle. For example, the bolt locations may need to be accessed for occasional maintenance and inspection. Therefore, considerations should be made for inspection and maintenance that can be accomplished at a low life-cycle cost.

Also, inventory and data management are important as the life-cycle of the rock bolt projects often will exceed the tenure of most of the individuals who were involved in design and construction. To support asset management practices, these data should be maintained so that future maintenance and inspection practices consider the decisions made during design. For example, some bolts may be designed without corrosion protection, which can be important information when prioritizing asset management needs.

Project Control Measures

A key element of a successfully completed project that meets the expectations of all project stakeholders is planning. Because of the unique aspects of completing rock bolting projects in sensitive environments, communication and planning is essential. The authors anticipate that the manual will provide guidance for communication and planning, as well as being an educational tool to provide information to the federal land management agencies regarding the aesthetics and maintenance requirements for finished rock bolting projects and to provide information for contractors regarding the types of rock bolt systems that are acceptable and the expectations for the finished product. At a minimum, the following basic steps should be completed to facilitate a quality finished rock bolting project.
Prior to completing the design phase of a project, the designers and management agencies should discuss project expectations and unique circumstances that pertain to the project, including aesthetic, biological, or historical limitations.

The designer should provide the rock bolt guidance manual as supplemental information in the bid package with references to specific rock bolting systems or installation methods, as applicable.

At a pre-bid meeting, the importance of minimizing disturbance during construction and project specific concerns and limitations should be emphasized.

The construction inspector should be familiar with the manual, including the protocol for addressing field changes, and all project specific concerns and limitations.

The construction inspector should have routine contact with the rock bolting contractor in which project goals and expectations are discussed. Daily reports from the inspector should include discussion of compliance with project goals in addition to technical aspects of the project.

The inspector, designer, and contractor should discuss project changes and their effect on the disturbance and aesthetic qualities of the project prior to implementation.

**Standardization without Sacrificing Flexibility**

A primary objective of the manual is to standardize the planning, design, inspection, and construction of rock bolt projects. However, the authors acknowledge that conditions in the field are often different than those that were assumed for design. This is especially true for geotechnical projects where geologic data is extrapolated over a relatively large area from limited exploration data. Therefore, changes to the rock bolt design are often necessary during the construction phase of the project. Field changes can range from relatively minor, such as moving a rock bolt a small distance, to relatively major, such as addition of rock bolts or changes in construction methods. This manual will provide the necessary information for the construction inspectors to determine whether a proposed change is minor enough that a field decision can be made or whether it major enough that the designer and stakeholders should be consulted prior to implementation. In addition, for spot bolting operations, the inspector needs to use engineering judgment and knowledge of project expectations and goals to determine the placement of rock bolts as construction progresses. The protocol for all of these decision processes will be provided in the guidance manual.

**CONCLUSION**

Because of the locations and associated uses of roadways on federal lands, the project goals typically include minimization of disturbance to the natural environment and a finished project that is aesthetically non-intrusive. Rock bolts are valuable tools for use on FLH projects, but the sectors in which rock bolt technologies have been developed are not as concerned with aesthetics and level of disturbance. Therefore, to accomplish the unique FLH goals, proper
planning and execution from project conception through project completion is necessary. Part of the planning and execution involves education of stakeholders, designers, contractors, and inspectors. The proposed rock bolt manual can be used as an educational tool, as well as guidance for all phases of a rock bolting project. While the primary audience of the manual is individuals working on FLH rock bolting projects, it is anticipated that it will be a valuable resource for practitioners involved in rock bolting on many types of low volume roads where aesthetics and minimizing disturbance are primary project goals. The guidance manual will likely be available in late 2014 or early 2015.

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Utilization of Displacement Monitoring to Modify Rock Slope Designs during Construction

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ABSTRACT

Construction of a 5-mile-long, multiphase, interstate-widening project on Snoqualmie Pass along Lake Keechelus will require 800,000 cy of steep side-hill excavation incorporating 0.5H:1V to 0.25H:1V rock cuts up to 130 feet in height. The highly variable condition of the volcanic rock mass includes deeply weathered/ altered and very closely fractured basalt; adversely oriented, clay-infilled flow boundaries; and very strong tuffs with extremely persistent, adversely oriented, planar discontinuities. The latter condition resulted in a catastrophic rock slide in 1957, when the original construction for the interstate unsuspectingly undercut such structure.

The rock slope design includes untensioned pattern reinforcement concurrent with excavation and real-time monitoring for slope deformation, utilizing 1) automated motorized total stations (AMTS) and survey prisms and 2) strain gages on 100-foot-long sacrificial anchors. Data is telemetered to web-based platforms, allowing real-time monitoring with remote access as well as automated messaging in the event displacement or anchor load thresholds are exceeded. Periodic ground-based laser scanning supplements the optical survey and strain gage data.

Experience from the first four construction seasons has demonstrated the need for refinements to the rock slope designs to account for geologic conditions encountered and to mitigate deformation behaviour of the slopes. These design changes have taken advantage of the complementary monitoring systems to integrate surface and subsurface point displacement measurements with areal displacements reported by laser scanning. Discontinuity orientations, spacing and persistence measurements have been obtained from the laser scans. These activities have confirmed structural controls for the rock slope displacements and provided greater confidence in the modified reinforcement designs.
INTRODUCTION

Interstate 90 (I-90) within Washington State is the most heavily used east-west crossing of the rugged Cascade Mountains. Currently, average daily traffic is about 27,000 vehicles a day, with weekend and holiday traffic climbing to as high as 58,000 vehicles a day. By 2030, traffic volumes on this facility are expected to grow to an average of 41,000 vehicles per day. I-90 across Snoqualmie Pass is a strategic freight corridor serving local, national, and international markets and shippers. It is estimated that on any given weekday twenty five percent of the traffic consists of trucks, and on annual basis they carry an estimated 35 million tons of freight with an estimated value of $500 billion (US).

The 5-mile-long highway corridor between Hyak, located just east of Snoqualmie Pass, and the Lake Keechelus Dam is currently a narrow four-lane highway facility located immediately adjacent to Lake Keechelus and bounded upslope by steep mountainous terrain (Figure 1). Construction of a $551 million (US) infrastructure improvement project to upgrade this highway facility, administered by the Washington State Department of Transportation (WSDOT), was started in spring 2010. The project will improve this section of I-90 by adding one additional lane in each direction accommodated by about 800,000 cy of large rock excavations along the uphill side, constructing new bridges, reducing sharp substandard curves, repairing deteriorated concrete pavement, replacement of the snow avalanche shed, installation of snow avalanche fences, and stabilizing unstable slope conditions along the project alignment. When completed in 2018, the highway capacity through this section of Snoqualmie Pass will be increased by 50 percent in each direction of travel, resulting in reduced congestion. In addition, winter closures due to snow avalanches and needed control work will be reduced, and the risk associated with slope instabilities will
The project has been divided into three construction phases, 1A, 1B, and 1C, ordered from west to east. The major rock excavations occur within 1B and 1C, which are the sections located to the west and east sides of the existing snow shed, respectively (Figure 1).

The geologic challenges associated with the hill-slope excavations are considerable. During the initial construction of the interstate in the late 1950s, a catastrophic slope failure occurred within a large rock excavation at a location now referred to as Slide Curve (Figure 1). The cause of the rock slide was the undercutting of extremely persistent, adversely-dipping structure, which is also present within the existing cuts and natural slopes throughout the project limits.

**Site geology**

Bedrock in the western project area consists primarily of basalt flows, local pillow structure; silty sandstones and siltstones are locally interbedded. These rocks also exhibit low-grade metamorphism. The overall rock mass can be characterized as weak to moderately strong.

Bedrock in the eastern project area consists of volcanic tuffs, primarily pyroclastic flows with variable welding and extreme heterogeneity in their chemical and depositional characteristics. Frequency of flows, flow thickness and proximity of sequential eruptive centers affected the cooling rates of both individual flow units and successive deposits. The variability in the original composition of the tuff, degree of welding and in-situ porosity and permeability throughout the flow deposits is further complicated in the project area by subsequent low-grade metamorphic alteration. The tuffs are characterized as strong with unconfined compressive strengths typically between 5,000 and 15,000 psi, with a few test results exceeding 20,000 psi.

The range of discontinuities within the rock mass include faults, shear zones, joints, and lava and pyroclastic flow boundaries. Many of the encountered discontinuities have adverse orientations, high persistence, and/or weak infilling material. They pose significant design concerns due to their potential for day-lighting in the excavations and the fact that they have proven to be sufficiently unpredictable or ubiquitous in nature that they can exist anywhere within the cuts. Joints in the vicinity of Slide Curve exhibit persistence values in excess of 300 feet with adverse inclinations directed out of the proposed cut slopes.

**Slope design**

Rock slopes as high as 130 feet are required for the new alignment. Where the basalt rock mass is of lower quality and the natural slopes above the cut are favorable, the cut slopes were designed with inclinations ranging from 1H:1V (45°) to ½ H:1V (63°). However, throughout most of the project, traditional steep ¼ H:1V (76°) slopes were employed with the integration of slope reinforcement and mechanical methods for rockfall control. Slope stabilization design measures include untensioned rock dowels, tensioned rock bolts, PVC-cased drain holes, cable net drapery, reinforced shotcrete, and scaling.

Rock conditions in this volcanic terrain are highly variable and defied accurate characterization, irrespective of drilling and mapping intensity. Provisions were made to make design changes to rock slopes during excavation as actual rock conditions were encountered. Site geotechnical engineering during construction coupled with predictive slope displacement monitoring were included in the project to recognize and mitigate slope instability in a timely manner, minimize traffic interruptions and to provide for worker and public safety.
SLOPE MONITORING
An integrated set of displacement monitoring methods were employed:

- Point measurement of surface displacements utilized conventional surveying methods consisting of targets and a robotic total station.
- Point measurement of subsurface displacements utilized vibrating wire strain gages attached to passive, fully-grouted steel bars.
- Terrestrial LiDAR to interpolate between surface and subsurface point measurements.

The overall objectives of these systems were to retrieve accurate displacement measurements in near real-time and to be able to collect, transmit and receive the data via internet connection. A secondary consideration was the desire to set threshold levels of displacement that would initiate warning messages to designated monitoring personnel responsible for slope performance.

Automated Motorized Total Station (AMTS) / Prism System
Permanently mounted, optical glass prisms (generic term for targets) were deployed at an approximate horizontal spacing of 75 to 100 feet and an approximate vertical spacing of 20 to 40 feet dependent on local structural geology. The initial row was placed on bedrock outcrops above the top-of-cut with subsequent rows on alternate excavation lifts. Prisms were offset between rows.

Instrument towers were situated along the eastbound shoulder of I-90 such that the maximum sight distance to any prism was approximately 300 feet. Multiple AMTS instruments were required to operate at any given time to provide full-face displacement monitoring. It was necessary to mount the total station instruments on towers some 15 feet above roadway elevation to provide line-of-sight over truck traffic and to minimize interference from road spray. Monitoring continued on each rock cut until several months after excavation to final grade but was not required when the area was under snow cover during winter construction shutdown.

The x, y and z coordinates for each prism were measured to a contractually specified accuracy of +/- 0.2 inches in the x, y and z directions. Polling frequency ranged from 15 minutes for cuts undergoing active blasting and excavation to as much as 60 minutes after the cuts were completed.

A Leica AMTS with IRIS control program was configured to run on the Campbell Scientific Inc, CR800/1000 data logger as a gateway platform to control and store data from the AMTS. It used PakBus protocol to send data via spread-spectrum radio so that a common ADAS Base Station (also a CR1000) may be used. The geodetic data was to be retrieved by CSI LoggerNet automatic polling of the Base Station and FTP forwarded along with other data tables to an ARGUS or ATLAS web-based database.

Strain gage system
Instrumented rock dowels were installed along the crest of the highest rock cuts for both the Phase 1B and 1C projects. Typical spacing was on the order of 150 to 250 feet. Each installation consisted of a fully-grouted, 100-foot long steel bar (#9 or #14 bar) inclined at -15° and equipped with vibrating wire strain gages (VWSG) at five predetermined depths. The strain gages were factory-assembled and consisted of ½ inch diameter, 3 to 4-foot long “sister bars” that were clamped to the dowels with cable clips (Figure 2). This methodology obviated the requirement to remove epoxy coating from the
production bars and greatly facilitated field fabrication. In addition, the installation procedure exactly mimicked that of the production reinforcement thereby providing crew familiarity for drilling and grouting.

![Image of production bars with strain gages](image)

**Figure 2.** A. Sister Bar with Vibrating Wire Strain Gages (VWSGs) Attached to an Untensioned Rock Dowel  B. Schematic Deployment of VWSG Along Bar

The specified data logger specified was a model CR1000 provided by Campbell Scientific Inc. (CSI). It utilized CRBasic programming language and PakBus communications protocol compatible with the ATLAS web-based monitoring system. A vibrating wire interface (Model AVW200 provided by CSI) was required to read the strain gages, which employs FFT spectral analysis to measure the frequency output of the strain gages. This was connected to a 16-channel multiplexer (Model AM16/32). A modular autonomous photovoltaic power supply system was to be provided at the ADAS site. The solar power supply system was based on a CSI Model SP65 Solar Panel with a 12-volt photovoltaic power module (solar panel) capable of producing minimum 65 Watts of power; a deep cycle type battery with a nominal capacity of 100 AH (amp-hours), and a charge/load controller. The data logger, multiplexers, and other related and necessary components were housed in a lockable rainproof enclosure. The ADAS site included necessary components to allow remote communication with the data logger using a model RAVEN XTA digital-cellular telephone package. The data was transmitted to an ARGUS or ATLAS web-based database (Figure 3).
Terrestrial LiDAR

The specified surface and subsurface monitoring above was supplemented with terrestrial light distance and ranging (LiDAR) technology to monitor for surface deformation, using WSDOT’s short-range, Leica laser scanner (Figure 4A). The scanner has a resolution of about 6 mm at 50 m (0.02 ft. at 150 ft.). The scan data was processed with the software PolyWorks V11 by InnovMetric. A triangular irregular network (TIN) was first created with the initial scan data. The point clouds from subsequent scans were then comparatively analyzed against the initial TIN mesh to identify areas of surficial slope movement (Figure 4B). Several hundred meters of slope face could be scanned and data transmitted within a few hours, and data processing was usually be completed within an hour. Scans were not typically scheduled in advance but made upon request, generally after significant exposure of new cut faces or when the strain gages and/or AMTS detected slope distress.
Figure 4. A) Portability of Laser Scanner Allowed for Rapid Set Up and Data Collection. B) Comparative Analyses of Two Scans Showing Magnitude Movement in feet. (Positive values/warm colors indicate movement out of slope)

ROCK SLOPE DESIGN MODIFICATION EXAMPLE

Phase 1C Sectors IX & X

This 2000-foot long interval is comprised of continuous 0.25H:1V rock cuts up to 130 feet in height. The image analysis software “PolyWorks” enabled the determination of mean orientation, persistence, and geo-referenced location of targeted discontinuities present on the existing natural and cut slopes (Figure 5). Stereonet analyses of the entire structural database derived from televIEWer logging, Sirovision mapping and conventional mapping in comparison to the structural data derived from the PolyWorks data showed good agreement for wedge-forming joints defined as sets J1 and J3 (Figure 6).

Figure 5. “PolyWorks” Mapping from Terrestrial LiDAR
At issue was the potential size of wedges formed by the intersection of these two joint sets which in turn was governed by the persistence of the wedge-forming joints. From the PolyWorks analysis of the laser scan data, the distribution of the measured persistence values was compiled (Figure 7) which showed that at a 95 percent probability level, the measured persistence values for J1 and J3 joints were 78 and 45 feet, respectively. It is noted that remote sensing has more opportunity to underestimate persistence than to overestimate it due to the limited exposure slope height and to vegetative or soil cover.
Figure 7. Persistence Distributions for Wedge-Forming Joints from “PolyWorks”

The “design” persistence values, in combination with the planned slope cut height and orientation, and the natural slope inclination above the planned cut, defined the potential wedge size. This analysis is summarized in Figure 8 in which the J1 and J3 trace lengths on the planned slope face were conservatively adjusted to be slightly more than the persistence values quoted above. Above the cut it was assumed that the wedge would be truncated by a tension crack coincident with the orientation of J2 joints previously mapped during the design investigations. Since the location of the tension crack (J2 joint) was unknown, an arbitrary and conservative assumption of 80 feet measured along J1 was assumed. Figure 8 shows the resultant generic wedge in cross section. The important feature was the line of intersection (LOI) which has an orientation of $40^\circ/248^\circ$ (plunge/trend) and a length of 124 feet. The trend was nearly orthogonal to the planned slope face ($255^\circ$ dip direction) and the plunge was greater than the anticipated frictional shear strength on the joint surfaces. This meant that in the absence of cohesion on the joints or in the presence of groundwater pressures, an unreinforced wedge was probably unstable. As shown, the wedge height, measured vertically at the slope face, was 50 feet or about 40 to 50 percent of the planned cut slope heights.

It should also be noted that although the persistence values for both J1 and J3 were selected at the 95% level, there was no way of knowing whether these outlier members would be located in space so as to actually intersect and form a wedge. This conservatism was offset by the potential for blasting or other processes to cause less persistent features to coalesce and lead to a larger scale wedge. Accordingly,
engineering judgment was used to accept the generic wedge geometry shown in Figure 8 as the basis for design.

![Generic Wedge Geometry](image)

**Figure 8. Generic Wedge Geometry**

**Wedge Stability**

Having defined the design wedge size, the follow-on issue was to determine the required passive dowel reinforcement to provide the minimum WSDOT margin of stability (static Factor of Safety, FS = 1.25). The project-standard reinforcement consisted of 40-foot Grade 75 #9 bars typically spaced at 12 ½ feet horizontal by 12 feet vertical to accommodate drilling, blasting and excavation sequencing. Figure 9 upper shows the superposition of a generic “design” wedge on the reinforced slope face. Depending on the exact location, the number of dowels perforating the wedge face ranged from 12 to 14. For the lower value, the total force available to the wedge was approximately 700 kips (12 dowels @ 60 kips per dowel).

To analyse for stability, two cases for shear strength (friction angle and cohesion) on joints J1 and J3 were considered:

- **Case 1:** \( \phi = 38^\circ, c = 250 \text{ psf} \)
- **Case 2:** \( \phi = 38^\circ, c = 0 \text{ psf} \)

The former corresponds to the values routinely used on the Phase 1B and 1C projects while the latter represents an extreme condition potentially resultant from extremely deficient blasting practices. The analyses indicated the planned reinforcement pattern should be adequate for reasonable groundwater conditions. For the most probable shear strength combination (Case 1), even at the extreme transient groundwater condition corresponding to total wedge saturation, nominal stability was predicted. These analyses underscored the importance of the horizontal drains specified in the contract design.
The remaining issue related to the dowel length in comparison to the “design” wedge size. This evaluation showed that the dowels should be lengthened to a minimum of 50 feet at the crest of the wedge to provide adequate bond zone beneath the line of intersection (see Figure 8 Cross Section).

![Graph showing shear strength and reinforcing conditions](image)

**Figure 9.** Generic Wedge Stability Evaluation.

The intent of the reinforcement layout design was to graduate the dowel length from the top down utilizing project-standard horizontal and vertical spacing. The upper rows were lengthened (60 feet at 2630 elevation or higher) followed by two 50-foot rows. 40-foot dowels were specified for the next three rows followed by the lowermost row (one lift above final grade) that was shortened to 30 feet. The rationale for this layout was that the structural geology would not be recognized until the first few lifts were excavated and therefore the initial reinforcement should assume adverse conditions. The mid to lower rows could be tailored to predictable structural geology that would be mappable on the exposed cut faces. An interval of greater J1 persistence was identified in the vicinity of 1369+00 to 1373+00 LW. Accordingly, it was recommended that all bars above nominal elevation 2630 feet (to nearest half lift) consist of Grade 75 #14 bars in place of the #9 bars. This was an insurance policy.
against the possibility of a larger size wedge and in recognition of the steep natural topography above the cut line.

**Construction Experience**

Figure 10 illustrates the partially completed cut slopes as the excavation proceeded from project east to project west. The final rock cut slopes were in excess of 100 feet in vertical height requiring at least four 24-foot benches (excavation lifts for drilling, blasting and excavation). Pattern dowel reinforcement was installed after each half-lift of excavation, supplemented with spot dowels and/or tensioned bolts as required. It was observed that J1 joints occurred in the rock mass as both short persistence features defining a structural fabric as well as highly persistent “master joints” that transected the entire slope height. Consistent with the design assumptions for generic wedges, the stability of the majority of the cut slope was derived from the limited persistence of the complementary J3 joints. The exception to this generalization was a more closely jointed zone located beneath a swale in the natural mountainside (highlighted zone above man-lifts in Figure 10). This 300-foot long interval also exhibited J2 joints steeply-dipping into the slope, many of which were characterized by clay development up to several inches in thickness. Groundwater seepage was more prevalent than for adjacent slopes. During excavation, slope monitoring detected anomalous deformation trends that eventually necessitated localized redesign of the reinforcement system.

![Figure 10. Interim Slopes Showing Persistent J1 Joints.](image)

Highlighted central interval indicates rock cut below swale in which rock mass is more closely jointed and exhibited greater groundwater seepage. Location of SGD03 indicated.
Slope Monitoring

Fortuitously, instrumented dowel SGD03 was located at the crest of the slope in the central area of the swale. Figure 11 summarizes the 2013 dowel load and blast history for the slope interval proximal to SGD03. The dates for all blasts within 100 feet of SGD03 are shown along with the bench number that was being shot. On Figure 11 the blasts are differentiated as presplit only, bench only, or presplit & bench. The noteworthy features of the 2013 and previous year’s load and blast histories were:

- The access ramp shots on 9/5 and 9/6, 2012 produced an 8 kip load at depth of 50 feet on SGD03.
- Strain gages at all depths reported increasing load over a one week period at the end of October 2012. At a depth of 50 feet the load increased by approximately 7 kips during this period. No construction activity was associated with this load increase.
- The 2013 incremental load reactions at 50 and 70-foot depths became progressively larger as the slope height increased (lower bench numbers).
- All load changes in 2013 appeared to be related to blast events. Following blast events, loads asymptotically approached a higher plateau value.
- The gages indicated the load on SGD03 was located between 50 and 70-foot depth. All other gages (<50 feet and >70 feet) showed nominal loads less than a few kips.

