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59th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM
SANTA FE, NEW MEXICO
May 6 – 9, 2008

LOCAL ORGANIZING COMMITTEE

Erik Rorem Geobrugg North America, LLC
Doug Bland New Mexico Bureau of Geology & Mineral Resources
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Susan Gallaher New Mexico Department of Transportation
Edward Rector New Mexico Department of Transportation
Thomas Brown New Mexico Department of Transportation
Eric Ruud Geobrugg North America, LLC
Deborah Johnson Geobrugg North America, LLC
Robert Thommen Rotec International, LLC

FIELD TRIP SPEAKERS

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Shari Kelly NM Bureau of Geology & Mineral Resources
Greg Kuyimigan US Forest Service
John Lommler AMEC Earth & Environmental, Inc.

Special Thanks to

Dina Horneffer Kleinfelder, Inc.
Kathy Webber Experient Sales Network (event planner)

Hosted by

New Mexico Department of Transportation
New Mexico Bureau of Geology & Mineral Resources
HIGHWAY GEOLOGY SYMPOSIUM
NATIONAL STEERING
COMMITTEE OFFICERS  2008

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<td>Oklahoma City, OK 73015</td>
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<td>San Luis Obispo, CA 93401</td>
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<td>Montpelier, VT 05633-5001</td>
<td></td>
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<td>336-744-2019</td>
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<td><a href="mailto:thommen@swcp.com">thommen@swcp.com</a></td>
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<td>Sam Thornton</td>
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Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on March 14, 