Figure 11. SGD03 Dowel Load History Related to Blasting Events.
Each Bench Represents 24 feet Above Final Ditch Grade.
Based upon the load sensitivity to blasting and the occurrence of a 7-kip instantaneous load increase at 50-foot depth on August 28th, 2013, apparently unrelated to blasting or excavation, the area of concern was designated “yellow alert”. This constrained the contractor’s activities in the area until a more detailed site specific stability evaluation could be undertaken. Blasting was suspended on September 16, 2013. Subsequent PolyWorks analyses identified and quantified J1 joints #1321 and #1322 as well as other major discontinuities.

Figure 12 shows in cross section view the sequential load reaction of SGD03 to the excavation of the adjacent slope. For the purposes of this schematic, the precise timing and duration of the blast muck removal activity was not known. Therefore, the dates shown for the benches are those at which the bench (i.e. production) blasting was completed. Noteworthy observations from Figure 12:

- The initial loads on SGD03 – 50 feet occurred during access construction and Bench 4 excavation. At this stage, the slope cut was at a higher elevation than the strain gage reporting the load increase.
- The progressively greater load increases as the bench elevation was lowered occurred in spite of the pattern reinforcement installed.
- As the bench elevation was lowered from Bench 3 to Bench 2 the load increase migrated down the gage bar to 70-foot depth.
- Plane 1322 (J1 joint with PolyWorks orientation 36°/211°) intersected SGD03 at a depth of 23 feet. This depth was too shallow to account for the observed loads.
- Plane 1321 (J1 joint with PolyWorks orientation 40°/209°) intersected SGD03 at a depth of 72 feet. This feature was probably associated with observed load increases.

![Figure 12. Load History SGD03 Related to Blast and Excavation Sequence.](image)
Figure 13 summarizes the vector displacements for prisms situated proximal to SGD03 superimposed on a change analysis from successive LiDAR scans for a similar one month period. The vector values include the color-coded 3D displacement (inches) and the plunge and trend of the movement vector. The change analysis is plotted in decimal feet. Salient observations:

- Three prisms (shown in red) reported 3D survey displacements greater than 0.3 inches. The corresponding LiDAR displacements for the same prisms were 0.72 to 0.48 inches indicating reasonable correlation.
- The vector orientations of movement for the three prisms were -03°/272°, -37°/241° and -14°/242°, also reasonably consistent with the J1/J3 wedge LOI (-35°/237°) suggesting a wedge mechanism for movement.
- The interval of slope deformation appeared to be limited to the west by Plane 1321 (1367+00 LW) and to the east between P127 and P125 (±1368+75 LW). Coincidentally, this was the only interval in Sector X that did not include pattern #14 dowels for the upper rows.

Figure 13. Comparison of Prism Displacements with Laser Scan Deformation Analysis.
Based on this ongoing slope deformation, the magnitude and depth of the measured loads, the consistency between the three monitoring techniques and the compatibility of the measured displacements with the structural geology, a program of supplemental slope reinforcement was recommended for immediate installation. Site specific analyses for the inferred wedge at 1368+00 LW indicate that 4000 kips of passive reinforcement should be installed along with slope drains to adequately stabilize the overall slope. In addition, a second instrumented dowel (SGD30) was installed at approximately mid-height below SGD03 to provide information on subsurface load accumulation for the two dowels on a common cross section. Figure 14 shows the supplemental reinforcement as well as the secondary instrumented dowel. It was interesting to observe that the lower dowel (SGD30) started to accumulate load within days of installation and at the depth corresponding to the intersection with PolyWorks plane 1321 (Figure 14).

Figure 14. Supplemental Reinforcement Design for “Yellow Alert” Area.

Note insufficient lengths of original lower capacity #9 Type “L” bars relative to projection of plane 1321. Mitigation also included multiple PVC-cased horizontal drains up to 100 feet long.

Post Reinforcement Slope Performance

During the month of September 2013, the project received essentially zero precipitation. Thus all the measured slope deformation was related to blasting and excavation activities leading up to the suspension of work on September 16, 2013. Between September 28 and October 1 a major storm event resulted in greater than 9 inches of precipitation as reported for a weather station a few miles to the
west. The supplemental reinforcement and drainage program had just been initiated and the work had to be suspended due to dangerous working conditions. This storm was followed by six weeks of very dry weather followed by a second slightly less intense storm on November 17th. Fortunately all the supplemental reinforcement and drainage was installed between these two storm events thereby providing an opportunity to compare the load reactions for the two storms.

On Figure 15, the time scale for the two storm events was matched by aligning the lag time between the start of precipitation and the initiation of load increase on dowel SGD03 50-foot depth for each storm. The lag time was approximately 85 hours and was clearly the result of hydrostatic loading within discontinuities penetrated by the instrumented dowel. The Oct 1 storm produced a steep increase in load of greater than 7 kips within a 24 hour time period. Conversely, the Nov 17 event produced an increase of 1.5 kips and reached a plateau value within 24 hours. This was attributed to the combined effects of more reinforcement to absorb the hydrostatic loading and to the beneficial effects of horizontal drains.

![Figure 16. Comparison of Load Reactions to Storms Before and After Supplemental Reinforcement and Drainage (SGD03-50 feet)](image-url)
The ultimate test for the supplemental work will be experienced mid-year 2014 after the slopes have had the opportunity to drain following the winter snow melt. It is anticipated that the hydrostatic loading component of the total loads being reported by the instrumented dowels will decrease. The blasting and excavation of the final lowermost bench will be the final confirmation of the adequacy of the supplemental reinforcement program.

CLOSURE

The monitoring program for the I-90 rock excavations has been largely successful in identifying areas of slope deformation, informing necessary modifications for excavation sequencing and for additional slope stabilization, and for “real-time” safeguarding of construction personnel and the traveling public. The integration of surface and subsurface displacement measurements with terrestrial LiDAR has greatly expanded the monitoring capabilities of the AMTS/prism and strain gage systems by providing critically important monitoring redundancy and slope deformation data between non-instrumented portions of the cuts. The strain gage system has proven to be both highly reliable and sensitive to loading induced by blasting, excavation, and groundwater infiltration. Accurate slope displacement measurements coupled with geo-referenced discontinuity mapping from LiDAR facilitated interpretation of structural controls and design (or redesign) of supplemental reinforcement strategies.
Wicking Geosynthetic Used for Frost Heave Prevention Pioneer Mountain Scenic Byway

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ABSTRACT

Problem
The Pioneer Mountain Scenic Byway, located just 30 miles West of Dillon Montana, seasonally experiences frost heave over a 20 mile stretch of roadway. The road is closed to winter traffic and typically opened in May. Longitudinal cracks are commonly found. The Moose Park location experiences the most dramatic problems and had been recently repaired in 2004.

Solution & Design
Federal Highways subsurface investigations indicate a high water table and a variable subgrade strength consisting of silts and pit run materials. The preference is to minimize over excavation and looked to geosynthetics as an option. The new wicking geosynthetic provided by TenCate was recently used on an Alaska DOT project with early indications of success. Three design solutions were proposed and presented to the county for review. The dual layer solution was selected to minimize the cross section depth and keep the wicking geosynthetic above the water table to mitigate the frost heave effects.

Construction
In the fall of 2013, the site was excavated to the required depth finding a mix of materials to include 4 drain tiles and miscellaneous geosynthetics used in previous repairs. The first layer of wicking geosynthetic was installed and covered with select pit run aggregate. Early wicking was witnessed shortly after installation. The project was put on hold as snow started falling and temperatures dropped at this 7625 foot elevation. The final installation will be completed in early summer of 2014.
INTRODUCTION

Pioneer Scenic Byway near Dillon, Montana is a seasonally open road travelled by tourists, ranchers, snow enthusiasts and makes a connection between Polaris and Wise River, Montana. The road traverses through an elevation differential of 2000-7500 ft and grants access to locations such as Maverick Mountain Ski Resort, Elkhorn Hot Springs, and the Gem Field.

The road is typically cleared from snow in late April-early May (Figure 1). Once uncovered, a longitudinal crack forms during the freeze-thaw cycle (Figure 2), and extends for over 20 miles, some of which are wider than a motorcycle tire and requires immediate maintenance attention for safety. The Moose Park location displays the most severe of these cracks generated by freeze thaw.

Figure 1 – Road Clearing
Figure 2 – Longitudinal Crack, Moose Park
Construction 2004

In 2004, the repair of the Moose Park location utilized traditional methods to combat frost heave. This included sub-excavation, geotextile, and a drain system. (Figure 3)

In 2008, the longitudinal cracks began re-appearing, and maintenance sub-excavation and chip sealing was used to repair the sections.

DESIGN

Background

In discussions with Federal Highways, Western Federal Lands in Vancouver, Washington, a geosynthetic solution would be reviewed. A new wicking geosynthetic from TenCate was recently used in Alaska to mitigate the effects of frost heave. Early testing conducted by Xiong Zhang, at the University of Alaska Fairbanks, Beaver Slide, provided the design guidance, (Figure 4).
Another project of a much larger scale was installed in 2012 by the Alaska DOT on the Dalton Highway 30 miles north of Coldfoot, Alaska. This road experiences severe frost heave and early success of the geosynthetics wicking capability was seen during the first spring break-up. (Figure 5)
Data such as borings, piezometer, and a gradation analysis were provided by FHWA to come up with design scenarios. Design constraints were listed as the following.

- Budget
- High water table
- Blend in with the surroundings (important for daylight considerations)

These items were reviewed with associate engineers from TenCate. The wicking geosynthetic (Mirafi\textsuperscript{®} H2Ri) also performs sub-grade stabilization, however needs to be located above the water table to perform wicking and act as a capillary break to the pavement structure.

Three solutions to repair the 1,100 feet of Moose Park roadway proposed by WFLHD are shown in Figure 6. Option #1 would not be an improvement over the existing roadway design conditions. Although better, Option #2 would not significantly extend the pavement life. The option chosen was Option #3. This option is anticipated to result in “less differential deformation at the surface, fewer surface cracks and lower long-term maintenance cost” according to WFLHD. The estimated repair costs were $475,000, $585,000 and $675,000 for options #1, #2 & #3 respectively.
The maintenance of the Scenic Byway roadway falls to Beaverhead County which in addition to this roadway has over 1600 miles of county roads spread over 5560 square miles to maintain with an annual road budget of less than $2.4 million. Beaverhead County’s matching share for the three options ranged from $70,000 for the no improvement option #1 to $99,650 for the extended longevity associated with Option #3.

Beaverhead County determined that the minimal increase in matching funds far out weighted the added long-term maintenance costs associated with the other two options. In addition upon reviewing the initial costs estimates for the three option, that due to the hauling of the aggregates to the remote site location, the actual cost difference in construction between the options was only the costs of the geosynthetics. This reinforced the decision to install the two layer system; a wicking geosynthetic layer on top of the subgrade fill and a second layer of geosynthetic under the base aggregates.
Construction began in October 2013 by the Beaverhead County maintenance department. Excavation uncovered the drainage pipe and geosynthetic that was utilized in 2004.
Beaverhead County Engineer was onsite along with technical representation from TenCate. Often geosynthetics are installed with a minimum overlap requirement. In the case of the wicking geosynthetic, the overlaps were installed “shingle style” so as to optimize water movement to the sides of the road. In super elevation cases, the material was overlapped from one side of the road to the next (Figure 7), and for a typical crown, the material was installed with the middle roll at the highest point similar to a roof shingle. (Figure 8)
The select borrow was placed in a compacted 10” lift. At this point the project was put on hold through as winter in Montana began to set in. Final construction is expected in June of 2014.
REFERENCES:


Title:
Location of Sub-Asphalt Voids Under US-34 After Severe Flooding using Nondestructive Evaluation Techniques

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ABSTRACT

In September of 2013, the Front Range area of Colorado experienced a series of extreme rain events, which resulted in severe flooding in a number of mountain canyons. These flooded canyons included the section of US-34 west of Loveland known as “The Narrows”. The section of US-34 in The Narrows was previously washed out by a major flood in 1976, and rebuilt using an anchored retaining wall system protected by riprap/boulders on the river side. During the flooding of 2013, the riprap and boulders were washed away and portions of the retaining wall were undermined by the water, resulting in washing away of the wall backfill from underneath and collapsing the asphalt pavement. This paper presents the results of an investigation conducted on the visually intact sections of the roadway to determine if there were any areas of undermining that did NOT result in visible pavement collapses. The investigation was carried out primarily with the Ground Penetrating Radar (GPR) test method, with additional information from other Nondestructive Evaluation (NDE) methods. The roadway was scanned from the surface, with no drilling or other intrusive sampling required. The results of the investigation showed one significant area of apparently intact pavement that had a major void underneath. The paper will present the typical testing done, and overview of the test methods, sample data showing both sound and voided areas, and photographs of the roadway and voided area after excavation.
INTRODUCTION

This paper presents background, investigation, and results of a nondestructive testing (NDT) investigation of a 1 to 2 mile section of Highway US-34 outside Loveland, Colorado, in a section known locally as “The Narrows” due to the narrow, winding canyon the roadway follows alongside the Big Thompson River, as shown in the photo below presented as Figure 1.

![Big Thompson Canyon in The Narrows](image)

In September of 2013, exceptionally heavy rains along the Front Range area of Colorado resulted in extensive flooding and flood-related damage. This damage was especially severe along highways adjacent to streams and rivers, such US-34 alongside the Big Thompson River. An important historical note for this section of the river is that this section of US-34 in The Narrows was previously washed out by a major flood in 1976, and rebuilt using an anchored retaining wall system. Retaining walls were founded on drilled shafts into bedrock. Steel H-piles extending up from the shafts were anchored to the canyon wall with steel threadbar tiebacks protected by PVC sleeves. Above the pier cap, precast concrete panels were installed between the H-piles cap and topped with a concrete sill. Concrete curbing and steel guardrail was installed above the sill. The reconstructed road was constructed on top of the silty sand and gravel wall backfill, and the outside toe of the retaining wall was protected by riprap/boulders. During this most recent flooding event, the riprap and boulders that protected the base of the wall were washed away. Weathered bedrock
was scoured from below the pier cap, resulting in a gap that allowed the wall embankment fill to be eroded away by piping between the shafts. The gap is visible in Figure 2.

![Retaining Wall with Gap Visible at Base](image)

As the wall backfill was eroded, voids propagated up behind the wall and the asphalt concrete collapsed into the cavities. A photo of a typical visible undermined and collapsed area of the roadway is shown in Figure 3. As seen in the photo (Figure 3), the retaining wall itself is undamaged, but all of the fill material supporting the roadway in the lane closest to the river has washed out. The tieback anchors that provide lateral restraint for the wall are clearly visible in the photo, and were also found to be relatively undamaged, other than the PVC casings. In some areas, the retaining walls were overtopped by the Big Thompson River and water flowed over the road, depositing debris on the pavement, guard rails, and in the collapse zones.
The washouts and subsequent collapse of the roadway sections in The Narrows were apparently caused by water flowing underneath the base of the retaining wall in a gap formed above the bedrock and below the retaining wall base. This gap was exposed in a number of areas when the riprap and other protective materials were washed downstream by the extreme force of the water flow. A photo showing the typical exposed gap area below the retaining wall in a damaged roadway section is shown below in Figure 3.

After the flooding receded, a number of collapses were observed at various locations along US-34 in this section. However, there were also fairly long stretches of highway that appeared to be relatively undamaged and unaffected by the flooding. In the interest of caution and to ensure that no potential damage was missed, a nondestructive evaluation (NDE) investigation was requested to look for possible hidden voids or other defects underneath the apparently undamaged sections of US-34.

This investigation was specifically requested to look for possible voids that had not propagated to the surface and/or caused collapse of the asphalt mat. The investigation focused on the roadway lane closest to the concrete retaining wall along the river, where the highest likelihood of hidden damage was expected. The fieldwork was performed by personnel from Olson Engineering over the course of two field days using primarily the Ground Penetrating Radar (GPR) test method, with limited testing by the Spectral Analysis of Surface Waves (SASW) method.
performed to provide additional supporting information. The test results from this investigation are discussed below followed by a discussion of the GPR and SASW test methods.

INVESTIGATION PROCEDURE AND METHODS RESULTS

The investigation was carried out primarily with the GPR test method, using both 400 MHz and 200 MHz antennae connected to a GSSI SIR-3000 GPR system. The use of two antennae provided a good range of sensitivity, resolution, and depth of penetration. The GPR testing was conducted by moving a given antenna along one of a set of three scan lines located at different distances from the retaining wall. The first scan line was located as close to the wall as possible, the next was at the center of the lane closest to the wall, and the third was near the centerline of the highway. As part of the investigation, supplementary tests were also conducted with the SASW test method to look at the shear wave velocity versus depth profile of selected “slices” across the roadway. As noted, GPR scan lines and SASW tests were only conducted in areas with apparently sound roadway conditions and no visible damage. As part of the investigation, GPR scans were purposely run over known void areas at the end or beginning of scans to verify the signature of typical voids under this asphalt.

Ground Penetrating Radar (GPR) Test Method

The GPR method involves moving an antenna across a test surface while periodically pulsing the antenna and recording the received echoes, as diagramed in Figure 4. Pulses are sent out from the GPR computer driving the antenna at a frequency range centered on the design center frequency of the antenna, in this case 200 and 400 Megahertz (MHz). These electromagnetic wave pulses propagate through the material directly under the antenna, with some energy reflecting back whenever the wave encounters a change in electrical impedance, such as at a rebar or other steel embedment or water/air-filled void. The antenna then receives these echoes, which are amplified and filtered in the GPR computer, and then digitized and stored. A distance wheel records scan distance across the test surface and embedded features can be located at a given distance from the scan start position. For repetitive scanning, a standard survey is designed and adhered to as field conditions allow to minimize mistakes and maximize data quality.

The scans for this investigation were created from pulses sent out at lateral intervals of approximately 24 pulses per foot. The resulting raw data is in the form of echo amplitude versus time. By inputting the dielectric constant, which defines the material velocity, and by estimating the signal zero point, the echo time data can be converted to echo depth. The following equations explain this conversion:
\[ V_{EM} = \frac{c}{M_r^{0.5}} \quad \text{Equation 1} \]
\[ D = \frac{(V_{EM} \cdot T)}{2} \quad \text{Equation 2} \]

Where \( V_{EM} \) is the material electromagnetic velocity, \( c \) is the speed of light (in air), \( M_r \) is the material relative dielectric constant, \( D \) is depth, and \( T \) is the two-way radar pulse travel time. A typical asphalt and granular soil dielectric constant of \( M_r = 6 \) was assumed for the data in this investigation. If more accurate depth data is required, a depth calibration can be done if an embedment of a known depth is available to scan over. The scans are then typically plotted as waterfall plots of all of the individual data traces collected, with the lightness or darkness (or color) of each point in the plot being set by the amplitude and polarity (positive or negative) of the data at a given depth in each trace. A photograph of GPR surveying with the 400 MHz antenna is presented in Figure 5, showing scanning along the shoulder of the roadway near the wall.

![Photograph of GPR Scanning with the 400 MHz Antenna](image)

**Figure 5** Photograph of GPR Scanning with the 400 MHz Antenna
Spectral Analysis of Surface Waves (SASW) Test Method

The SASW method is based upon measuring surface waves propagating in layered elastic media and is illustrated in Figure 6. The ratio of surface wave velocity to shear wave velocity varies with Poisson's ratio. However, reasonable estimates of Poisson's ratio and mass density for soils and other materials can normally be made with only a small effect on the accuracy of the determined shear wave velocity profile. Knowledge of the shear wave velocity combined with reasonable estimates of mass density of the material layers allows calculation of shear moduli for low-strain amplitudes.

Surface wave (also termed Rayleigh or R-wave) velocity varies with frequency in a layered velocity system. This variation in velocity with frequency is termed dispersion. A plot of surface wave velocity versus wavelength is called a dispersion curve.

The SASW tests and analyses are generally performed in three phases: (1) collection of data in situ; (2) construction of an experimental dispersion curve from the field data; and (3) inversion (forward modeling) of the theoretical dispersion curve, if desired, to match theoretical and experimental curves so that a true shear wave velocity versus depth profile can be constructed. Wavelength ($\lambda$), frequency ($f$), and wave velocity ($V_r$), are related as follows:

$$V_r = f \times \lambda$$

Surface wave dispersion can be expressed in a plot of surface wave velocity versus wavelength. This type of plot is used in this report. The dispersion curves analyzed during the course of this investigation were looked at for the typical shape of the curve, with a high velocity from the asphalt expected at short wavelengths (shallow depths) followed by a relatively gradual drop into the velocity of the fill and finally a velocity increase at longer wavelengths due to the bedrock affects. In the void area, the velocity just below the asphalt dropped to near-zero due to the

Figure 6  Schematic of field setup for typical SASW testing.
presence of an air void. Some signals were able to get “around” the void on the sides and below, but the apparent velocities were very low compared to areas with no void.

INVESTIGATION RESULTS

The NDE investigation confirmed that most of the visibly undamaged areas of the roadway were indeed sound. However, the testing located one large, severe sub-asphalt void in an unexpected area. This void was in a visibly undamaged area, and was found from initial testing with the 400 MHz antenna to extend over 20 feet in length and at least 6 feet out from the wall edge. Retesting this section several days later with the 200 MHz antenna showed that the void area had apparently grown over several days, although the roadway pavement was still undamaged with no surface manifestation of the underlying void. A photo of the visibly undamaged roadway over the large void area is shown in Figure 7. Note that the painted marks in this photo are based on the initial 400 MHz GPR results. Testing 3 days later with the 200 MHz antenna showed an even larger void.

Figure 7 Undamaged Roadway Surface Over Void Showing Initial Approximate Void Area
The apparent void area located by the GPR scanning was subsequently verified by removal of the asphalt, and found to be somewhat larger than even the second set of GPR results appeared to indicate. While it is possible that the GPR testing underestimated the extent of the void, it is considered to be more likely that there was a continued expansion of the void due to progressive collapse of material under the asphalt during and after the GPR field testing. As noted above, the initial 400 MHz testing showed the smallest extent of the void area. The 200 MHz testing conducted 3 days later showed an apparently larger void area, either due to the deeper penetration depth, additional collapse of the void area, or both.

The SASW test method was also used along this stretch of roadway, in support of the GPR test results. The SASW results did not indicate the presence of any voids not seen with the GPR results, but did give a good overview of the shear wave velocity profile of the fill material and bedrock. It also provided confirmation of the lack of apparent voids in a few areas that had unusual features in the GPR data.

GROUND PENETRATING RADAR (GPR) SAMPLE TEST RECORDS

Several sample 200 and 400 MHz GPR scans from the data collected during this investigation are presented in Figures 8 through 10 below. For each scan, the vertical axis represents the depth into the subgrade and the horizontal axis represents the distance (in feet) from the beginning of the scan. The scans presented in Figures 8 and 9 are from scans over the severe void located by the NDE along Wall 3. As seen, there is a very large, severe near-surface disturbance evident in both scans, with the 200 MHz antenna showing a more severe void response and a larger lateral extent. Note that these scans were taken in opposite directions over the void area, and were also taken with different vertical and horizontal scales.

In Figure 10, an example 200 MHz GPR scan is presented that was collected in an area with no apparent hidden damage. This scan shows very clearly the regular, even layering of the subgrade material, but also shows a series of clear reflectors. The evenly-spaced parabolic reflectors seen in this scan taken near the retaining wall are at about the same spacing as the H-pile supports for the precast panels in the retaining wall, and therefore also match the approximate spacing of the steel rock anchors. Based on the even depth and spacing, as well as the regular parabolic shape, these reflectors are indicated to be from planned subgrade features and not from voids. They do give an indication of the typical depth of penetration of the 200 MHz antenna as well as the sensitivity to buried features. Note that similar reflectors are seen in many of the other scans, and thus were also not included in the results tables.
Figure 8 400 MHz GPR Scan Near the Wall Along Wall 3, Showing Void Initial Area

Figure 9 200 MHz GPR Scan Near the Wall Along Wall 3, Showing Larger Void Area
Figure 10 Sample 200 MHz Scan Showing Tieback Anchor Reflectors

Repeating Deep Features: Tieback Anchors
SPECTRAL ANALYSIS OF SURFACE WAVES TEST RESULTS

The Spectral Analysis of Surface Waves (SASW) test method was used to evaluate the relative velocity of the asphalt/subgrade/bedrock system under selected visually undamaged areas of the roadway. Tests were performed in random points along the roadway, as well as in areas where anomalies or unusual responses were located with the GPR testing. These areas included a test directly over the confirmed sub-asphalt void. The results in this area showed a clear difference in the response profile over the void compared to nearby sound areas.

Sample SASW dispersion curves (showing surface wave velocity versus wavelength, related to depth), are presented in Figures 11 and 12 below for tests in the area of the void. Figure 11 presents the results from an SASW test over a sound area of the roadway near the void but not over it. As seen, there is a high velocity seen at shallow depths associated with the asphalt, followed by a relatively gradual drop in velocity due to the influence of the lower velocity fill material. The velocity quickly starts to increase again as the wavelengths extend into the bedrock depth.

In Figure 12, a result from a test near or over an apparent subgrade void is shown. Again, there is a high velocity at short wavelengths (shallow depths in the asphalt), followed by a very abrupt drop. This drop is much more abrupt and ends at a significantly lower velocity than that seen in Figure 11, and is typical of the type of result seen in SASW tests over voids. The low velocity zone also extends for a greater wavelength range compared to the apparent sound area. Forward modeling of these two areas to get a shear wave velocity versus depth profile for each shows the high velocity asphalt at shallow depths. However, the void area then shows a drop to a very low apparent velocity in the void area (likely some signal is carried by the nearby retaining wall and opposite uncollapsed fill zone), while the test in the sound area showed more typical asphalt/fill/bedrock profiles.
Figure 11  SASW Dispersion Curve, Wall 4A Centerline (center divider line), Sound Subgrade
Figure 12  SASW Dispersion Curve, Wall 3B, Over (or near) Void Under Asphalt.
CONCLUSIONS

The NDE investigation was conducted with two different test methods to look for evidence of hidden voids under the roadway. As seen in the examples above, the GPR test method worked very well for this task, with the 200 MHz antenna found to be the most effective at locating the void area as well as mapping other features in the roadbed material. The SASW testing was done to support the GPR testing, and provided additional information about the shear wave velocities of the various layers in sound areas. In general, the SASW test method would be considered to be too slow to be used for large-scale mapping of shallow voids. The GPR method can be conducted at a walking pace (or faster) and was therefore used to evaluate relatively long stretches of roadway in a relatively short period of time.
Geologic Causation and Mitigation Design for the US-89 Landslide Disaster, Bitter Springs, Arizona

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ABSTRACT

Large ancient rock landslides or "megaslides" exist throughout much of the Grand Canyon area in Arizona. On February 20, 2013, a 500-foot-long section of pavement failed, closing US 89 in the Bitter Springs area. The Echo Cliffs of Northern Arizona consists of steep Triassic sedimentary rock that has been tectonically uplifted into a large monocline. Vertical relief of the cliffs is 1,500 feet. Slope failures along part of the Echo Cliffs are unique because they occur in the upper 1,000 feet and do not extend to the base of the cliffs.

Geomorphic mapping and extensive subsurface exploration indicates that the cliffs initially went through a period of extensive debris slide failures (Pleistocene). Successive erosion and exposure of Chinle Formation claystone, along with the wet Pleistocene climate, resulted in the formation of large translational megaslides. Individual blocks are thousands of feet wide and range from 300- to 500-feet-thick.

This translational failure is part of the toe of the ancient megaslide. Failure occurs along a relatively shallow dipping slip surface. Subsurface exploration indicates that, the re-activation of the landslide occurred under "dry" conditions. Re-activation of the landslide is considered to be due to long-term and episodic erosion at the base of the megaslide as well as long-term creep and plastic deformation of the Chinle Formation. To rebuild, a new road alignment shifted just east of the active slide, an extensive rock cut and subsequent massive rock buttress fill with an estimated volume of 1,000,000 yd$^3$ is proposed to stabilize the landslide.

Keywords: Megaslide, Translational, Rock, Re-activation, Chinle Formation.
INTRODUCTION

A landslide that occurred on February 20, 2013 on US Highway 89, just north of Mile Post marker 526 near Bitter Springs, Arizona, is a reactivation of a portion of the toe of a larger, ancient megaslide. Although there is no specific definition for “megaslide”, for the purpose of this discussion and general consensus, megaslides are hereby defined as landslides with overall areal dimensions of several hundreds to thousands of feet and depth/thickness of 100 to several hundreds of feet.

Figure 1 - Oblique Aerial View of 500 feet of Displaced Highway

Closure of the highway prompted a detailed geologic and geotechnical investigation of the subject area. The headscarp of the re-activated landslide disrupted approximately 500 feet of roadway and failed across all travel lanes (Figure 1). The headscarp area is characterized by a series of open, tension cracks that extend vertically into the ground for several feet. Maximum apparent vertical offset in the headscarp area within the road is on the order of 4.5 feet (Figures 2 and 3).

Figure 2 and 3 - View of Damaged Highway Looking South and North, Respectively.
The larger ancient landslide measures approximately 2,500 feet across. The base, or toe of the active landslide, is approximately 1,200 feet across and is characterized by multiple, easily traceable ground cracks and pronounced bulging and buckling. The body of the active landslide is characterized by a series of ground cracks (some with voids extending up to 30-feet-deep), extensional (tensional) graben features and vertically displaced/warped bench-like platforms. Locally, eroded graben features and healed surface cracks indicate that landslide displacement has occurred in the past. The general surface features of the landslide are shown on Figure 4.

**Figure 4 - Oblique Google Earth Image (base) Showing Active and Ancient Landslide Dimensions**

The subject area has a unique geologic and geomorphic character, compared to exposures of the Echo Cliffs farther south of this site. This portion of Echo Cliffs is characterized by a series of ancient and highly eroded translational (planar failure surfaces sub-parallel to the ground surface) “megaslides”, which have formed between the cliff top and a pronounced midslope bench (located below and west of the highway). These megaslides are not rotational landslides and do not “toe-out” on the valley floor, which is common elsewhere in the Grand Canyon region. The megaslides appear to predominantly fail through and toe-out into, the clay-rich Chinle Formation.

**REGIONAL GEOLOGY**

The site is located within the Colorado Plateau Physiographic Province of Arizona. This province is characterized by great thicknesses of flat to gently-dipping Paleozoic and Mesozoic sedimentary strata that are deeply eroded into scenic plateaus, cliffs and canyons (1, 2, 3). In many locations the otherwise continuous sedimentary sequences have been interrupted by faults and/or uplifted into generally north-south trending monoclines, domes and folds. Extrusive and intrusive igneous features are found intermittently throughout the province (4).
The site is located within the Echo Cliffs area which presumably was formed as a result of uplift along the Echo Cliffs Monocline, the axis of which runs through the site in a north-south direction. The base of the cliffs west of the highway is the eastern edge of the Kaibab Plateau, which exposes the Kaibab Limestone at the surface. Tectonic forces have uplifted and tilted formerly horizontal strata in the Echo Cliffs area, toward the northeast.

For most of its length, the Echo Cliffs exist as a generally unbroken wall of stratified rock with marker beds that can be observed for miles. In this vicinity, several faults have offset and broken up the strata. In addition, vertically displaced/uplifted weak bedrock has resulted in massive landsliding which has created a bench-like feature. This bench feature, upon which US 89 was constructed, allowed for a more gradual gradient transition from the valley floor, across the face of the Echo Cliffs, to the upper Kaibito Plateau.

LOCAL GEOLOGY

The project site has been mapped by Cooley, et al. (5) and Billingsley and Priest (6). Both references generally agree that the site vicinity is underlain by stratified Triassic to Jurassic age sedimentary bedrock sequences.

Cooley, et al. (5) generally show only bedrock units in the site vicinity while Billingsley and Priest (6) show much of the bedrock covered by more recent (Quaternary) massive landslide and talus deposits. Both maps show the valley floor to the west being underlain by the Permian age Kaibab Formation. The ascending stratigraphic sequence includes the Early Triassic Moenkopi Formation, Late Triassic Chinle Formation and the Early Jurassic Glen Canyon Group which is composed of the Moenave Formation, Springdale Formation, Kayenta Formation and the Navajo Sandstone (6). The Chinle Formation in this area is further divided into the basal Shinarump (conglomerate) Member (which forms a distinct cliff/bench west of the highway), Petrified Forest Member and the Owl Rock Member. The upper portion of the Echo Cliffs is capped by the distinctive white, massive and cross-bedded Navajo Sandstone.

FAULTS AND SEISMICITY

Cooley, et al. (5) and Billingsley and Priest (6) both show multiple fault traces immediately south of the site where US 89 veers eastward from US 89A. Cooley, et al. (5) labels this as the Bitter Spring fault, with uplift on the northwest side. There is an apparent right-lateral displacement of approximately 2 miles (as measured from the plateau edge) of the Echo Cliff along this fault trace. This fault is not considered active (movement within the last 10,000 years).
GEOLOGIC INVESTIGATION

Geologic Mapping and Aerial Photograph Interpretation

Geologic/geomorphic mapping of the site and vicinity was performed during several visits between February 24 and April 11, 2013. Geologic reconnaissance also included an initial helicopter aerial reconnaissance on February 25, 2013, through courtesy of the Arizona Department of Public Safety (DPS). Aerial photographic interpretations were conducted throughout the study period on electronic images flown especially for development of a topographic map for this project (post slope failure) as well as Google Earth images (pre-failure dated 8-19-2010), which allowed interaction and manipulation of the view angle for enhanced imagery and terrain interpretation.

The purposes of the geologic mapping and aerial photograph interpretation varied from initial emergency assessment, identification of selected subsurface exploration sites, general geologic type and structure characterization and landslide mapping. Primary emphasis of the geologic mapping was to identify and assess slope instabilities in the immediate site vicinity and to prepare a geologic site plan for the site.

Identification and mapping of geomorphic features were utilized to determining the overall limits of both ancient and active landsliding, as well as determining the type and failure mechanism of sliding. Of prime importance was the recognition of numerous extensional/tensional pull-apart grabens throughout the active and ancient portions of the landslide, which indicate that this is a translational landslide (Figures 5 and 6).

Figure 5 – Tensional Pull-Apart Graben. Approximately 30-feet-wide Across the Body of the Active Landslide.
Subsurface Exploration

A subsurface exploration program was conducted in four phases based on the time and specific needs as the project progressed. Explorations, which consisted of borings and test pits, were placed in strategic locations within and beyond the limits of the currently active landslide and damaged roadway. The method and purpose of each phase is described below.

Phase 1 Borings

The Phase 1 borings consisted of ten (10), primarily hollow-stem auger borings, in and adjacent to the failed highway. The main purpose of the Phase 1 borings was to mobilize quickly to try and determine the depth of slope movement and general subsurface geology beneath the roadway; by general interval sampling and relatively quick installation of inclinometer casing. The borings were drilled with two CME-75 truck-mounted drill rigs.
The Phase 1 borings were drilled and sampled to depths ranging from 88 to 152 feet below existing grade using 8-inch-outside-diameter hollow stem augers. Materials encountered in the test borings were visually classified in the field and a log for each was recorded. Drilling and bulk sampling was also conducted by air rotary methods when and if drilling refusal was met using HSA sampling.

Each of the borings was outfitted with 3.34-inch O.D. inclinometer casing and backfilled to the surface with low strength cement grout. Each of the installations was outfitted with a flush-mount traffic rated monument box with cover.

**Phase 2 Borings**

The Phase 2 borings consisted of eight (8), mud-rotary rock core borings. The main purpose of the Phase 2 borings was to obtain a more complete sampling record of the subsurface geology so that the depth of landslide could be more accurately identified. The borings were drilled with two track-mounted componentized drill rigs. Each rig was disassembled into smaller (lighter) pieces for helicopter transport and the re-assembled in order to drill.

The Phase 2 borings were drilled and sampled to depths ranging from 148.7 to 250 feet below existing grade using a mud-rotary, HQ3 (2.406-inch core diameter) wireline coring system to obtain core samples. A casing advance system was utilized through overburden materials and through caving zones within the rock, as needed. Materials encountered in the test borings were visually classified in the field and a log was recorded. In addition, downhole optical televiewer data acquisition (COBL) was conducted in uncased portions of three borings. Because of caving problems in the upper portions of the holes and the need to case for stability, the optical logs were of limited extent.
The borings was outfitted with 2.75-inch O.D. inclinometer casing and Time Domain Reflectometry (TDR) cable secured to the outside of the SI casing, and then backfilled to the surface with grout.

**Phase 3 Borings**

The Phase 3 borings consisted of nine (9), mud-rotary rock core borings advanced near the base of the active landslide. The main purpose of the Phase 3 borings was to obtain geologic and geotechnical engineering data within in-place Chinle Formation bedrock materials for use in developing a conceptual buttress design for slope stabilization. The borings were drilled with a track-mounted drill rig with helicopter assistance for moving equipment and supplies. The Phase 3 borings were drilled and sampled to depths ranging from 29.1 to 39.3 feet below existing grade and into bedrock.

**Phase 4 Test Pits**

In order to obtain additional geologic information at the toe of the active landslide and within the potential buttress area, thirteen (13) test pits were excavated with a Caterpillar 329E excavator, equipped with a 24-inch-wide rock bucket. The test pits ranged in depth up to 24 feet below existing ground surface. The test pit walls were cleaned of smeared materials to expose the subsurface geologic structure and a log was created by our geologist.
Additional Design Exploration

Follow-up deep core borings (8) and geophysical explorations consisting of several thousands of feet of seismic refraction/REMI lines were completed above the existing highway road cut in order to obtain additional information regarding excavatability, blasting characteristics and subsurface geologic profile. This data was utilized to finalize construction design and construction procedures for creation of a new cut slope and generation of rock fill material.

INSTRUMENTATION AND MONITORING

Extensometers

At the initiation of our study, surface observations and rudimentary surface monitoring was setup to assess if additional movement of the landslide was occurring following the initial failure. Our initial efforts consisted of measurements across asphalt cracks in the pavement and establishing a series of align stakes between which measurements could be made on a daily basis as a form of an extensometer. The ADOT Materials Group installed two tripod-cable extensometers with a graduated scale that extended from the unfailed road prism (outside the apparent active slide), down onto the active landslide block. ADOT also installed two surface crack extensometers across asphalt road cracks in the highway crack in the highway. Other PK nail and pavement extensometers (measurements between two points across a specific crack) were also established and read in the early days soon after the initial slope failure.
Inclinometers

In addition to the surface monitoring, slope inclinometer casing was installed in 17 of the borings throughout the site. Upon completion of drilling and installation, each of the inclinometers was initialized for a baseline reading to compare future readings. Initially daily readings were recorded, until such time that it was determined that little to no movement was occurring and readings were then taken at weekly to monthly intervals up until the time of this report.

Time Domain Reflectometry (TDR)

As a backup measure for subsurface slope monitoring, TDR cable was attached to the inclinometer casing in seven of the borings prior to backfilling with grout. Monitoring with TDR allows detection of slip surfaces even after advanced movement has rendered inclinometer casing inoperable due to casing distortion. Though TDR does not provide the magnitude of movement it does allow for determination of the location of slip surfaces in the field during reading.

Summary of Monitoring

In general, little to no additional movement occurred throughout the majority of the landslide after the initial February 20 event. The majority of movement which impacted the road occurred in the first several hours after the initial early morning incident. Monitoring of instruments installed subsequent to the initial event, have been ongoing. Minor subsurface deflections in several of the inclinometers occurred within the first couple of weeks following installation, allowing precise demarcation of the depth of movement. Movements up to approximately 1/2 inch occurred within 6 months after the initial landslide event and only in
select inclinometer locations. Little to no movement was detected from any of the surface mounted extensometers measured over time.

GEOLOGIC MODEL/LANDSLIDE ASSESSMENT

Numerous continuous rock core borings and exploratory test pits as well geomorphic mapping were utilized to characterize both the recent and ancient slide mass. No groundwater or seeps were identified in any of the borings drilled or test pits excavated for this investigation or in the retrieved samples from the borings. In addition, no surface seeps were identified in the immediate site vicinity. Using the surface and subsurface data collected, a model of the geologic conditions for the current active landslide, as well as a conceptual geologic model for the formation of the ancient landslide through geologic time was developed. Based on our interpretation of the data, it is clear that the active landslide that disrupted the highway is a re-activated portion of the toe of a large, deep ancient translational landslide. Similar sized rotational landslides in the Grand Canyon region have been termed “megaslides”. Classifying this megaslide as translational is best supported by the similar structural orientation of the displaced bedrock units as compared to “in-place” strata orientations (generally northward strike with shallow to moderate dips to the northeast), the depth and geometry of the landslide determined from the borings and extrapolation of that failure surface to the currently exposed toe of the recent, active landslide (Figure 12).

![Figure 12 - View Looking North Showing Translational Nature of Landslide](image-url)
CONCEPTUAL GEOLOGIC MODEL THROUGH TIME

It is clear that the Echo Cliffs area has undergone extensive modification through geologic time. Works by others have postulated that massive landsliding, caused by various phenomenon such as rapid draining of past lava lakes or intense wet periods, occurred during the Pleistocene. There is no doubt that some of these phenomena have affected this site, as well as continued erosion through time. There is clear evidence for past massive landsliding all along the Echo Cliffs area and the site of this study. Additionally, there is ample geomorphic evidence for numerous bedrock faults (presumably not currently active) that have displaced and sheared large portions of the Echo Cliffs area, including this site. Past seismic activity may have also contributed to the formation of massive landslides.

Based on our review of geologic studies regarding formation of megaslides in the Colorado Plateau area, it is our understanding that most of these landslides developed in Pleistocene time when climatic conditions were much wetter. Our Conceptual Geologic Model through time is shown on Figures 13-18, and presents a series of scenario boards depicting geologic conditions from the Pleistocene to the present.

Initially the site was characterized by steep cliffs that extended farther to the west than present day (Figure 13). During some future time span in the Pleistocene, widespread debris slides/flows and ancient talus were deposited and blanketed much of the slopes extending from the top of the plateau to the mid-slope bench area and beyond, on top of the Chinle (Figure 14). Through time, erosion or mass wasting of the Chinle Formation, from the mid-slope bench area, removed lateral support from the cliffs resulting in the initiation of the ancient megaslide plane (Figure 15), predominantly failing through the weaker Chinle (Petrified Forest Member) clay-rich bedrock material. Bedrock fault(s) shear zones which exist in the upper portion of the cliff (and also identified during our field work) may have contributed structurally and seismically to the formation of the ancient headscarp and landslide formation.

Translational movement of the landslide block occurred and infilled the eroded void area of the Chinle materials in the mid-slope bench area and displaced Navajo and Kayenta/Moenave bedrock blocks in the upper slope (Figure 16). Continued translational movement through the Pleistocene epoch created tensional pull-apart grabens in the upper slope area and erosion of the Chinle-derived landslide materials occurred in the lower slope and mid-slope areas (Figure 17). Based on the relatively coherent nature of most of the megaslide block, it is our interpretation that movement of the megaslide through the Pleistocene most likely occurred episodically and/or as a result of long-term creep. Continued erosion of Chinle-derived landslide deposits during the Holocene epoch at the base of the slope and mid-slope bench areas resulted in partial reactivation of the megaslide toe, as the active slide that occurred in February 2013 (Figure 18).
Figure 13. Landslide Sequence Board 1 Showing Conceptual Geologic Cross Section in the Early-Mid Pleistocene Prior to Landslide Initiation.

Figure 14. Landslide Sequence Board 2 Showing Initial Widespread Debris Slide/ Rockfall Formation in the Pleistocene.

Figure 15. Landslide Sequence Board 3 Showing Initial Development of the Ancient Megaslide Failure Surface by Erosion at the Toe in Mid Pleistocene.

Figure 16. Landslide Sequence Board 4 Showing Translational Movement of the Ancient Megaslide in Mid Pleistocene.

Figure 17. Landslide Sequence Board 5 Showing Continued Translational Movement of the Landslide Block and Continued Erosion of the Chinle Fm. Materials at the Toe in the Late Pleistocene.

Figure 18. Landslide Sequence Board 6 Showing Continued Erosion and Development of the Active Landslide by Long-Term Creep in Holocene to Recent Time.
GEOLOGIC CAUSATION-RECENT ACTIVE LANDSLIDE

The recent landslide failed along the pre-existing ancient megaslide slip surface, which in this area is a shallow westward-dipping sheared clay zone overlying “in-place”/intact plastic claystone of the Chinle Formation. Since groundwater or seeps were not encountered in the subsurface explorations, this feature can be considered a “dry” landslide; one not triggered by subsurface pore water pressure or as a result of saturation and infiltration from intense rainstorm activity.

We conclude that the recent landslide failed as a result of long-term erosion (by wind, rain and snow) at the base of the slope and removal of lateral support and long-term, continuous creep of the landslide mass along the shallow-dipping, sheared clay slip surface. Long-term creep has reduced the slip surface to low residual strengths allowing failure when the critical retaining mass/lateral support has been removed. More recent evidence that supports creep deformation over time in this active landslide is demonstrated by a previous inclinometer that was installed in 1995 and was monitored until 2011. Cumulative plots through time show continual and gradual downslope lean (creep deformation), down to the base of the inclinometer. The recent failure also occurred along the same headscarp that displaced the highway a couple of inches in 1988.

Figure 19 - View of In-Place, Multi-Colored Claystone of the Petrified Forest Member of the Chinle Formation Just Beyond (west) of the Landslide Toe.

GEOTECHNICAL MITIGATION DESIGN

Stability Analyses

Both static and seismic stability analyses of several cross-sections developed through the ancient and active landslide masses were performed using SLIDE V6®. The analyses were completed to model the increased resistance to movement afforded by an earthfill rock buttress placed at the base of the active slide mass, as well as determine the impact of excavating into the existing cut slope and shifting the road to the east. A target factor of safety (FOS) of 1.3 (resistance versus driving force for the active landslide mass) was requested by ADOT in terms of sizing the rockfill buttress. Stability of the active landslide mass was modeled by targeting a
back calculated FOS of 1.0 (the point at which movement occurs) and residual strength (6 to 10 degrees determined by torsional ring shear) of the Chinle clay at the base of the slide. The results indicated that with sufficient buttress mass (approximately 750,000 cubic yards over a distance of 1,400 feet), combined with removing an equal portion of the road cut slope driving mass, the target values were achievable through all sections analyzed.

**Landslide Buttress and Upslope Lane Adjustment**

We consider the most geotechnically feasible alternative to be construction of a buttress at the base of the slope and an upslope adjustment of the travel lanes of the highway (off of the active slide limits) by widening the back cut slope. This is based on ADOT’s preference to reopen the highway in or adjacent to its existing easement, a more time sensitive/quicker fix and presumably less costly than the alternatives. As presented in our slope stability analysis, we conclude that since we currently do not see any evidence for mass movement within the ancient landslide block above the road, and the materials within the upper portion of the ancient landslide have higher strengths along the ancient slip surface, then stabilizing the active landslide will result in the overall slope being restored to an acceptable factor of safety against failure. It should be noted that conceptually this buttress is designed only to address the current activity and would not be considered sufficient to stabilize the entire ancient landslide mass, should other factors come into play to destabilize the entire ancient mass. Based on the data we have collected and analyzed to date, it is our opinion that the potential for global destabilization to occur above, after construction of the buttress is considered to be low.

In order to construct the buttress efficiently, a local source of earth material is desired from a cost and aesthetics perspective. The most logical supply would come from within the site itself and the most preferred material from a geotechnical viewpoint would be the Kayenta/Moenave-derived sandstone material from the slope above the existing road. Additional cuts in the back slope would provide a three-fold benefit to the project, 1) it is a locally-derived, geotechnically suitable source material with which to construct a buttress, 2) it allows for an upslope adjustment of the travel lanes farther away from that portion of the roadway prism (and edge) that experienced the majority of the damage, and 3) it addresses an existing rockfall issue with the current (steeper) cut slope. The current back cut slope concept presented by ADOT moves the cut approximately 60 feet (locally) eastward, with a single bench, 1.25H:1V cut upward from the roadway.

**CONCEPTUAL BUTTRESS DESIGN**

To restrain further movement of the active landslide mass as a whole, a relatively large buttress is required at the base of the slope. Based on the results of the laboratory testing, slope stability analyses and an ADOT request for a minimum Factor of Safety of 1.3, the buttress will have a footprint of up to 350-feet-wide, have a height of 70 to 80 feet, and have an upper bench width of 150 to 225 feet. The buttress will be founded a minimum of 10 feet into competent bedrock and have a minimum keyway width of 50 feet. The buttress will require subdrains within the keyway and at selected elevations within cut benches. Subdrains will flow by gravity to selected discharge points west (downhill) of the buttress.
Buttress construction will consist of engineered fill compacted to a minimum relative density of 90% (ASTM-D 1557). Where testing for compaction is not practical due to coarseness of the material, a roller compaction specification will be developed. The buttress will be constructed mainly of Kayenta/Moenave-derived sandstone engineered fill material that will be generated from the proposed back cut above the highway, which has relatively high remolded strengths. Construction will include excavation and ultimately placement of Chinle-derived claystone fill, which is plastic and has weaker strengths than the Kayenta/Moenave sandstone derived fill. The claystone fill will be wasted into non-critical portions of the buttress such that stability will not be affected. In addition to (and subsequent to) buttress construction, large cracks throughout the body of the landslide will be track-walked and in-filled (preferable with clayey material) as much as practical to reduce the potential for rain and surface water to infiltrate into the landslide mass.

As part of the overall mitigation project, re-grading of the existing road section and prism will be required to create a new graded slope. Because this area is underlain by several ground cracks that will not be mitigated for their full depth and the fill may still be subject to some settlement and creep, approximately 10 feet of the road fill prism will be reconstructed as a geogrid reinforced compacted engineered fill of low plasticity. This will seal cracks beneath the road shoulder and provide a unitized fill platform that will reduce the potential for large settlement cracks or offsets adjacent to the pavement. Even with this reinforcement, periodic maintenance of this road shoulder will be anticipated. The new road surface and section will be shifted to the east completely away from the currently disturbed ground. This will serve to reduce (minor effect) the driving force on the currently active landslide.

**Conceptual Buttress/Mitigation Plan**

In order to stabilize the 1,200-foot-wide active landslide, a buttress approximately 1,400 feet in length (measured roughly in a north-south direction along the base of the slope) will be required. The ends of the buttress will extend into stable ridges. The approximate footprint of the buttress, proposed 1.5H:1V back-cut and possible geogrid roadway reinforcement/regarded slope are presented on Figure 20 and 21.
Figure 20 - Conceptual Buttress and Cut Slope Design Detail from Geotechnical Design Report.

Figure 21 - Conceptual Buttress/Back Cut Plan from Geotechnical Design Report.
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Rockfall source detection and volume measurement from autonomous UAV-acquired photogrammetry:

A case study from a transportation corridor in northwestern Ontario, Canada

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ABSTRACT

Land-based and traditional aerial photogrammetry techniques have been applied widely and successfully to develop precise three-dimensional (3D) terrain models for many applications, including geological or structural analyses of simple rock cuts. However, whereas the geometry and scale of many hazardous rock slopes adjacent to highways or railways are not really conducive to either terrestrial or downward-looking aerial cameras and laser scanners, the newest generation of autonomous, unmanned aerial vehicles (UAV) equipped with high-resolution side-looking digital cameras present a solution to this vantage-point issue. In this study we generated detailed 3D photogrammetric models of a slope along a railway adjacent to Lake Superior near Marathon, Ontario, Canada, and compared them quantitatively for changes. This slope has a similar configuration to local highway slopes. The models were created using hundreds of digital photographs collected by an autonomous UAV in August 2012 and October 2013, between which a number of rockfall events had occurred and affected railway operations. We calibrated the change-detection routine using a large rockfall event that occurred at another site in western Canada, for which we also had detailed LiDAR as ground-truth. At the Lake Superior site we were able to identify both the source location and volume (approximately 9m$^3$) of a single rockfall event, using only the UAV-acquired photogrammetric data. In the paper we discuss the data collection and model development, the change-detection and volume calculation methodology for the single 9m$^3$ rockfall, and we explore the best practices and major limitations in the analysis and applications of these methods to highway problems.
INTRODUCTION

Terrestrial remote-sensing methods, such as LiDAR, have been used extensively to develop digital terrain models and characterize geomorphological or geomechanical features (e.g. Olariu et al., 2008; Lato et al., 2009; Sturznegger and Stead, 2009; Brodu and Lague, 2012; Gigli et al., 2014). Multiple LiDAR data may be compared, and changes detected (e.g. Rosser et al., 2007; Oppikofer et al., 2008, 2009; Abellan et al., 2009, 2010, 2011; Stock et al., 2011, 2012; Royan et al., 2013). (See Andrew et al. (2013) and Abellan et al. (2014) for excellent reviews of the applications of LiDAR techniques to rock slope problems).

More recently, ‘multi-view stereo’ (MVS) or ‘structure-from-motion’ (SFM) photogrammetric modeling has advanced to the point where in some cases the 3D terrain models generated in this way can match or exceed the fidelity and utility of terrestrial LiDAR (e.g. Westoby et al., 2012; James and Robson, 2012; Fonstad et al., 2013; Hugenholtz et al., 2013). Consumer-grade camera equipment and modern software has allowed for the development of rich 3D terrain models from a set of overlapping oblique photographs taken from ground level, or with traditional vertical aerial photography, for rock slope characterization (e.g. Haneberg, 2008; Sturznegger and Stead, 2009; Wolter et al., 2014), landslides (e.g. Gonzalez-Diaz et al., 2013), glacial mapping (Whitehead et al., 2013), and other applications in the geosciences (e.g. Javernick et al., 2014).

The geometry and scale of many hazardous rock slopes adjacent to highways or railways are often not conducive to either terrestrial or downward-looking aerial cameras and laser scanners (i.e. the traditional approach to LiDAR). For example, terrestrial LIDAR or photographic data for transportation corridors may be difficult or impossible to collect in narrow through-cuts, or for any location with both tall slopes and some restriction in possible scan sites with a useful vantage-point (e.g. along steep valleys, or lakeshores and river valleys). Furthermore, the downward-looking aerial techniques, like traditional airborne LiDAR, are typically unable to resolve very steep or vertical slopes. In these cases, the newest generation of autonomous, unmanned aerial vehicles (UAV) equipped with high-resolution side-looking digital cameras present a relatively inexpensive solution to data collection. When paired with modern, MVS-capable photogrammetric modeling software, the UAV-acquired imagery can be used to generate detailed 3D models of otherwise inaccessible slopes, with sufficient resolution to detect change between models collected at different times, although up to now this has been mostly qualitative (e.g. Hugenholtz et al., 2013) or DEM-based (i.e. not truly 3D; e.g. Gonzalez-Diez et al., 2012).

In this study we generated detailed 3D photogrammetric models of a slope along a railway adjacent to Lake Superior near Marathon, Ontario, Canada, and compared them quantitatively for changes. This slope has a similar configuration to local highway slopes. The models were created using hundreds of digital photographs collected by an autonomous UAV in August 2012 and October 2013, between which a number of rockfall events had occurred and affected railway operations. We calibrated a quantitative change-detection routine using a large rockfall event that occurred at another site in western Canada, for which we also had detailed LiDAR as ground-truth.
The purpose of this study was to investigate a potential new input to rockfall hazard rating or monitoring systems, which rely heavily on slope geometry and kinematics of failures (see Pierson, 2012). The use of quantitative change-detection from 3D photogrammetric models of rock slopes may offer the potential to add a quantitative measure of slope activity into existing or new hazard rating schemes.

Study Area

We conducted this study at a highly active rockfall hazard site, along the Canadian Pacific Railway on the northeast shore of Lake Superior, in Ontario, Canada (Heron Bay Subdivision Mile 71.6, Northern Ontario Service Area). The configuration of the slope and the hazards it presents are similar to other local slopes along the TransCanada Highway #17 between Wawa and Nipigon, ON, particularly where the highway is located on a bench cut into the near-shore slope, as the railway is at this location. Figure 1 shows the location of the site and nearby highways and towns, and the overall configuration of the site.

The rock hazard at our study site is due mostly to discrete rockfall, although larger and more complex failures are certainly possible in this area. The exceptional relief and general ruggedness of the terrain posed a significant engineering problem during the construction of the railway in the 1880’s, as did the rockmass conditions they encountered during both benching and tunneling. The rockfall hazard has existed continuously since the railway was constructed.

Figure 1 – Location map (inset) and oblique photo of the study site. Scale varies (tunnel portal is ~8 m tall).
The site is located within the Coldwell Intrusive Complex, an 1100 Ma old intrusion of gabbro and syenite associated with mid-continental rifting (Heaman and Machado, 1992; Mitchell et al., 1993). Detailed mapping found mostly nepheline syenite and lesser gabbroic rock on the slopes above the railway (Walker et al., 1993). The rockmass is characterized locally and within the site by at least two sets of orthogonal, persistent, near vertical joints, and one set of subhorizontal sheet joints. The terrain is surely bedrock-controlled, and bedrock exposures are extensive, both on the vertical faces and on flatter, higher terrain (Figure 1). The jointing makes for a blocky, failure-prone rockmass, although the intact strength of the rock material likely very high. The blockiness interacts with the very high and rugged relief to leave innumerable discrete blocks susceptible to dislodgement by an appropriate trigger, e.g. freeze-thaw action or root wedging at this site, similar to other large, sheeted rock slopes (e.g. Stock et al., 2012). From crest-to-rail the slopes may reach 100 m, but the most active rockfall source zones seem to be in the middle-heights of the slope. The slope is generally east-facing here (shoreline is north-south), although aspects from south through east to north are represented in the local area.

The railway is protected my numerous engineered measures in this area: rockfall and slide detector wires, rockfall catchment fences, ditches, concrete walls, etc. Furthermore, an annual maintenance program is executed here, which includes rock scaling, brush removal, and installation of rock stabilization measures (e.g. bolts and buttresses). There is a short tunnel in this section, although it is original to the railway and rockfall hazard likely did not factor into the decision to construct it.

METHODS AND DATA

We collected overlapping oblique, digital photographs of the site using a side looking camera (Nikon J1) attached to an autonomous UAV, in August 2012 and October 2013. The complete flight line was about 2 km long, and we made two passes at different heights during each flight (Figure 2). Individual photos covered an area on the slope approximately 100-300 m across (depending on distance between the UAV and the slope), and we achieved up to 90% overlap between adjacent photos. The complete photo-sets for each flight contained hundreds of images. Most were retained for 3D modeling, although for this study we focus on a subarea of the full survey. The view angle of the camera was set at 45 degrees down from the wing plane, in order to optimize the vantage point for the slope configuration.

The UAV itself was a pre-commercial test unit in both cases, and the flights were conducted by Precisionhawk Inc. It was a lightweight, battery-powered fixed wing aircraft, which was totally autonomous in flight, both for flight systems and data collection. It was launched manually from ground level by throwing it into the air, and landed on Lake Superior using foam floats attached to the fuselage. Our experience suggests that this technology requires a fairly skilled operator, a full-time technician, and a certain amount of patience and resiliency to operate in field conditions, particularly those encountered along the north shore of Lake Superior (e.g. high winds and waves). Each field campaign took most of a day to complete, although the actual flights were never longer than 10 minutes.

We processed all of the images and generated 3D models using the commercial software ‘Photoscan Pro’ version 1.0.4 by Agisoft (www.agisoft.ru). The software uses the SFM
algorithms to resolve the relative locations and camera orientations of each image using automatically detected matching points across multiple images taken from different locations. A triangulation routine (MVS) then generates a depth map for each image, which contain the 3D coordinate information for individual pixels. From these, dense point clouds (i.e. synthetic LiDAR) or 3D polygonal meshes can be generated. Haneberg (2008), Sturznegger and Stead (2009a), Lato et al., (2012), and Andrew et al. (2012) describe terrestrial photogrammetry data collection, processing, and applications to rock slopes, in detail.

In our study, we controlled error by first omitting from the image alignment stage any points having a reprojection error of greater than 0.5 pixels. This means that matching alignment points used in the generating the 3D model are in the same 3D location (plus or minus 0.5 pixels) in each photo. The dense point clouds we generated for this study had greater than 10 million points. The limitation here is mostly in the processing hardware; the theoretical limit to point density (without reducing precision) is so large as to be unachievable without huge computer resources. Since the photogrammetry models are not natively registered in real world scale or coordinates, we collected both on-board UAV GPS coordinates for each photo location, and a selection of about 30 sub-meter GPS points on the ground. These ‘ground control’ points were applied to the model, and it was translated to proper orientation, scale, and coordinates. The scaling function is a simple linear transform, and could be completed using only a single known scale item in the model, e.g. the rail gauge, if geo-referencing was not required.

Figure 2 – 3D rendering of the 2012 photogrammetry model, showing UAV camera locations for each photo used. Scale varies (tunnel portal is ~8 m tall).
We used the free software ‘Cloud Compare’ version 2.5.1 (http://www.danielgm.net/cc) to make the quantitative comparisons between the models collected at different times. This is accomplished by first performing a best-fit alignment between the two models, and essentially mapping the residuals of the best-fit as shortest distances from each vertex in one model to the surface of the other. However, despite the application of ground control during model generation, we found that a systematic mismatch in the scale of the two models was common. To overcome this we allowed the scale of one model to be adjusted during the best-fit alignment. In each case the best-fit was achieved with about a 2% adjustment in the scale in one model (i.e. a 1 m feature in one model was 0.98 m long in the other). This is probably the best measure of precision we can quote.

In this approach, the mapped residuals represent a combination of general mismatch between the models, error in the model(s) resulting in a local mismatch, and actual change that occurred on the slope. The general mismatch is widespread and small in magnitude, whereas the other two may be indistinguishable, but generally spatially constrained and larger in magnitude. We simply omit the general mismatch from further analysis by ignoring the small, widespread residuals, which are typically less than 0.25 m magnitude in this study. This value represents a threshold, below which we cannot confidently attribute a residual difference between the two models to an actual change on the slope.

We calibrated the change-detection routine using a large rockfall event that occurred at another site in western Canada, for which we also had detailed LiDAR as ground-truth. The mapped change using photogrammetry models was indistinguishable from that in the LiDAR models. We also used this case to test a change volume estimation routine. To estimate the volume of discrete areas of change between the photogrammetry models, we applied a simple numerical approach, which relies on the high spatial density of the points/vertices in the model:

\[ \text{Estimated Volume} = \frac{\text{Sum of shortest distances}}{\text{Total area} / \text{Number of vertices}} \]

This method essentially treats each distance measure at each vertex as a miniscule, equal-area column in the space for which the volume is calculated. Then, by summing the volume of each it is possible to estimate the total volume of the space. For the calibration rockfall, we calculated a volume of 2505 m$^3$, compared to just over 2600 m$^3$ using the LiDAR. The difference is well within the confidence limits for the LiDAR measurement.

For the Lake Superior site we also had a series of known rockfall events against which to compare our results. There were at least three events with deposits reaching the railway in the period between our field campaigns, with block-size at track level measuring approximately 1 – 3 m$^3$. A suspected source location for one of these events is visible in the UAV photos, and was well captured in the modeling.

RESULTS

Figure 3 is an oblique view of the 2013 3D model for the entire area west of the tunnel portal, with the residuals mapped over the 2013 data. Since we are searching for rockfall (i.e. loss of material between 2012 and 2013) in the source areas, we only included negative change
in the figure. Overall, the change-detection comparison found differences between the modeled surfaces ranging from approximately 15 m of positive change, to 15 m of negative change. However, if we ignore all of the change detected near the boundaries of the models and in any heavily vegetated areas, this range drops to between 1 m of positive and approximately 2 m of negative change. In any case, the detected change values for each vertex in the analysis shows that greater than 99% of the residuals were between 1 m and -1 m. In the non-vegetated areas of the model, we found that negative differences between the models below about 0.25 m were widespread across the surface (this is the assumed error), while greater changes were discrete spatially, in general. The discretion is not perfect though: in Figure 3 there remain areas of widespread but not large negative change near the threshold value, e.g. along the concrete block-wall and tunnel portal, and along rock features nearby, as well as near the crest of the slope above the portal in the figure. However, in the area of the presumed source of the 2013-2013 rockfalls there are a number of discrete areas of larger negative change, which warranted further investigation.

Figure 3 – Negative change-detection map, comparing 2012 to 2013 3D models.
By manually comparing the 2012 and 2013 models in these areas in detail, we found that the majority of the suspected change areas were located on small, vegetated benches exploiting the sub horizontal sheet jointing. We were able to observe that in 2012 we had captured – and modeled – the leaf canopy of the bushes on these benches, whereas in 2013 the benches were mostly bare, or at least leaf-free. However, in one of these locations the comparison of 2012 to 2013 models clearly showed a grey-colored convex rock protrusion, while in 2013 that location was a tan-colored convex surface.

We investigated this ‘missing’ area of the rock face further by resampling the models at a much higher fidelity over just the area of the potential rockfall source, and redoing the alignment and change-detection analysis (Figure 4). When doing this it was critical to select an area large enough to contain many matching surfaces, as well as the area of change; otherwise, the alignment would fail, since the two surfaces would be mostly unique. Figure 4 shows the modeled area for 2012 and 2013, and the mapped change. The missing block is adjacent to a pre-existing oxidized face, and wraps over a flat and near vertical joint surface. In the 2012 model one can observe that the perimeter of the missing block was cracked.

We reviewed the rockfall records submitted by railway staff, and found at least two events suspected to have originated in this source area. In fact, one of the reports contained photos of this source location taken from track-level, which, despite the awkward perspective, show the tan-colored surface clearly. These rockfall events resulted in several blocks in the range of 1 – 3 m³ reaching the rail right of way.

We applied the volume estimation method to the missing block (boundary selected manually, based on the change values and inspection of the models), using 18196 change values spread over 15.28 m². The volume of the missing block was approximately 8.92 m³ using this method. This is certainly within the range of the accumulation of volume at track-level, and suggests that a larger block released en masse, and broke-up during its descent.

DISCUSSION

Our comparison of the model results from 2012 and 2013 showed many areas of large magnitude differences, up to 15 m positive and negative in places. Further inspection revealed that these were related to the location of larger trees, poorly modeled boundaries, other artefacts or differences in the models, rather than the actual change in the rock slope which would be of interest for rockfall hazard assessment. These large changes are ignorable for this analysis, and in the future it would probably be best to filter or mask out areas of poor model results, e.g large trees, etc., before even making the model. The advantage of this would be to avoid detecting irrelevant change, but also to avoid inefficient use of hardware resources during the 3D model generation. Furthermore, these poorly matching areas reduce the best possible fit of the alignment, and thereby reduce the accuracy of the change-detection routine.

Smaller magnitude, more widespread differences we detected between the two models were related to either general minor misalignment, or basic variability in the models. We could improve the general alignment by masking (as above), or reduce variability in the models by reducing variability and error in the raw data, i.e. the raw photographs. In order to characterize...
Figure 4 – Negative change-detection map for rockfall source area (a), and comparison 3D model renderings for 2012 (b) and 2013 (c).
the general error more specifically we could leverage the tie points between the models, that we
presume aren’t changing e.g. the tunnel portal. This is really a measure of precision, rather than
accuracy, since we have no real low-error ground truth against which we could compare the
model. Even the GPS coordinates we used are subject to errors of 1 m or more, both in measured
position and in the plotting of the points on the model. In any case, the goal here would be to
reduce the change-detection threshold from 0.25 m down to 0.1 m or less, which would be about
the same as a terrestrial LiDAR and probably better than many aerial LiDAR surveys, and to
increase our confidence that the modeled surface accurately represents the actual surface.

Once we ignored the largest and smallest magnitude changes, we found that there were
many discrete areas on the slope, a few to 10 meters across, that showed some amount of ‘real’
change between August 2012 and October 2013. As one might expect, many of these areas of
change were real, but not directly relevant to the rock hazard. They represent bushes and other
low vegetation, which had leaves in August, but not in October. The general colour tone of the
3D models confirms that the October data were collected well into the fall season. These areas
could have been masked out in the August data, but not as easily in the October data, in which
case the modeled surfaces would have been a better match. Or, another approach is available:
newer software applications permit the classification of the 3D data into ‘ground’ and ‘non-
ground’ points, based on the roughness or noisiness of the surface, which has the potential to
omit vegetation and leave a smooth (assumed) ground surface, which would likely match better.
In any case, the best approach would be to always collect data in the spring and fall when the
trees and shrubs are foliage-free, or to focus the analysis on areas of bare rock only.

In terms of ‘real’ change relevant to the rock hazard at the site, we found that a 9 m$^3$
missing block was detectable in our data, and that this location corresponded to the suspected
source of a rockfall event(s) in the spring and summer of 2013. The volume estimation method
we applied proved to be a useful numerical approach, provided that a very high resolution 3D
was available. Further investigation showed that the block was cracked around its perimeter prior
to failure, and that both the pre-and post-failure geometry (and kinematics) could have been
measured easily. We could surely detect smaller missing blocks in the future with better
photography and modeling. Confirming the shape of the block, and its in situ volume was not
possible previously with traditional approaches, although often a fresh scar would have been
detectable by eye from track-level. But, this is probably the exception, since most of the source
locations here and elsewhere are not visible from track level, and no one would ever look if the
rockfall had run out past the rails or stopped short of them.

We probably couldn’t have detected minor displacements or deformation on the slope
preceding a rockfall, but we may be able to use this technique to identify spatial clusters of
smaller rockfalls, which could be precursors to larger ones. Or, simply provide targets for rock
scaling and stabilization efforts based on activity on the slope. And, knowing the exact source
location, shape, and volume can be leveraged for investigating the conditioning and triggering of
the rockfalls, and modeling their trajectory, fall path, and runout energy for design of
remediation, in true 3D (Ondercin et al., 2014).
We didn’t look for deposition volumes, since the camera look angle was optimized for observing the steep part of the slope. However, this could be handled in the same manner, and may be as important as source observations, both for maintenance and for protection design.

Accurate measurements required effort in setting the scale of model, and orienting it in the real world, although the latter was only for convenience and conversion to other mapping and modeling programs. Photogrammetric models are never natively scaled, and therefore some number of items of known size must be in every model in order to make accurate measurements. This is probably simple enough in a highway alignment, where monuments, lane widths, pole spacing etc. are well known, or scaling targets are easily added to the scene in the field. Otherwise, this method seems to be a useful companion or low cost replacement for LiDAR, and other terrestrial remote sensing methods – with the added advantage of a full-color representation of the surface. Note that similar results to this study could have been obtained using carefully-collected photos taken manually from a helicopter, or at other locations from the ground, where the terrain and vantage points allow it.

Where current rockfall hazard rating or monitoring systems rely heavily on slope geometry and kinematics of failures (see Pierson, 2012), the potential to add a low cost and efficient quantitative measure of slope activity, or to document the actual movement of rock material, presents an opportunity that did not previously exist. In some cases the current approach to activity monitoring is to make subjective comparisons of serial photographs, to ‘look’ for changes; in this paper we’ve described a way to do this quantitatively and in 3D using only site photos, which frees the scientist or engineer to spend their time studying more complex aspects of the problem, or to apply more of their experience and judgment to the subjective aspects of rock hazard management.

CONCLUSIONS

In this study, we found that:

- UAV photogrammetry oblique, of steep slopes is a useful alternative to aerial LiDAR or terrestrial methods where vantage point issues exist
- Quantitative change-detection between photogrammetric slope models is possible, and with some expert judgment it is also possible to discriminate between error, non-relevant ‘real’ change, and the ‘real’ changes on the slope due to rockfall
- Volumes of manually selected change areas can be estimated using a simple calculation, which relies on the very high spatial resolution of the model for its specificity. Confidence in these estimations is similar to that of LiDAR volume calculations.
- Careful fieldwork and processing are required to achieve useful thresholds for detection of change, and the error sources are well-known and manageable. We ignored model differences less than about 0.25 m. This could be reduced down to 0.1 m with some effort, but may not be improvable for large slopes, or for oblique photos taken in difficult field conditions. The theoretical limit of detection is much lower, but will almost never be achieved.
Expert judgment is always required when applying these techniques, at all stages: data collection, processing, analysis, and interpretation. Both technological and geological interpretation is critical.

Future research work in this area will look at deposition areas, and larger areas with few rockfall records. We suggest field work during spring or fall, when there are no leaves, although the timing of serial scanning should be driven by the character of the hazard, rather than by convenience.

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Geotechnical Challenges in Laboratory Investigation of Bridge Abutment Scour

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ABSTRACT

Bridge abutment failure attributable to scour at bridge waterways involves combined geotechnical and hydraulic processes, interacting to erode the compacted earth fill typically forming abutments and their approach embankments. However, the leading guides for bridge design and monitoring inadequately take account of geotechnical factors influencing scour-depth estimation. Our paper presents the findings of a series of laboratory flume experiments conducted to investigate the effect of soil strength on scour depth at spill-through abutments. We particularly focus on the challenges of conducting laboratory experiments on scour of erodible model abutments at a model scale of 1:30. These challenges, or difficulties, include attaining scale-reduced shear strengths, controlling and verifying the soil compaction, and quantifying the in-situ shear strength of both cohesionless and cohesive model soils. We addressed these difficulties through a sequence of soil tests designed to relate model soil strength to laboratory compaction, using hand-held devices to check soil strengths in flume experiments. Two correlation charts were developed to relate the relative density of a cohesionless soil embankment model to 1) a penetration resistance measured using a needle penetrometer, and 2) shear strength determined from direct shear tests. Similarly, two correlation charts were developed for cohesive soil to relate percent of compaction to 1) undrained shear strength determined from both pocket penetrometer and torsional vane shear tester, and 2) shear strength determined from direct shear tests. Our paper explains the successes and unresolved issues relating to the shear strength of model soils used in flume studies of abutment scour.
INTRODUCTION

Scour, particularly abutment scour, is the most common cause of bridge failure at waterway crossings in the United States (1). Although bridge abutment scour has been investigated by many researchers, the relationship between scour and the geotechnical properties of a spill-through bridge abutment (the most common abutment form) is only just beginning to be investigated. We present field case evidence and laboratory experiments showing that bridge scour involves the interaction of both hydraulic and geotechnical processes. In particular, we describe the challenges, or difficulties, faced during laboratory investigation of this interaction.

The leading bridge design and monitoring guides, notably the Federal highway Administration (FHWA) design guide HEC-18 (2), indicate the importance of geotechnical processes regarding scour, but do not take them into account. This status reflects the recent nature of scour-related research involving the combined effects of geotechnical and hydraulic processes. Prior to our study, the most developed effort to consider geotechnical effects is the conceptual method proposed by Ettema et al. (3) to estimate the maximum scour depth limited by the geotechnical stability of channel bank and earth fill approach embankment. However, this formulation lacked verification by means of laboratory experiments. Such experiments have not yet been conducted to verify this formulation in part because of the complexity of laboratory simulation of combined geotechnical and hydraulic processes. In particular, other than the experiments completed for this study (4, 5), no prior work has considered laboratory scaling of soil strength.

Including geotechnical factors in scour-depth calculations requires that flume experiments be conducted with controlled soil compaction and soil shear strength measurements so as to determine how embankment soil strength influences scour depth. Our paper reports a preliminary effort to conduct such a series of laboratory flume experiments using erodible 1:30-scale model abutments. Several experimental difficulties were encountered, including how to select model soils with scale-reduced shear strengths, and how to control and verify the in-situ shear strength properties of the model soils. Methods were developed for addressing these difficulties. These methods are explained, as are unresolved issues relating to the shear strength of model soils used in flume studies of abutment scour. The research concludes that laboratory observations are related to field observations of abutment scour.

FIELD CASE EVIDENCE

Field case evidence provides valuable illustrations on how bridge abutments and embankments actually fail as a result of scour. This field evidence essentially provides the basis for developing scale-down laboratory experiments described in this paper. To ensure realistic laboratory experiments were performed, actual failures observed in the field and the simulated failure processes obtained from the laboratory were compared. The following field cases reported in Wyoming suggest two important observations (6):

(1) Bridge abutment and embankment failures involve the interaction of both hydraulic and geotechnical processes.
(2) Two main failure modes occur: a) erosion at the bridge abutment toe; and b) erosion at the flow waterline. This latter failure mode has been largely overlooked even though it seems to commonly occur.

The writers present two sets of photographs illustrating field cases of abutment failure in Wyoming. Figure 1 shows the eroded spill-slope of a spill-through abutment at Cottonwood Creek, WY. The slope likely began eroding at the flow waterline, continued eroding back to the abutment column, and left a vertical back face and a modest depth of abutment scour. The riprap rock protected the abutment slope from collapsing. Little evidence of scour was observed at the abutment toe.

Figure 1 – Views of abutment erosion at a bridge on Cottonwood Creek, WY

Figure 2 shows different failures at spill-through abutments reported in Wyoming and elsewhere. As evidenced by the removal of spill-slope soil, Figure 2 (a) shows that the bridge abutment along Platte Creek, NY failed by erosion at the flow waterline with apparent erosion at the toe. Figure 2 (b) shows the embankment failure along the Murphy Creek, WY. The embankment fill was formed from compacted silty sand. The stream bed consisted of gravel and 4 in (100 mm) diameter rock. Blue shale bedrock was located 10 ft (3.05 m) to 15 ft (4.57 m) below the streambed. Similar to Plate Creek (Figure 2 (a)), this failure was caused by erosion at the flow waterline. Figure 2 (c) shows erosion at the flow waterline at the bridge abutment along the Richeau Creek, WY. Scour depth was limited by the shallow bedrock. Long contraction and choked flow conditions were observed. Figure 2 (d) shows the same erosion condition at the bridge abutment along the North Platte River, WY.

Figures 1 and 2 show that abutment erosion occurred primarily at the flow waterline around each abutment and involved the interaction of both hydraulic and geotechnical processes. Swift flow combined with wave action loosen protective riprap rock, entrain embankment soil, and further expose embankment soil to erosion. Although the failure mode is evident in all the field cases shown, laboratory experiments are needed to fully reveal the progressive failure of embankment soil beginning at the abutment’s upstream corner. The literature on abutment scour (e.g., 7) usefully summarizes the findings of earlier research, but inadequately addresses the progressive erosion associated with the abutment failures shown in Figures 1 and 2.
Figure 2 – Images of different spill-through abutments exhibiting failure. From upper left, clockwise; (a) Platte Creek, NY (b) Murphy Creek, WY (c) Richeau Creek, WY (d) North Platte River, WY

LABORATORY FLUME EXPERIMENTS

Overview

We conducted laboratory flume experiments at the University of Wyoming using a 1:30 geometric-scale, hydraulic model of a spill-through bridge abutment as shown in Figure 3 (4). All experiments were run in a 60 ft (18.3 m) long by 8 ft (2.44 m) wide channel, which was raised 1 ft (0.3 m) higher than the flume base. The flume comprised a headbox (inlet), upstream and downstream channel beds between a sediment recess, and a tailbox (outlet) along its length. Recirculation of water for continuous flow through the flume was controlled by a variable frequency-drive pump. To ensure sediment transport through the test section, the pump’s frequency of 34.1 Hz produced a discharge of 3.7 ft³/s (0.105 m³/s) at a velocity of 3 ft/s (0.92 m/s) for all experiments.

The 1:30-scale model spill-through abutment was constructed on a 1 ft (0.3 m) thick sediment recess filled with compacted uniform medium sand as shown in Figure 4. Penetration resistance tests using a needle penetrometer were performed along the transverse line of abutment to ensure consistent compaction. At least three measurements were recorded in an interval of 1 ft (30 cm) from abutment spill-slope toe to the plywood wall as shown in Figure 5. The 7.9 in (0.2 m) thick abutment model consisted of 1) a 1:1.5 erodible spill-through abutment spill-slope formed using soil materials with different shear strengths; and 2) a 1.3 ft (0.4 m) wide and 2.1 ft (0.64 m) long straight embankment extension. The abutment’s top width of 1.3 ft (0.40
m) was selected in line with a 1:30 scale of a standard two-lane road having two, 13.1 ft (4 m) wide lanes and 7.9 ft (2.4 m) wide shoulders.

Figure 3 – A typical pre-run condition of the large recirculating flume and an abutment model setup

Figure 4 – A 1:30-scale model spill-through abutment located on the sediment recess in a pre-run condition
Uniform Sand Abutment

Poorly graded (uniform) sand (SP) classified in accordance with the Unified Soil Classification System (USCS), ASTM D2487 (8), was used for constructing the abutment spill-slope. Water was added to sand to produce an approximate 10% moisture content necessary to facilitate compaction. A mark on the sand bed indicating area to be covered by the spill-slope was made. The model sand was spread and compacted in successive layers using an 8 in (203mm) by 8 in (203mm), 10 lb (4.65 kg) flat tamper. When the layer thickness reached 4 in (100mm), the sand was compacted around the slope edges so as to maintain a slope of 1V:1.5H. While compacting close to the edge of a layer, a support to the slope surface was provided using a rectangular flat plate trowel to prevent collapse due to lateral pressure and vibration. This support helped to maintain a smooth side-slope. The embankment model was left for a day before the flume experiment was performed. Penetration resistance measurements and moisture content were recorded before each flume experiment. At least five in-situ penetration resistance tests were performed on soil at the top of the embankment as shown in Figure 6 to ensure compaction consistency.

Clayey Sand Abutment

Construction procedures for the clayey-sand abutment were similar to the sand abutment procedures. The target densities were achieved by mixing the clayey sand with calculated moisture content and compaction effort. Spreading of soil, sprinkling water and mixing were carried out carefully to produce a homogeneous soil. The soil was compacted in layers in the test section. Figure 7 shows 15 torsional vane shear tests typically performed on a clayey sand abutment to ensure compaction consistency. The in-situ shear strength of the compacted clayey sand was determined based on the correlation charts developed for using the torsional vane shear tester.
LABORATORY SOIL EXPERIMENTS

Overview

A substantial challenge for this study was to determine the shear strength of different model soils used in the flume experiments. Because the shear strength of the model soil generally varies with the length scale of the model, the present study required shear strengths about 1/30th of shear strengths at actual abutments. A further challenge was that the relatively small size and
modest thickness of the model abutments complicated the accurate measurement of soil strength. Furthermore, a standard device was not available to predict the strength of model soil used to form the small size model spill-slope abutment. In fact, actual in-situ soil testing equipment is intended for, and works best with, thick soil formations. This challenge was addressed by development of an indirect approach to determine shear strengths of uniform sand and clayey sand using customized in-situ test devices. A series of laboratory soil experiments was conducted to calibrate the measurements obtained from customized in-situ test devices with soil strengths. Correlation charts developed based on the proposed calibration procedures were used for quantifying the shear strengths of model spill-through abutments constructed in the flume experiments. Calibration procedures were proposed for both uniform sand and clayey sand as described in the following sections. The full description of the calibration procedure was documented in Chakradhar (5).

Shear Strength of Uniform Sand

The particle size distribution of sand was performed in accordance with ASTM D422 (9), and it was classified as a poorly graded (uniform) sand (SP) as per the USCS, ASTM D2487 (8). Relative density (Dr) tests to determine maximum and minimum dry index unit weights of sand were carried out in accordance with ASTM D4253 (10) and ASTM D4254 (11), respectively. The maximum dry index unit weight was determined to be 108 pcf (17 kN/m³) at a minimum void ratio of 0.53, while the minimum dry index unit weight was 88 pcf (13.9 kN/m³) at a maximum void ratio of 0.87. Seven sand samples were prepared to attain relative densities ranging from 50% to 100%. Since actual sand abutments are generally compacted with Dr values well above 50%, testing on sand samples with Dr values less than 50% was not needed. To achieve a series of relative densities, methods from manual tapping on the mold surface using steel rod to create vibration to changing the total surcharge load during vibration using a shake-table were adopted.

A needle penetrometer consisting of a spring-loaded plunger with a needle attached to its end was used to determine the penetration resistance (R) of each compacted sand sample. Although the needle penetrometer was developed for testing fine grained soils (12), a similar test procedure was adopted here as an approach for controlling sand compaction in the flume experiments. The test procedure of pushing a needle into sand is analogous to pushing an electric cone penetrometer during a Cone Penetration Test (CPT) or a split-spoon sampler during a Standard Penetration Test (SPT). Using this analogy, it is justifiable to adopt the needle penetrometer test on sand in this study. A minimum of three penetration tests using the needle penetrometer were carried out on each compacted sand sample. An excellent correlation with a coefficient of determination (R²) of 0.95 between penetration resistances (R) and relative densities was developed as shown in Figure 8. This correlation further validates the application of the needle penetrometer on sand. The correlation shows that a higher resistance is achieved when penetrating the needle penetrometer into a highly compacted sand sample. Note that test data were not available between 82% and 100%, because compaction within this range could not be adequately and sensibly controlled in spite of many trials. This challenge was attributed to the smaller range of unit weights (from 104 pcf or 16.4 kN/m³ to 108 pcf or 17 kN/m³) in a relatively higher range of relative densities (from 82% to 100%).
A series of direct shear tests of sand were performed in accordance with ASTM D3080 (13) on sand samples compacted to three targeted relative densities of 50%, 60% and 80%. The tests were performed at effective normal stresses ($\sigma'$) ranging from 313 psf (15 kPa) to 2611 psf (125 kPa). Nonlinear failure envelopes were observed in Figure 9, because the compaction caused interlocking and dilation of sand particles during shearing. Sand samples compacted in thicker layers could not produce a consistent $D_r$ of 50%, resulting in relatively scattered data points in Figure 9. This challenge was attributed to the greater effect of compactive forces on the top layer compared to underlying layers. In order to generate a reliable failure envelope, seven direct shear tests were carried out on this sand. In contrast, the compaction of thin sand layers for $D_r$ of 60% and 80% produced more consistent results. Thus, only four direct tests were performed on sands for $D_r$ of 60% and 80%. Using the failure envelopes developed in Figure 9 and the effective normal stresses calculated in Table 1 based on an 8-in (200 mm) thick model sand abutment with a water level at 5.5 in (140 mm), shear strengths ($\tau$) determined from Figure 9 were plotted with the corresponding relative densities in Figure 10. The correlations developed in Figure 8 and Figure 10 were then used to quantify the shear strength of the sand compacted in the flume experiments.

### Table 1 – Stresses of sand at three different relative densities

<table>
<thead>
<tr>
<th>Relative Density, $D_r$ (%)</th>
<th>Unit Weight, $\gamma$ (pcf)</th>
<th>Normal Stress at 8-in Depth, $\sigma$ (psf)</th>
<th>Pore Pressure of 5.5-in Water, $u_w$ (psf)</th>
<th>Effective Normal Stress at 8-in Depth, $\sigma'$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>97.27</td>
<td>64.85</td>
<td>28.60</td>
<td>36.25</td>
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<td>60</td>
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<td>80</td>
<td>103.56</td>
<td>69.04</td>
<td>28.60</td>
<td>40.44</td>
</tr>
</tbody>
</table>
Similar to the uniform sand, a series of standard laboratory soil tests were performed to classify the cohesive soil and determine basic soil properties. These standard laboratory soil tests were 1) particle size distribution and hydrometer tests (9), 2) Atterberg limit tests (14), and 3) standard Proctor compaction tests (15). The soil was classified as clayey sand (SC) as per USCS (8). The maximum dry unit weight (γ_d-max) was determined to be 123 pcf (19.45 kN/m³) and the corresponding optimum moisture content (ω_{opt}) was 12%. After completing a standard Proctor test, a laboratory miniature vane shear device was used in accordance with ASTM D4648 (16) to determine the undrained shear strength (S_u) of the compacted clayey sand as illustrated in Figure 11. A minimum of four vane shear tests were performed on each soil sample. Tests were
repeated on soil samples compacted from 89% to 100% of $\gamma_{d\text{-max}}$. Similar compaction was not feasibly performed on soil samples at the wet-side of $\omega_{\text{opt}}$ at dry unit weights less than 89% of $\gamma_{d\text{-max}}$ (i.e., at a higher moisture content), because adhesion of soil to the Proctor hammer hindered the compaction process. Tests were not performed on clayey sand at the dry-side of $\omega_{\text{opt}}$, because it was difficult to obtain an adequate penetration and rotation of the vane shear tester on the soil. Furthermore, clayey sand model abutments were compacted at the wet-side of $\omega_{\text{opt}}$. Based on these test results, a relationship between percent compaction and $S_u$ was developed (Figure 12).

![Figure 11 – A torsional vane shear test performed on a clayey sand sample compacted in a 4-in mold using the standard Proctor method](image)

A series of direct shear tests was performed to determine two strength parameters (i.e., cohesion and friction angle) of clayey sand in accordance with ASTM D3080 (13). To adequately control the dry unit weight of clayey sand samples for the direct shear test, the following soil sample preparation steps were developed:

1. Clayey sand was compacted in a standard 4-in (101.6 mm) mold in accordance with ASTM D698 (15);

2. Using a soil extruder, the compacted soil was pushed from the mold into a 2.5 in (63.5 mm) diameter thin-wall Shelby tube with a metal plate custom-made to collect a 2.5 in (63.5 mm) diameter undisturbed soil sample as illustrated in Figure 13. The process of sampling the soil into the Shelby tube was carried out with caution to minimize soil disturbance and prevent any loss of moisture;

3. Soil collected in the Shelby tube was extruded and trimmed into a 1 in (25.4 mm) length without crumbling the soil;

4. During the trimming process, a smooth cut surface was required to ensure a uniform distribution of the normal stress applied during the direct shear test; and
(5) The prepared soil sample was carefully placed into the 2.5 in (63.5 mm) diameter shear box for a subsequent direct shear test.

![Graph showing the relationship between percent compaction and undrained shear strength](image)

**Figure 12** – A relationship of percent compaction and undrained shear strength of clayey sand using vane shear tester

![Illustrations showing sample extraction into the Shelby tube](image)

**Figure 13** – Illustrations showing how a clayey sand sample was extracted into the Shelby tube

Direct shear tests were carried on soil samples prepared at 91%, 97% and 98% of $\gamma_{d,\text{max}}$, and the respective failure envelopes were plotted in Figure 14. Table 2 summarizes the cohesion values and friction angles determined from the direct shear tests. Using these strength parameters and the corresponding normal stress at the model abutment depth of 8 in (200 mm), shear strength ($\tau$) was determined using the linear Mohr-Coulomb failure relationship and plotted...
against percent compaction (Figure 15). This relationship shows that shear strength of clayey sand increases with increasing percent compaction. Using the the correlation developed in Figure 12, the percent compaction of clayey sand model abutments constructed for the flume experiments can be estimated based on the $S_u$ value measured using the vane shear tester. Knowing the percent compaction, the shear strength of the model abutment can be estimated using the correlation developed in Figure 15.

**Figure 14** – A plot of normal stress vs. shear stress of clayey sand obtained from direct shear tests

**Figure 15** – A relationship of percent compaction and shear strength of clayey sand
Table 2 – Clayey sand properties at three levels of compaction

<table>
<thead>
<tr>
<th>Percent Compaction (%)</th>
<th>Unit Weight, $\gamma$ (pcf)</th>
<th>Cohesion, $c$ (psf)</th>
<th>Friction Angle, $\phi$ (degree)</th>
<th>Normal Stress at 8-in Depth, $\sigma$ (psf)</th>
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<tbody>
<tr>
<td>91</td>
<td>111.93</td>
<td>456.51</td>
<td>16.3</td>
<td>74.62</td>
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<td>97</td>
<td>119.31</td>
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<tr>
<td>98</td>
<td>120.54</td>
<td>594.27</td>
<td>39.0</td>
<td>80.36</td>
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</table>

DISCUSSION

This study describes the difficulties associated with controlling the overall compaction of model soil abutments in laboratory flume experiments and quantifying the shear strength of the model soil abutments. Several unresolved difficulties require further investigation:

1. While consistent compaction of model abutments was achieved in successive layers by controlling the amount of compaction effort and moisture content, it has been a challenge to apply the same compaction procedure on and near the model spill-slope, where the thickness and curvature changed with depth. Careful attention should be given to forming the model spill-slope and preventing collapse due to compaction.

2. Errors accumulated from the correlation methods for soil shear strength estimation were recognized. These errors could affect the accuracy of the estimated soil shear strength, which is an important factor in the scour investigation. Fortunately, the excellent correlations obtained from this study (i.e., $R^2$ greater than 0.8) will minimize these errors. A direct measurement of soil shear strength in flume experiments will be a good option if an in-situ measurement device, such as a miniature cone penetrometer, is available. This device will improve the efficiency of performing flume experiments and increase the accuracy of shear strength measurements.

3. In-situ shear strength was quantified on the flat surface of model soil abutments, as illustrated in Figures 6 and 7, using the correlations developed based on hand-held devices. Since the scour process is significantly influenced by the combination of geotechnical and hydraulic processes on the compacted spill-slope surface, it is reasonable to determine the shear strength of soil on or near the slope.

4. Changes in the shear strength of model soils before and after filling the flume with water as well as during the experiment were very difficult to quantify. When the flume was filled with water, the development of negative capillary pressure in model soils above the water level due to initial wetting created an apparent cohesion which attracted soil particles and contributed additional shear strength. However, the continuous wetting of model soils over time and during flume experiments could destroy this apparent cohesion. These changes in soil shear strength explain the typical abutment failure mode observed in the flume experiment: under-cutting at abutment toe, development of tension cracks at abutment crest and subsequent toppling of soil blocks.
(5) Consistent compaction of sand bed flood plain was required between each experiment in order to make them comparable. Uniform and consistent compaction as well as the tests to confirm the consistency was a big challenge.

CONCLUSIONS

Our laboratory study led to the following early new insights regarding geotechnical effects of abutment scours:

(1) A combination of geotechnical and hydraulic processes erode the compacted earth fill spill-slope of bridge abutments. These processes cause under-cutting, the development of tension cracks due to loss of support, and the subsequent toppling of soil blocks to occur sequentially along the face of the spill-slope, starting at the upstream corner where the velocity was highest. This process erodes the spill-slope, and eventually exposes the abutment column. Further erosion then can result in breaching of the embankment approach to an abutment.

(2) The approximate failure block size observed in the scale-reduced flume model varied with soil strength.

(3) The scour depths measured near the model abutments formed of sand compacted to a range of densities were found to correlate with soil shear strength, although only when soil strength exceeded a value of about 167 psf (8.0 kPa). For soil strengths less than this value, scour depths varied little with soil strength. This trend occurred largely because the strength of the spill-slope soil was difficult to control consistently for the less compacted sands. For soil (sand) strengths above about 167 psf (8.0 kPa), scour depth increased with increasing soil (sand) strength. This observation is attributed to a slower erosion rate of a higher strength spill-slope, which provides a long time and higher flow velocity to erode the sand bed (i.e., river bed), causing a deeper scour.

(4) We used a correlation method to determine the strength of compacted embankment soil used to form model abutments in the flume. This indirect method was useful but can be substantially improved, as it involved several error sources. One source was the relatively small number of data used to develop the correlation. A more efficient procedure would be to use an instrument that directly measures the shear strength in flume experiments. One good option is the use of a miniature Electric Cone Penetrometer. This instrument would make it much easier to conduct experiments with a broader range of soil strengths.
REFERENCES


FIELD TRIP GUIDEBOOK FOR THE
65th HIGHWAY GEOLOGY SYMPOSIUM
Laramie, Wyoming
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Field Trip Logistics

Welcome to the 65th Highway Geology Symposium in Laramie Wyoming. We will be leaving the Hilton Garden Inn for the field trip at 7:30 AM in deluxe 50 passenger coaches, and heading up the Snowy Range Scenic Byway. We will be traveling and stopping at elevations ranging from 7,000 feet to close to 11,000 feet. The weather can change quickly so please bring warm clothing and be sure to drink plenty of water. A catered lunch will be provided at the Snowy Range Ski Area. After lunch we will travel to the Vedauwoo Recreational Area before returning to Laramie by 5:00 PM.

Field Trip Sponsors include; **Geobrugg**, Lunch; **Golder Associates**, field trip refreshments; and **Uretek USA**, Box Breakfast.

---

Field Trip Itinerary

<table>
<thead>
<tr>
<th>Time</th>
<th>Activity</th>
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</thead>
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<tr>
<td>7:00 AM</td>
<td>Busses Arrive at Hilton Garden Inn</td>
</tr>
<tr>
<td>7:30 AM</td>
<td>Busses Leave Hilton Garden Inn, Box Breakfast Provided</td>
</tr>
<tr>
<td>7:50 - 8:30 AM</td>
<td>Stop 1: Overland Trail Turnout</td>
</tr>
<tr>
<td>9:00 - 9:40 AM</td>
<td>Stop 2: French Slate, Nash Fork Campground</td>
</tr>
<tr>
<td>9:45 - 10:45 AM</td>
<td>Stop 3: Libby Flats Observation Point</td>
</tr>
<tr>
<td>10:50 - 11:50 AM</td>
<td>Stop 4: Mirror Lake and Lake Marie</td>
</tr>
<tr>
<td>12:00 - 1:30 PM</td>
<td>Lunch: Snowy Range Ski Area</td>
</tr>
<tr>
<td>1:35 - 2:15 PM</td>
<td>Stop 5: Pinedale and Bull Lake Glacial Deposits</td>
</tr>
<tr>
<td>3:00 - 3:30 PM</td>
<td>Stop 6: Lincoln Memorial &amp; Rest Area</td>
</tr>
<tr>
<td>3:40 - 4:40 PM</td>
<td>Stop 7: Vedauwoo</td>
</tr>
<tr>
<td>5:00 PM</td>
<td>Arrive Back at Hilton Garden Inn</td>
</tr>
</tbody>
</table>
Highway Map showing the Field Trip Route and stop locations. In the morning we will travel to the Medicine Bow Mountains along the Snowy Range Scenic Byway. After lunch we will travel east to the Laramie Mountains and the Vedauwoo Recreational Area.
Field Trip Overview

TRANSPORTATION AND HISTORY ACROSS THE LARAMIE PLAINS (Julie Francis)

Despite the high elevations, long winters, and howling winds, humans have called the Laramie Plains and surrounding areas home since the end of the Pleistocene. A few of the oldest sites in North America (ca. 13,000 years old) have been found in the mountainous uplifts surrounding the basin. These document usage of extinct Pleistocene megafauna (mammoth and bison) and ceremonial use of red ochre deposits found near Sunrise, WY. Younger Paleo-Indian sites suggest that small groups moved through the area, utilizing a wider range of large and small game.

In the interior Laramie Basin, limited Holocene deposition (often gravels) along the Laramie River and severely deflated upland surfaces are not conducive to site preservation. In addition, the near treeless plain offers little shelter. For these reasons, most of the known archaeological sites have been found in the foothills of the surrounding mountains and consist of a few rock shelters and large stone circle (tipi ring) encampments at the mouths of drainages. Archaeologists have generally inferred that prehistoric peoples practiced generalized hunting and gathering throughout the Archaic and Late Prehistoric periods. However, the Late Prehistoric age Willow Springs bison jump in the southern basin reflects the intensified communal bison hunting that developed across the entire region during the last 500 years. Without doubt, the ancestors of many different modern Native American tribes traveled through the Laramie Plains. By the Protohistoric period (immediately prior to contact with Euro-Americans), the area was part of the homeland of the Arapaho people. Their usage of the area reflects more than day-to-day activities; rock art sites, vision quest sites and medicine wheels chronicle the spiritual importance of the region to native peoples.

Trappers and traders were the earliest Europeans and Americans in the Laramie Plains, arriving perhaps as early as the 1810's. The region was not heavily exploited for beaver and other pelts. Nevertheless, a river system, mountain range, a prominent peak, a fort, a county and a city (in a county of another name) bear the name of Jacques LaRamie, a French trapper who died in 1820.
With increased need for transcontinental communication, transportation played a key role in fostering permanent settlement and development of the Laramie Plains in the latter half of the 18th century. In 1862 Ben Holladay opened the Overland Trail as a southern alternative to the Oregon Trail. Home stations such as the Big Laramie Stage Station became some of the earliest permanent settlements on the Laramie Plains.

The first transcontinental railroad forever changed the history of the Laramie Plains. In May 1868, the first train arrived in the newly founded town of Laramie, consisting of tents and shacks. The establishment of the Fort Sanders Military Reservation in 1866 also played an important role in the development of the young town. Fort Sanders was de-commissioned in 1882, with many of the buildings sold and moved into Laramie. Several of these buildings still survive, but the imprint of the UPRR on Laramie is also clear. With steady employment and the influx of families, what are now some of the oldest homes in town were built. Concentrated on the Westside (immediately west of the UPRR tracks), the modest workers housing reflects the longstanding importance of the railroad and later industries to the community.

![Arrival of Union Pacific Officials, Laramie City, June 1868 (from Wyoming Tales and Trails).](image)

Railroad workers, soldiers, freighters, station tenders, and livestock all have to eat, and transportation provided the initial stimulus for agricultural development of the Laramie Basin. The Overland Trail stations became some of the earliest ranches, and for a few short years in the 1880s, the cattle barons reigned supreme. There were also attempts at large scale irrigation to develop farmsteads. However, by 1893 the harsh climate doomed this venture to failure. Other attempts at farming, primarily by small scale
homesteaders between 1917 and 1924 also failed, with only a few root cellars now remaining. Today, the agricultural economy consists primarily of stock raising.

Transportation also played a pivotal role in the development of the timber industry. A continuous need for railroad ties by the UPRR spurred the development of tie hack camps throughout the mountains. Ties were hand-cut and floated to Tie Siding, where they continued down the Laramie River to the treatment plant on the south side of Laramie. The automobile age arrived in Laramie at an early date. Local bicycle shop owner and inventor, Elmer Lovejoy, is credited with building Wyoming’s first car in 1897-1898. This pre-dated the October 31, 1913 dedication of the Lincoln Highway, the nation’s first trans-continental automobile road, by several years. As one of the larger stops on the Lincoln Highway across Wyoming, the transformation of Laramie from horse to automobile, blacksmith shops and livery stables to garages and gas stations, freight wagons to semis, and dirt two-track to Interstate had begun.

The Wyoming Highway Department was founded in 1917 and began improvements on the Lincoln Highway in Albany County in the early 1920's. Often, routes were changed and shortened by miles. The Lincoln Highway became U.S. 30 in 1928, and Interstate 80 opened between Laramie and Walcott Junction in the early 1970's. This route cut-off
nearly 19 miles from the old U.S. 30 route and was nicknamed the “Snow Chi Minh” Trail for its severe winter weather and treacherous conditions. The adverse conditions along the new route lead to the development of snow fences by the late physicist Dr. Ron Tabler. These fences act to catch snow, reduce drifting and ultimately lower plowing costs. In addition, variable speed limits reduce vehicular speeds during inclement weather. Coupled with the snow fences, this has greatly improved the safety of winter time travel across the Laramie Plains.

Snow fence along Wyoming Highway 130.

Despite the hazards of long winters and short growing seasons, the City of Laramie has grown throughout the 20th and into the 21st centuries. Along with transportation and businesses built around the travel industry and commercial shipping, the University of Wyoming anchors the city. Founded in 1886, four years prior to statehood, it is the only four year institution of higher learning in Wyoming. With over 13,000 students from all 50 states and 94 countries, UW offers 80 undergraduate and over 90 graduate programs of study, many of which are internationally recognized. UW is the hub of education in Wyoming and hosts many of the major cultural and athletic events in the State, creating a unique atmosphere in what remains a town steeped in the western heritage.

MINING THE MEDICINE BOWS (Julie Francis)

The Medicine Bows Mountains are dotted with evidence of historic mining of a variety of precious metals. Evidence includes prospect pits, adits, tunnels, shafts, shaft houses, stamp mills and other processing facilities in varying states of repair, outhouses, and what is left of the small nearby settlements. Thybony, Rosenberg, and Rosenberg (1985) and Hausel (1993) have extensively studied mining in the Medicine Bows. The mining history of the Medicine Bow Mountains is largely one of failed dreams, and as
noted by Rosenberg in reference to the Queen Mine outside Centennial is “mere footnote” in comparison to the gold rushes in California, Colorado, Montana and Nevada. Nevertheless, mining has done much to shape the history and modern character of the area as reflected by the many colorful place names.

Placer, lode and some hydraulic mining were undertaken in the Medicine Bows. Gold nuggets were discovered along Moore’s Gulch in 1868, with hydraulic mining commencing by 1876. Several townsites, including Cinnabar City and Douglass City, were built for the miners. There is still some evidence of hydraulic mining, but Rob Roy Reservoir has inundated much of the physical remains. Gold in 1876 and platinum in 1923 were mined on Centennial Ridge. The Keystone and New Rambler districts are likely the most successful lode mining operations in the Medicine Bows. Gold and copper were discovered at Keystone in 1876, and the New Rambler District mines were active between 1900 and 1918. Although it was primarily copper recovered in the New Rambler District, platinum and palladium were also commercially produced.

Exaggerated claims in the media, along with promotional schemes, and wishful thinking fueled the so-called “rushes” in the late 19th and early 20th centuries. Increased mining activity can be associated with higher gold prices. In addition, many unemployed citizens came to the area during the Great Depression to re-work both placer and lode deposits. As a result, the surviving sites have been considerably altered and modified from their original development.

Remains of a miners cabin at Libby Flats.
Mountains: Most mountains in Wyoming were exposed and elevated about 60 million years ago during Laramide deformation. These mountains are quite young by geologic standards yet, the oldest rocks in the core of the mountains are about 2.8 billion years old. The episode of mountain building elevated rocks above sea level and together with further elevation gains in the Tertiary provided the necessary stream gradients whereby the streams of the region could proceed to dissect the rocks into the existing landforms. In most of Wyoming’s mountainous areas, the difference of elevation is in part due to uplift of large segments of the Earth’s crust in the form of folds and wrinkles, or blocks bounded by faults, or a combination of both. Most of Wyoming’s larger mountain ranges also have an exposed core of very ancient Precambrian metamorphic and igneous rocks.

Basins: Wyoming Basins are a group of intermontane depressions that lie within the Rocky Mountains between both large and small uplifts. The large uplifts lack the continuity expressed in the surrounding provinces, and in traveling west from Rawlins on Interstate 80 one may not realize how many mountain ranges Wyoming has. The basins are surface topographic depressions, and they are compound downfolds (synclines) in which layered sedimentary rocks dip towards the lowest point or trough. Along the basin margins the sedimentary strata often out crop as hogbacks exposing limestone, gypsum, bentonite, phosphate rocks, and building stones. Farther out in the basin are some of the great coal fields and uranium deposits of Wyoming. The rock units that extend under the basins at depth act as reservoirs for oil and gas accumulations. Major basins within Wyoming include the Powder River Basin, Bighorn Basin, Wind River Basin, Greater Green River Basin, Hanna Basin, Laramie Basin and the Denver Basin.

Despite the rather limited vegetative cover, wide lonesome stretches of Wyoming's basins provide a large amount of pasture for sheep, cattle, and wild life that is utilized particularly in the winter months, when snowmelt provides water for animals.

Plains: In Wyoming the high plains are developed primarily of a series of flat lying rocks of Cenozoic age. These Cenozoic rocks had their origin in debris ejected from volcanoes far to the west and carried eastward by a complex of rivers. Later, uplift of the crust allowed streams to cut deeply into these rocks, to develop broad floored valleys, and to leave broad interstream divides and occasional scarps with small buttes along the margins. A typical high plains landscape is seen along Interstate 80 from near Pine
Bluffs west towards Cheyenne. The surface of the plain rises steadily in elevation until at a point about 20 miles west of Cheyenne it extends continuously onto the flank of the Laramie Mountains forming the “Gangplank”. The “Gangplank” is a narrow strip of Tertiary rocks upon which the high plains surface is preserved. The following four block diagrams (Stages 1 through 4) illustrate the schematic sequence of the formation of Wyoming's mountains and basins: (1) faulting, folding and erosion expose the Precambrian basement rock; (2) basins are filled with younger sediment derived from adjacent mountains and from intermittent ash fall; (3) regional uplift and faulting and; (4) erosional processes shape the current landforms.

Stage 1: Mountains and basins formed by folding and faulting, extensively modified by erosion. (Diagram by Jim Rogers)

Stage 2: Basins partially filled with younger sediments derived from adjacent mountains and intermittent volcanic ash fall. (Diagram by Jim Rodgers)
Stage 3: Basin filled to overflowing at low places on divides, followed by regional uplift with tilting and faulting. (Diagram by Jim Rodgers)

Stage 4: Present cycle of erosion- basin excavated. Youthful canyons cut across resistant cores of mountains. Basin floor are lowered as the canyons are deepened. (Diagram by Jim Rodgers)
EXPLANATION

Quaternary Rocks and Unconsolidated Deposits
Qa - Alluvium and colluvium
Qt - Gravel, pediment and fan deposits
Qg - Glacial Deposits-till and outwash of sand, gravel, and boulders
Qls - Landslide Deposits locally includes intermixed landslide and glacial deposits, talus, and rock-glacier deposits

Lower Quaternary and Tertiary Rocks
QTC - Conglomerate Pleistocene to Miocene giant granite boulders in an arkose matrix
Tmu - Upper Miocene Rocks. Light colored tuffaceous claystone, sandstone, and conglomerate
Tha - Hanna Formation - brown and gray sandstone, shale, conglomerate, and coal; giant quartzite boulders near Medicine Bow Mountains

Cretaceous
Kmb - Medicine Bow Formation - brown and gray sandstone and shale; thin coal and carbonaceous shale beds
Kmv - Mesaverde Group Light gray sandstone, soft shale, and thin coal beds
Ks - Steele Shale (78-82 Ma) Gray soft marine shale containing numerous bentonite beds and thin lenticular sandstone
Kn - Niobrara Formation (~83 Ma) Light colored limestone and gray to yellow limy shale
Kf - Frontier Formation Gray sandstone and sandy shale

Jurassic
Js - Sundance Formation Greenish gray glauconitic sandstone and shale underlain by red and gray non-glauconitic sandstone and shale
Triassic
Trc - Chugwater Group - red shale and siltstone containing thin gypsum partings near the base
TrPjs - Satanka Shale - red shale

Pennsylvaniaian
Pfs - Forelle Limestone - thin bedded limestone. Locally is a member of the Goose Egg Formation
PPcf - Casper and Fountain Formation - gray, tan, and red thick bedded sandstone underlain by interbedded sandstone and pink and gray limestone.

Middle Proterozoic
Ys - Sherman Granite (1,415 - 1,435 Ma)
Yla - Laramie Anorthosite Complex Anorthosite and norite

Early Proterozoic
XLc - Libby Creek Group Pelitic schist, amphibole schist, quartzite, diamicite, quartz-pebble conglomerate and marble
Xsv - Meta sedimentary and metavolcanic rocks. Granite gneiss, layered amphibolite, hornblende gneiss, and amphibolite
Xgy - Granite rocks of 1,700 Ma age group
Xm - Mullen Creek and Lake Owens Mafic Complexes; older than 1,700 Ma

A portion of the Geologic Map of Wyoming (Love and Christiansen, 1985) showing the field trip route and stops.
FIELD TRIP GEOLOGY OVERVIEW

Just south of the Colorado-Wyoming border, the Southern Rocky Mountains divide into three prongs that extend northward into Wyoming. These three prongs are known, from east to west, as the Laramie Mountains, the Medicine Bow Mountains, and the Sierra Madre. The Laramie Mountains are the most clearly related to, and are an extension of, the Colorado Front Range. In Wyoming, the Laramie Mountains separate the Denver Basin and Hartville uplift from the Laramie and Shirley basins to the west. Casper Mountain, a northern salient of the Laramie Mountains, borders the southern margin of both the Casper Arch, and the Powder River Basin.

![Map of Wyoming](image)

Major structural elements of Wyoming; (Geology of Wyoming)
**Laramie Mountains**: Near the border between Wyoming and Colorado, the Laramie Mountains have been reduced by erosion to low relief. The Laramie Mountains are more rugged to the north, and they are capped by Laramie Peak (elevation of 10,274 feet), one of the famous landmarks along the Oregon Trail. The Laramie Mountains were referred to many trappers and military men as the “black hills” because of their dark profile when viewed from the east near Fort Laramie. The basic geologic structure is that of a large asymmetric arch in the earth’s crust with a steep flank on the eastern side and a gentle slope to the west. Precambrian rocks are extensively exposed in its core, which is flanked by sedimentary strata. South of Laramie diamonds were discovered and are thought to be at least one billion years old. The diamonds formed under high temperatures and pressures in the Earth’s mantle.

Looking eastward toward Cheyenne at "the Gangplank." Interstate Highway 80 and the Union Pacific Railroad follow the Gangplank from the High Plains in the distance onto the Precambrian rocks (older than 570 m.y.) of the Laramie Mountains in the foreground. (From Trimble, 1980).

Crossing the Laramie Mountains between Cheyenne and Laramie, Interstate 80 and the Union Pacific Railroad first traverse a relatively flat surface of late Tertiary sedimentary rocks called the “Gangplank” finally reaching the Precambrian core of the mountains and a rolling upland of low relief about 8,000 feet above sea level called the Sherman surface. The Gangplank is the only place along the entire eastern mountain front where
the Tertiary rocks that once buried the Laramie Mountains are still preserved and are in contact with the Precambrian core of the mountain. Further erosion during later Tertiary time left a few high granite boulder knobs such as the area known as Vedauwoo.

**Laramie, Hanna and Carbon Basins:** Located between the Laramie Range and the Medicine Bow Mountains, is a complex syncline structural feature, the trend of which roughly parallels the sweeping arc of US Highway 30 and the Union Pacific Railroad from Laramie to Rawlins. The basin is shallowest in the southern part but reaches an extreme depth in the area north of the coal mining community of Hanna. The deep basin is quite small as intermontane depressions go, only 35 miles by 20 miles, but it is unique because of the great depth to which sedimentary rocks are depressed.

The Hanna and Carbon basin contain a thick sequence (up to 23,000 feet) of Upper Cretaceous and Tertiary clastic sedimentary rocks derived in part from adjacent uplands. The Hanna and Ferris Formations contain thick coal seams. Coal was originally mined underground at old Carbon and later at Hanna and was used to fuel steam locomotives on the Union Pacific Railroad; now coal is extracted by underground and surface mining methods and is used as fuel in electric-generating plants.

**Medicine Bow Mountains:** Located near the Wyoming-Colorado border west of Laramie the Medicine Bow Mountains encompass a large mountainous uplift of more than 900 square miles of which the Snowy Range is only a small part. The mountains are characterized by a rather broad flat summit at an elevation of about 9,000 feet above which rise two erosional remnants of rugged peaks with much higher elevation. The Rawah Peaks, in Colorado, are located near the southern extent of the range and the Snowy Range is located in the northern portion.

These ancient mountains record more than 2.5 billion years of geologic history in their outcrops, peaks and ridges. Rock successions similar to those found in the core of the Medicine Bow Mountains are quite rare on the surface of the earth today; only a few other places in the world, most notably in the Witwatersrand Basin of south Africa, contain similar rock successions.
In Wyoming, rocks of Precambrian age were buried under thick successions of younger rocks of Paleozoic, Mesozoic, and Cenozoic ages. However, there are extensive areas where erosion has removed these younger rocks and the older Precambrian rocks are exposed. The uplifted and exposed core of the Medicine Bow Mountains is such an area. The Medicine Bow Mountains of Wyoming are complex uplifted mass of Precambrian rocks forming the core of large asymmetric anticline bounded by thrust faults to the east. During the late Cretaceous to early Eocene the Medicine Bow Mountains were uplifted. A complex of structures developed along the margins of the mountains, and thick deposits of conglomerate and arkosic sandstone were laid down in the basins adjacent to the mountain uplift. In some areas there is clear evidence that the Precambrian basement was involved during this period of deformation. Major faults that developed during the Laramide deformation show a clear correlation with the basement structures. Sedimentary rocks of Paleozoic and Mesozoic age are along the flanks of
the mountains and rocks of Tertiary age are along the flank and caps gently dipping sedimentary rocks in the central part of the area. Glacial deposits of Pleistocene age are abundant in the northern part of the Medicine Bow Mountains, especially near the Snowy Range, where they originated as an ice cap near Medicine Bow Peak.

The same major shear zone (suture) that cuts across the Medicine Bow Mountains is also present in the Sierra Madre. This suture, called the Cheyenne belt, separates the oldest Archean rocks of what is known as the Wyoming Province to the north from younger igneous and metamorphic rocks to the south. Attached to the rocks north of the Cheyenne belt are Early Proterozoic metasedimentary rocks similar to those in the Medicine Bow Mountains to the east.

The oldest gneisses and schist of this range are about 2.5 billion years old. Stromatolites are also preserved in the Medicine Bow Mountains. These are some of the oldest known forms of life found as fossils in sedimentary rocks. These fossils are the remains of a type of marine algae, which lived approximately 1.7 billion years ago.

**Sierra Madre:** The Sierra Madre is the western prong of the Colorado Front Range that projects into Wyoming. The uplift separates the Saratoga Valley from the Washakie Basin, and may be structurally connected to the Rawlins uplift to the North. The crest of the Sierra Madre is the Continental Divide, and the highest point is Bridger Peak at 11,007 feet. Drainages on its west flank flow to the Colorado River by way of Little Snake River; drainages in its east flank flow to the North Platte River.

Geologically, the Sierra Madre are quite similar to the Medicine Bow Mountains, but they lack the broad upland surface of the latter. The Archean rocks of the Wyoming Province extend northward from the Sierra Madre to form the Precambrian cores of nearly all Wyoming mountain ranges. South of the Cheyenne Belt in the Sierra Madre are well-preserved metamorphosed volcanic and sedimentary rocks (including basalt, andesite, rhyolite flows, tuffaceous rocks, shales, and greywackes), as well as some younger granitic intrusions approximately 1.7 and 1.4 billion years old.
View of the Snowy Range and Sugarloaf from Brown's Peak.

Stromatolite. These fossils are the remains of a type of marine algae which lived approximately 1.7 billion years ago.
Above: Stratigraphic column of metasedimentary rocks in the Medicine Bow Mountains (modified and adapted from Houston and others, 1992).

Opposite: Geologic map of the Sierra Madre and Medicine Bow Mountains, southeastern Wyoming (From Houston, 1993). Key: BBLB- Barber Lake block, BCZ-Bear Creek shear zone, BLB-Bear Lake block, CB-Centennial block, CMZ-Central mylonite zone, DP-Divide Peak synclinorium, FCF-French Creek fold, LLF-Lewis Lake fault, LOMC-Lake Owen mafic complex, MCMB-Mullen Creek mafic complex, MCNF Mullen Creek-Nash Fork shear zone, MCQM-Mullen Creek Quartz Monzonite, RG-Rambler Granite, RLF-Reservoir Lake fault, RSZ-Rambler shear zone, SMZ-Southern mylonite zone, RS-Rudefeha syncline, SMG-Sierra Madre Granite.
Field Trip Stops

As we travel from Laramie to Centennial, we can observe six distinct alluvial surfaces. They range from the currently active Optimist terrace along the Big Laramie River to the Late Pliocene/Early Pleistocene (~12,000 bp) Table Mountain surface. The six alluvial surfaces are:

(1) The Optimist Terrace is the active terrace along the Laramie River.

(2) Paradise Stock Farm Terrace-bottom land along the river that includes the surface at the Territorial Prison.

(3) Pathlow Strath and Terrace is located at the slight rise in Highway 230 near the Four Seasons Angler Fly Shop. It is visible looking west as we pass under the interstate.

(4) Airport Terrace is the large expanse that includes the Laramie Regional Airport. The airport terrace gradient is 60 feet per mile in the west and 20 feet per mile at the eastern edge.

(5) Eagle Rocks Surface is a small remnant surface that is primarily a thin mantle of gravels, sourced from the Medicine Bow Mountains.

(6) Table Mountain Surface is the oldest discernible surface in the basin; there may be some other comparable surfaces in Centennial Valley. Although those surfaces may be younger terraces associated with the middle and south forks of the Little Laramie River.

STOP 1: OVERLAND TURNOUT (Highway 130, M.P. 9.6)

The Overland Trail: First opened to mail, stage, and freight traffic in 1862, the Overland Trail served as a southern alternative for transcontinental transportation to the Oregon Trail. Beginning in Atchison, Kansas, the route went up the South Platte River across Kansas, Nebraska and northeastern Colorado to the mouth of the Cache La Poudre River, turned north and up a valley over the Laramie Range, across the Laramie Plains, west through Bridger’s Pass and the Red Desert, and across the Black’s Fork River to Ft. Bridger. Well-known fur trapper William Henry Ashley first traveled this general route in 1825, followed by a group of Cherokee Indians headed to the California gold fields in 1849. Captain Howard Stansbury mapped it in 1850.
The earlier transcontinental mail and freight lines, operated by Russell, Majors, and Waddell, followed the Oregon Trail, along the North Platte River. Due largely to their financial investment in the Pony Express, this line failed in 1861 and was purchased by Ben Holladay. He moved the line south and began construction of the network of regularly-spaced Overland Stage Line home and swing stations. The home stations served as stops to change teams and drivers, provide food for passengers and included sleeping quarters, dining area, telegraph office, and barns. Swing stations included only a stable, granary, and room for one or two stock tenders. Two home stations were built on the Laramie Plains: one on the Big Laramie River, the other on the Little Laramie. These became some of the earliest ranches in the region.

The Overland Stage Line was short-lived. In 1866, Holladay sold all his overland mail holdings, and they were consolidated under the name Wells Fargo and Company. Wells Fargo stages traveled the Overland Trail until 1869, when stages were run between the termini of the Union Pacific and Central Pacific railroads. The completion of the transcontinental railroad spelled the end of commercial overland stage traffic. Today, the Overland Trail survives as a series of faint swales or wagon ruts, with a few short segments of corduroy road at the crossing of the Big Laramie River.

Source: Francis, Carroll, and Evans 1991

The Laramie, Hahn’s Peak and Pacific Railway: The narrow earthen berm that parallels much of the north side of State Highway 130 is the remains of the Laramie, Hahn’s Peak and Pacific Railway. Isaac Van Horn of Boston formed the company of the same name in 1901, with the goal of connecting Laramie with the gold mining districts near Centennial, serving the Grand Encampment Mining District in the Sierra Madre Range, and terminating at Steamboat Springs, Colorado. Construction of the railroad proceeded slowly, taking six years to cross the 30 miles between Laramie and Centennial. By 1907, hard rock mining in the Snowy Range had also sharply declined. To justify its existence, the railroad purchased a coal deposit known as Coalmont near Walden, Colorado and the route was detoured south of Centennial towards this new destination. Construction of the grade over the Snowy Range from Centennial was difficult, at best, due to the severe changes in elevation and included a sweeping set of horseshoe curves up the slope above Albany to Lake Owen. The first train reached Walden, Colorado on October 25, 1911 and reached the terminus at Coalmont in December of that same year. It had taken 10 years to construct the 111 mile long rail line, with a depot and associated rail yard facilities in the Westside Neighborhood of Laramie.
The Laramie, Hahn’s Peak and Pacific Railway was expensive and difficult to operate during the winter months due to heavy snows. During the winter of 1917, 21 days were needed to complete one round trip. Profitability of the railroad was minimal, if not non-existent, and in 1914, the first of many sales and name changes had begun. By the 1930s, the Interstate Commerce Commission requested Union Pacific to take over operation of the line. A merger deal was struck in 1935, but the Union Pacific petitioned for abandonment in 1941. The ICC denied this request, and the line continued to ship coal, timber, livestock and minerals. By 1951, the line became known as the Coalmont Branch of the Union Pacific, and the old depot was closed. Tri-weekly service continued to the 1980s, when the line was purchased by the Wyoming/Colorado railroad and operated as an excursion train. This operation ceased in the 1990s, and the railroad has been officially abandoned. Portions of the grade serve as a Rails to Trails route in the Medicine Bow-Routt National Forest.

Source: Rosenberg 1998
**Gateway to the Snowy Range and Medicine Bow Mountains:** The Overland Trail turnout, is a great place to view the gateway to the Snowy Range and Medicine Bow Mountains. Jelm Mountain, Sheep Mountain, Centennial Ridge, Snowy Range, Table Mountain, and Big Hollow are visible on the western horizon from this vantage point.

Physiographic map of the Medicine Bow Mountains and adjacent areas (From S.H. Knight, 1990).
Panorama of the gateway to the Snowy Range and Medicine Bow Mountains viewed from Wyoming 130 in the vicinity of Stop #1. The road sits on the Airport pediment surface. (Upper) In the foreground a few yards south of the road and fence is the 150-foot deep Big Hollow. The lone peak on the left (southwest) is Jelm Mountain, capped by the University of Wyoming infrared telescope observatory. On the horizon north (right of Jelm) Mountain is a long prominent ridge called Sheep Mountain, and lying in the Big Hollow basin east of Sheep Mountain is the Big Hollow oil field. (Lower) Highway 130 points at Centennial Ridge, the prominent ridge in the distance slightly to the north and west of Sheep Mountain. The Snowy Range rises above the rest of the Medicine Bow Mountains as the perennial, snow covered peaks (Adapted and modified from Hausel, 1993).
**Big Hollow Oil Field:** Big Hollow is a huge wind excavated depression (basin) that covers an area of more than 25 square miles and is 9 miles long, 3 miles wide, and 150 feet deep. No streams or rivers drain the basin. It is suggested that the basin formed by strong winds that have persisted in the area over a long period of time. Similar, but smaller wind deflation features are found at a number of sites along the western edge of the Laramie Basin. Many of these features are now filled with water, such as Lake Hattie and Porter Lake. It is thought that the Airport surface to the north and south was better armored by gravel derived from the Medicine Bow Mountains allowing the soft Cretaceous bedrock to be removed by elevated adiabatic winds coming from the ice sheets in the mountains to the west.

The Big Hollow oil field was discovered in 1938 and produced oil from the Muddy Sandstone (Early Cretaceous) at a depth of 800 feet. The oil is structurally controlled and trapped in the Big Hollow anticline.

**Table Mountain:** Table Mountain is the prominent, flat-topped hill about one mile south of the highway. The hill is an erosional remnant of the Table Mountain pediment surface. As the road proceeds to the west, down from the high bench into the Little Laramie River valley, a small lake is visible south of the highway. This is Porter Lake, a small, often water-filled, wind deflation feature.

Table Mountain as seen from the Snowy Range Scenic Byway at about M.P. 19.0.
M.P. 22.04 Pinedale Outwash Plain: Once we cross the Little Laramie River, gravel and cobble sized rocks become apparent on the surface. These rocks are associated with the large outwash plain from the retreat of the Pinedale glaciers. The cobbles become noticeably larger as we get closer to the mountains and their source.

Glacial outwash plain (boulder field) from the last Ice Age. The area of Centennial Valley is covered by an extensive field of boulders that originated in the high peaks of the Snowy Range.

**Centennial Ridge:** Centennial Ridge is the prominent ridge that abruptly rises from the basin floor immediately west of Centennial. Gold was discovered along Centennial Ridge in 1874, and was also found in the Middle Fork of the Little Laramie River even earlier. A short time later in 1876, the Centennial Ridge mining district was organized.
View of Centennial Ridge showing the location of a prospect pit (arrow) above the main tunnel of the Centennial mine. The main adit is hidden in the trees.

M.P. 27.42 Bull Lake Terminus: The town of Centennial is located at the terminal moraine of the Bull Lake, an Illinoian glacial maximum (~130,000 bp). The till associated with the Bull Lake glacial episode represents the glacial maximum during the last ice age for the Medicine Bow Mountains.

M.P. ~29.27 Pinedale Terminus: The Pinedale terminus is located at the neck of the valley. The terminal moraine of the Pinedale glacial stage, Wisconsin Stage (~12,000 bp), extends from the south eastern side of Libby Creek Canyon, across the Bull Lake till and down the North Fork of the Little Laramie River and abuts Corner Mountain.
Remains of the early gold rush in the Centennial Ridge mining district. The Queen Mine headframe on top of Centennial Ridge as it appeared in 1980 (Hausel, 1993).

Historical Train Depot in Centennial, now a museum.
Xvm- metavolcanic and metasedimentary rocks
Xvma-mylonitic biotite augen gneiss
Xvmb-migmatitic biotite schist and gneiss
Xmn Mafic intrusive rocks north of the Cheyenne Belt
~1700-2300 Ma
Xqm-metasedimentary rocks of the Snowy Pass Super Group
Libby Creek Group
Xf-French Slate

A portion of the Preliminary Geological Map of the Saratoga 30'x60' Quadrangle (Sutherland and Hausel, 2005).
STOP 2: FRENCH SLATE Nash Fork Campground (Highway 130, M.P. 35.19)

Folds in the French Slate Formation, Libby Creek Group of the Snowy Pass Supergroup.

The Precambrian rocks are divided into two major units by the Mullen Creek-Nash Fork shear zone, which is a part of the Cheyenne belt. North of the shear zone the older Precambrian rocks are a complex sequence of quartzo-feldspathic gneisses of the almandine-amphibolite facies. These gneisses are marked by the north to the northwest trending structure and cut by hundreds of dikes of basaltic composition that are also deformed and metamorphosed. South of the shear zone are Precambrian metamorphic rocks that include hornblende gneiss, sillimanite-biotite gneiss, diopside gneiss, marble, and quartz-biotite-andesine gneiss.

The structural geometry of rocks south of the shear zone is extremely complex. Most folds in the gneisses have axes that trend east-north-east to northeast, and it is evident from studies of the geometry of folds and lineations that the folds have been refolded. This evidence of refolding plus the obliteration of primary structure in the dikes, and the
lack of retention of the 2.4 b.y. event in the older granite and gneiss of this area combine to suggest that rocks south of the shear zone have been largely affected by deformation and metamorphism.

The French Slate is a dark brown to black finely laminated graphitic slate and phyllite that is the uppermost (youngest) unit of the Libby Creek Group. The French Slate consists of interbedded muscovite-chlorite slates and phyllites and includes some highly siliceous slates as well as one to two beds of quartzite near the top of the unit. Metacrysts of pyrite are scattered through the slates and phyllites. The slate has been deformed by movement along the adjacent Mullen Creek-Nash Fork shear zone and has produced some spectacular, complexly folded and crenulated beds. These rocks have been folded at least two different times. An early folding event produced tight isoclinal folds with parallel limbs. These were later refolded by open buckle folds.

STOP 3: LIBBY FLATS OBSERVATION POINT (Highway 130, M.P. 39.18)

View of the Snowy Range from Libby Flat. (Rick Carpenter, WYDOT Photographer)

The observation point platform is located on metadolomite of the Nash Fork Formation (Upper Libby Creek Group). Libby Flats is an open grassy terrain south and southeast of the observation platform. The area surrounding the observation point was once known as the LaPlata mining district, which included a number of small prospects and mines as well as the Lewis Lake gold deposit, the Red Mask mine, and the Bellamy Lake copper prospect. The mines and prospects in this district were never developed to any great extent because most of the rocks were poorly mineralized.
Libby Flats Observational Tower. Most of the rock types found in the Snowy Range Supergroup can be found in the tower, some of which show evidence of glacial movement.

To the southwest of the observation point, is the Saratoga Valley with the Sierra Madre range on the horizon. The Sierra Madre are geologically similar to the Medicine Bow Mountains, and includes the Encampment (also known as the Grand Encampment) mining district where more than 21 million pounds of copper, 29,000 ounces of silver, and 2,000 ounces of gold were recovered between 1898 and 1911.

About a quarter of a mile south of the Libby Flats observation point, along a jeep trail, are the remains of an historical cabin, which marks the site of an old prospect. The prospect is in brecciated metadolomite of the Nash Fork Formation and consists of angular metadolomite clasts cemented by massive and boxwork limonite. The limonite boxworks are butterscotch, spongy and porous, iron oxides that represent the remains of iron carbonate (siderite). Much of the former iron carbonate has been leached from the rock leaving open vugs in place of the former crystals. The matrix surrounding the vugs has been replaced by limonite. A sample of the boxworks collected from the prospect contained no detectable gold or silver.
Evidence of glaciation at Libby Flats consists of displaced rocks showing slieken sides, chatter marks, and other glacial wear. However, due to its high elevation, over 10,000 feet, and its geology, the ice that moved across it was not deep enough to significantly down cut into the surface. Libby Flats contains moraines associated with the 3rd Pinedale glacial sequence (P3). One of the moraines is north of the observation point and several are located east along Black Jack Lake. Looking north you can see the Snowy Range, which consists of highly resistant quartzite. The range remains today due in part to the inability of the glaciers to erode it.
Snow removal on the Snowy Range Scenic Byway (Rick Carpenter, WYDOT Photographer)

Snow Management of the Snowy Range Scenic Byway: Trying to keep the Snowy Range Scenic Byway open in an alpine environment throughout the winter would be a monumental task. Given the extreme difficulty of keeping the highway open and the low traffic volumes during the winter, WYDOT elected to close the highway to regular traffic during the winter months. Depending on the year, WYDOT closes the highway with the first major storm after the 1st of November. Waiting until at least November 1st allows access to the area for the elk hunting season, which usually ends around the first of November. The highway is closed for a 12-mile section over the top of Snowy Range Pass.

After the road is closed to regular vehicular traffic, another recreational activity begins. Snowmobiling along the route is very popular, and the closed portion of the roadway is used as a snowmobile trail. Parking areas are provided for snowmobiles at both ends of the road closure. Tall wooden poles are attached to the delineator posts along the road. These poles serve a dual purpose, they mark the trail for the snowmobilers to follow and they also mark the location of the road which is necessary when snow removal begins in the spring.
Snow removal on the Snowy Range Scenic Byway (Rick Carpenter, WYDOT Photographer)

The goal on a typical year is to have the highway opened to vehicle traffic by Memorial Day Weekend, the traditional beginning of tourist season. Snow removal begins from both ends with maintenance crews from towns on either side working to clear the road. Given the depth of the snow, the removal is a top down process with tracked snow cats beginning the process of delineating the path to be cleared and pushing away the upper layers of the snow. When the snow cats reach a depth that the snow has been compacted to ice due to snowmobile traffic and trail grooming, the bulldozers take over and continue to push snow out of the way. Once the bulldozers get to the point that the snow excavation is so deep they can no longer effectively push the snow away from the road, the rotary snow plows are used to chew up the layers of snow and ice and blow them out of the way until eventually the road surface is exposed again.

The effort required to open the road depends on the severity of the winter. In the near record setting snowfall year of 2010-2011 it took 6 weeks and $370,000 to open the road, which did not occur until mid June. The following year it took less than a week and $68,000 to complete the task. Planning and budgeting to open the road is a very complex task for WYDOT District Maintenance personnel.
October 7, 1955 Local authorities, reporters, and two priests reached the site where United Airlines Flight 409 crashed into Medicine Bow Peak—at the time, the deadliest accident in U.S. commercial aviation history, killing all 66 passengers (Randy Wagner, Laramie Daily Bulletin-Boomerang.

**United Airlines Flight 409:** A United Airlines Douglas DC-4 crashed into Medicine Bow Peak on October 6, 1955, killing all 66 people aboard the aircraft. Flight 409 took off from Chicago in the early morning and landed in Denver at 5:51 AM, more than an hour late because of bad weather. It was bound for Salt Lake City, then on to San Francisco. The customary route would take the flight north into Wyoming then west at the radio beacon at the tiny town of Rock River, safely around the Snowy Range. The pilot was familiar with the route, having flown it 45 times the previous year. But this time, he took a shortcut directly over the mountains, some 25 miles off course. The DC-4 was not pressurized and was attempting to fly well over the recommended altitude. The plane failed to clear the 12,013 foot peak by about 75 feet and crashed at approximately 7:30 A.M.
Recovery of the widely scattered remains of the victims was extremely challenging due to the difficult terrain at the crash site. At the base of the almost perpendicular cliff where the aircraft hit, movement was hindered by talus and large boulders all piled loosely on a steep slope. Mountaineers doing the recovery work also had to be wary of rock falls triggered by activities of those above them on the cliff face. Cold weather and occasional snow also played a role in impeding recovery efforts. The recovery of human remains was not completed until the evening of October 11, 1955, five full days after the accident.

After the investigation of the accessible wreckage was completed, United Airlines requested that the military destroy the remaining debris. Attempts were made to accomplish this, but despite the use of explosives, artillery fire, and according to most sources, napalm bombs dropped from seven F-80 fighter aircraft, complete obliteration of the wreckage was not possible. Small fragments of flight 409’s airframe and parts of the engines still exist in the area surrounding the crash site, which is now more than 50 years old and is federally protected (Couch).
In August 2001, a bronze memorial plaque was placed just west of the Medicine Bow Libby Flats lookout off of Wyoming Highway 130 (Snowy Range Road). The location faces the mountain where the crash occurred.
STOP 4: MIRROR LAKE (Highway 130, M.P. 40.81)

Mirror Lake fills a shallow glacially-carved basin in quartzite. The small tree covered hill on the far side of the lake is underlain by Sugarloaf Quartzite and the high cliffs of the Snowy Range ridge are underlain by Medicine Bow Quartzite. On the ridge, a dark colored mafic igneous dike cuts through the white Medicine Peak Quartzite (Photograph by Rick Carpenter, WYDOT).

The Snowy Range ridge rises approximately 1,000 feet above Mirror Lake to a height of 12,013 feet above sea level at Medicine Bow Peak. To the right of Medicine Bow Peak is Sugarloaf Mountain, which is comprised of the Sugarloaf Quartzite (Lower Libby Creek Group).

Mirror Lake fills a shallow glacially-carved basin in the Sugarloaf Quartzite. Outcrops of the quartzite form the low-lying ridges on the north side of the lake. The high cliffs of the Snowy Range ridge skyline are underlain by Medicine Peak Quartzite.
Like Mirror Lake, Lake Marie also occupies a shallow, glacially-carved basin. The south half of the lake is on resident Sugarloaf Quartzite, while the northern half of the lake lies in laminated, less resistant Lookout Schist.

The Medicine Peak Quartzite is a highly resistant, medium-to-coarse-grained white quartzite with layers of quartz-pebble conglomerate. The quartzite contains minor grains of sericite, black tourmaline, and light-blue transparent kyanite. The Sugarloaf Quartzite is also a white, medium-grained quartzite with minor sericite and black tourmaline.

Some of the sedimentary structures preserved in the quartzite boulders have provided clues on the depositional environment. Some of the boulders have bedding, cross bedding, and occasional beds of quartz pebble conglomerate. These suggest the Medicine Peak Quartzite was deposited as quartz sand in a riverbed or in the subtidal portions of a delta or an estuary. Paleocurrent directions from the quartzite outcrops elsewhere indicate the sediment that formed the quartzite was carried by water in the west-southwest direction. This implies there was a highland source area to the northeast, which supplied the sediment. Small-scale oscillation ripples in the quartzite suggest some of the quartzite was deposited as quartz sand in shallow water along the edge of an ancient ocean.

Mirror Lake sits in the 4th Pinedale glacial sequence. Based on its location, and the location of the 5th Pinedale Sequence, both were limited in time and coverage. This also means that the rocks moved by the two sequences are very angular and still contain easily viewable evidence of their glacial movement. A short ways up the trail at Mirror Lake you can cross over the P4 and P5 moraines and find Lookout Lake. This lake was formed by water backing up between the P4/5 moraines and the protalus rampart. The map on page 44 shows Wisconsin till and moraines and Holocene protalus along the headwaters of French Creek, east of the Snowy Range, Wyoming.

The protalus rampart was created at a cooler time when snow fields larger than those found today existed along the range front. This allows the talus falling from the higher elevations to accumulate farther from the slope. When the snow fields receded, the pile of talus remained. The protalus rampart may be related to the continued recession of ice associated with the 5th Pinedale glaciation, the Younger Dryas, or it may also be related to the "Little Ice Age" which lasted from approximately 1400 to 1800 a.d.

Far above you on the top of the Snowy Range is evidence that the ice did not completely cover the mountain. The peaks of the range contain evidence of patterned ground, frost shattered blocks, and stone nets.
Wisconsin till and moraines, and Holocene protalus along the headwaters of French Creek, east of the Snowy Range, Wyoming (modified from Mears, 2001).
Patterned ground below the summit of Medicine Bow Peak. Note the white Medicine Peak Quartzite blocks and the steeply dipping quartzite beds on the nearby peak.
LUNCH STOP: SNOWY RANGE SKI AREA

The Snowy Range Ski Area has a base elevation of 9,000 feet, typically receives 240 inches of snow annually, and is generally open November through April. The ski area was first opened in 1960 as the Nash Fork Ski Area and sometime in the 1980's the name was changed to the Snowy Range Ski Area. Today, Snowy Range has four chairlifts and a "magic carpet" servicing 27 trails with 1,000 feet of vertical relief. The lodge, where we will be dining, was built in 2004, after a fire destroyed the previous lodge.
STOP 5: PINEDALE AND BULL LAKE GLACIAL DEPOSITS (Highway 130, M.P. 28.73)

Pinedale till (bare rocks) lapped up on the older Sacajawea till (left).

The Medicine Bow Mountains contain records of two major times of glaciation. The Precambrian Headquarters Formation contains metamorphic tillites and phyllites, which have been interpreted as ancient glacial deposits. The best-known and most recent glaciation occurred during the Pleistocene ice age roughly 2,600,000 to 12,000 years ago. The Pleistocene glaciation contains numerous glacial episodes that can be grouped into three time periods.

The earliest episode is pre-Illinoian (>160,000 bp) and is evident by a few small discontinuous deposits of extremely weathered till found on the flanks of high mountain valleys. The till is different than either the Bull Lake or Pinedale tills found in the valleys.

A second significant glacial advance occurred in Illinoian times (~135,000 bp) and is known as the Bull Lake glaciation. The Bull Lake glaciation removed most of the earlier pre-Illinoian deposits and extended the farthest into the valley of the three episodes.
Bull Lake till is differentiated from the Pinedale till by having more subdued hummocky surfaces and typically larger rocks contained in the till.

The youngest glacial episode is Wisconsinan in age (~12,000) and is known as the Pinedale glaciation. The Pinedale glacial record contains 5 distinct glacial pulses, each defined by distinct moraines. Being the youngest, the Pinedale glaciation removed much of the previous glacial records and incorporated them into the younger till.

Numerous glacial (meltwater) ponds were formed by the retreating ice of the Pinedale maximum. A good example of one of these kettle ponds is Barber Lake. In at least one location the sediments that collected at the bottom of the ponds are preserved as varves. Each varve can be separated by season, during the summer months the ponds collected sand and small gravel. During the winter months, while the pond was frozen, fine sediment settled to the bottom of the pond.

A short hike from the U.S. Forest Service’s Visitor Center allows for a side-by-side comparison of the Pinedale and Bull Lake tills. The tills incorporate rocks that are found farther up the mountain, which show evidence of glacial transport, including slickensides, chatter marks, and percussion marks.
Illinoian and Wisconsin glacial deposits and features in the vicinity of the mountain front near Centennial, Wyoming Stop #5 (modified from Mears, 2001)
East Bound on I80: As we travel east on I80 to the next stop we will cross the east side of the Laramie Basin. Paleozoic rock units dip west off the Laramie mountains and continue beneath Laramie. As we drive from Laramie to Vedauwoo you can observe rock slopes of interbedded gray marine shale and red wind deposited sandstone that belong to the Pennsylvanian-Permian Casper Formation. These rocks were deposited in and adjacent to a shallow sea that covered Wyoming 290 million years ago (Lageson and Spearing, 1988). The slopes where presplit in 1994-1995 to widen the interstate to three lanes. Rockfall mesh was then placed on the slopes.

STOP 6: LINCOLN MEMORIAL & REST AREA (I-80 M.P. 323.0)

October 31, 1913 - bonfires, fireworks, concerts, parades, dances and speeches in cities and towns from New York City to San Francisco heralded the official opening of the Lincoln Highway, the nation’s first transcontinental automobile road. Spanning nearly 3400 miles, the Lincoln Highway first consisted of a network of existing farm and ranch roads, section line roads, wagon trails, and railroad grade marked only by red, white and blue directional signs to point adventurous travelers in the (hopefully) right direction. Carl Fisher, founder of the Indianapolis Motor Speedway, and the Lincoln Highway Association, conceived the notion of a “Coast to Coast Rock Highway” and enlisted the financial support of automobile enthusiasts and the fledgling automobile industry to bring the idea to fruition. Henry Joy, President of Packard Motor Company, guided the selection of the shortest route between the coasts in the three months preceding the grand opening.

Much of the route consisted of unimproved dirt roads, and cross-country travel was not without challenges. The 1916 Lincoln Highway Guidebook estimated 20 to 30 days (driving 10 hours/day) for the coast-to-coast trip, with a full complement of camping gear, food, water, extra gasoline, oil, water and tires, chains, shovels, and goggles. The mass production of the affordable Ford Model T allowed many Americans to undertake automobile travel, and the Lincoln Highway facilitated the
birth of the great American road trip, auto-tourism and the associated businesses, which continue today. The 1919 U.S. Army military convoy expedition on the Lincoln Highway, which took over 60 days and resulted in the rebuilding and repair of innumerable bridges, proved to have long-reaching consequences: a young Lt. Col. Dwight David Eisenhower first saw the need for a network of high quality interstate roads while on that convoy. Coupled with his observations of the German autobahn during World War II, in 1956 President Eisenhower signed the landmark legislation creating today's Interstate Highway system.

During the early years, the Lincoln Highway Association relied on private donations to fund development. Federal aid highway funds first became available in 1916, but were not widely used until 1921. In Wyoming, the Highway Department was founded in 1917 and began some improvements along the Lincoln Highway with the assistance of federal-aid. The improvements included grading, gravel surfacing and some realignment. By 1925, federal and state officials, in cooperation with the Lincoln Highway Association, began considering a consistent numbering system for the myriad of named highways which had sprung up across the country. In 1926, the states approved the numbering system, which designated much of the Lincoln Highway as U.S. 30, and on September 1, 1928, the Lincoln Highway Association closed its doors and commemorated the route with the placing of 3,000 concrete monuments along the route by groups of Boy Scouts.

STOP 7: VEDAUWOO (I-80 M.P. 329.3)

Turtle Rock at the Vedauwoo Recreational Area.

Vedauwoo is Arapahoe for “earth-born,” which is a fitting name for a disorderly mound of rocks. Vedauwoo represents the geological phenomenon of Earth’s past, created in Precambrian time, 1.4 billion years ago. The granite is part of the Sherman batholith. The 1.43-Ga Sherman batholith of southeastern Wyoming and northeastern Colorado comprises rocks that include Sherman Granite, Lincoln granite, porphyritic granites, and a suite of mafic rocks ranging from gabbro to monzodiorite. The magma that crystallized to form the Sherman Granite at Vedauwoo is similar to the magma found below the geysers and thermal features in Yellowstone National Park, but it is much older.

Vedauwoo’s Sherman Granite is one of the first rock units of Precambrian age to be documented in Wyoming. Geologist Nelson Darton formally named the granite in 1910 during the Hayden Geological Survey. It was named after Union Army General William T. Sherman.

This area also includes the “Gangplank,” a geologic feature that forms a natural land bridge, linking the high prairie to the mountains. The Laramie Mountains were uplifted and over time the overlying sedimentary rocks were eroded, exposing Vedauwoo’s Precambrian granite that weathered along fractures and cracks into rounded to subrounded boulders.
STOP 8: AMES MONUMENT AND THE UNION PACIFIC RAILROAD (Optional stop depending on time)

One of the least expected sites on the windswept plateau of the Laramie Range is the 60 ft tall, granite pyramid known as Ames Monument. The Union Pacific Railroad commissioned the monument in honor of the Ames brothers, Oakes and Oliver, of Massachusetts. The Ames brothers provided significant financial and political support for construction of the first transcontinental railroad. The Ames brothers owned the Ames Shovel Company, invested in railroads, and became significant stockholders in the Union Pacific holding company, Credit Mobilier of America. Both were also involved in politics. Eventually Oliver became president of the Union Pacific and Oakes the president of Credit Mobilier. Oakes had been elected to the U. S. House of Representatives in 1862. In 1867, when Credit Mobilier was about to come under greater Congressional oversight, Oakes began selling Credit Mobilier stock to other congressmen. He was nearly expelled from Congress for these practices. In 1873, public debate resulted in a censure, with no expulsion. Oakes died ten weeks thereafter.

The monument was designed by well-known American architect Henry Hobson Richardson, and the bas-relief medallions of the two brothers on the east and west faces of the pyramid were sculpted by Augustus Saint-Gaudens. Ames monument was completed in 1882 at a cost of about $65,000, standing about 32 ft higher than the original 1868 UP railroad grade to the north. The old railroad town of Sherman sat just to the north of the track. In 1901, UP re-aligned the track to the south of the monument.
Ames Monument is the process of being designated as a National Historic Landmark by the National Park Service due to its significance in both American history and architecture.

Source: Junge, Mark
References


References

Junge, Mark, 1971, National Register of Historic Places nomination form. Wyoming Recreation Commission, Cheyenne, WY.


References


References

A metamorphic facies is a set of metamorphic mineral assemblages that form under similar pressures and temperatures. The assemblage is typical of what is formed in conditions corresponding to an area on the two dimensional graph of temperature vs. pressure. Rocks, which contain certain minerals, can therefore be linked to certain tectonic settings, times and places in geological history of an area. The boundaries between facies (and corresponding areas on the temperature v. pressure graph), are wide, because they are gradational and approximate.
Glossary of Selected Geological Terms from the Guidebook

Almandine: Garnet with iron and aluminum

Amphibolite Facies: An assemblage of rocks which are considered to have formed under similar conditions of temperature and pressure. Amphibolite facies occurs at temperatures greater than 500° C and pressure less than 1.2 Gpa (see page 57)

Andesine: A plagioclase feldspar of intermediate composition having sub-equal amounts of sodium (Na) and Calcium (Ca)

Andesite: A fine grained intermediate volcanic igneous rock characterized by the presence of oligoclase or andesine feldspar

Anticline: A fold in rock in the form of an inverted u. The oldest rock layers are found in the core of an anticline.

Arkosic Sandstone: A sandstone that contains notable quantities of feldspar grains in addition to quartz.

Basalt: A fine grained extrusive igneous rock (lava) containing plagioclase, pyroxene and with or without olivine.

Batholith: Large intrusive mass of igneous rocks, usually of granitic composition.

Bedding: Or beds are tabular or lenticular layers of sedimentary rock that have lithologic, textural, or structural unity that clearly distinguishes them from layers above and below.

Biotite: A black sheet silicate mineral charaterized by a remarkably fine cleavage in one direction

Boxwork: The pattern of limonite left after sulphide minerals containing iron have been oxidized and other cations removed.

Chatter Marks: Is one or, more commonly, a series of wedge shaped marks left by chipping of a bedrock surface by rock fragments carried in the base of a glacier. Marks tend to be crescent-shaped and oriented at right angles to the direction of ice movement.

Conglomerate: A sedimentary rock consisting of subrounded to rounded fragments of gravel, cobbles, and boulders in a finer grained matrix.

Crossbedding: Sediment that make up the internal structure of some beds are deposited at an angle to the bounding surfaces of the bed and are called cross-beds.
Glossary of Selected Geological Terms from the Guidebook

**Dike:** A sheet like body of igneous rock that cuts across the bedding or structural planes of the host rock.

**Diopside:** A mineral of the pyroxene group with a chemical formula of CaMgSi$_2$O$_6$. It is found in igneous rocks and metamorphised impure dolomites.

**Fault:** A fracture in rock along which there has been an observable amount of displacement.

**Glacial till:** Unsorted glacial sediment deposited directly by a glacier.

**Glaucconite:** A green phylosilicate (mica group) mineral. Glaucconite forms under reducing conditions in sediment commonly found in shelf type deposits.

**Gneiss:** A banded rock formed during high grade (temperature and pressure) regional metamorphism.

**Granite:** A coarse grained intrusive igneous rock consisting of quartz and alkali feldspar and very commonly mica.

**Greywacke:** A sandstone that consists of fine to coarse, angular to subangular particles which are mainly rock fragments.

**Hogback:** A long narrow ridge or series of ridges with a steep slopes of nearly equal inclination on both flanks.

**Hornblende:** A mineral of the amphibole group, widespread in metamorphic and igneous rocks.

**Kyanite:** Silicate mineral characteristically found in rocks developed by intense metamorphism of clay rich shales.

**Limonite:** A iron hydroxide mineral that occurs widely as the weathering product of all iron containing minerals.

**Marble:** A metamorphised limestone produced by recrystallization by thermal or regional metamorphism.

**Metamorphism:** The process by which changes are brought about in rocks within the earths crust by heat, pressure, and chemically active fluids (see page 57).

**Metadolomite:** Metamorphised dolomite.

**Mylonite:** A fine grained rock produced by dynamic recrystallization of constituent minerals resulting in a reduction of grain size of the rock. Mylonites form in ductile shear zones and show higher strains than rocks outside the shear zone.

**Outwash Plain:** Glacial sediments deposited by meltwater outwash at the terminus of a glacier.
Glossary of Selected Geological Terms from the Guidebook

**Phyllite**: A metamorphic rock that is coarser grained than slate and finer grained than schists. They are formed by low temperature regional metamorphism (typically greenschist facies).

**Protalus rampart**: An arcute ridge of boulders and other coarse debris marking the downslope edge of an existing or melted snowbank.

**Pyrite**: An iron sulphide mineral (FeS)

**Quartzite**: A metamorphosed sandstone composed predominantly of quartz.

**Rhyolite**: Fine grained to glassy volcanic rock, mineralogically similar to granites.

**Schist**: A regionally metamorphosed rock characterised by a parallel arrangement of the bulk of the constituent minerals.

**Sercite**: A minutely crystalline variety of muscovite.

**Shale**: Is a rock made up chiefly of clay sized particles that possess fissility.

**Shear Zone**: Zone of closely spaced anastomosing brittle faults and crushed rocks near the surface or a zone of ductile faults and associated mylonitic rocks at great depth

**Sillimanite**: Silicate mineral characteristically found in rocks developed by intense metamorphism of clay rich shales

**Slate**: Low grade regionally metamorphosed shale or claystone which has a well developed cleavage.

**Slickensides**: Striated polished stepped surface along a fault or bedding surface that have undergone movement. Slickensides are also formed by the movement of a glacier over rock.

**Stone Nets**: In permafrost the active layer of soil close to the surface of permafrost regions undergoes many seasonal and daily freeze-thaw cycles. The constant change in the volume of water tends to move the coarser particles in the soil to the surface. Further frost heaving arranges the stones and rocks according to their sizes to produce patterned ground. Circular arrangements of the larger rocks are termed stone rings. When stone rings coalesce, they form polygonal stone nets.

**Stromatolite**: Are layered bio-chemical accretionary structures formed in shallow water by the trapping, binding, and cementation of sediment by microbial mats. Stromatolites provide the most ancient records of life on earth and can be 3.5 billion years old.

**Syncline**: A fold in rock in the shape of an u. Younger rocks are found in the core of the syncline.
Glossary of Selected Geological Terms from the Guidebook

**Talus**: The accumulation of rock fragments from mechanical weathering, usually at the foot of cliffs and steep slopes.

**Terminal moraine**: A pile of debris that forms at the terminus of a glacier and marks its maximum advance.

**Terrace**: A terrace consists of a flat or gently sloping geomorphic surface, called a tread that is typically bounded by a steeper ascending slope, which is called a riser.

**Tillites**: A sedimentary rock consisting of consolidated masses of unweathered blocks (large, angular, detached rock bodies) and glacial till (unsorted and unstratified rock material deposited by glacial ice).

**Tuffaceous**: Tuff is fragments (less than 2 mm across) ejected from a volcano. When this material is incorporated in sedimentary rocks the name may be modified as in tuffaceous sandstone.

**Varve**: A layer of sediment deposited in a single year almost always used to describe sediments deposited in glacial meltwater lakes. It consists of a coarser layer deposited in the summer and a finer layer deposited in the winter.
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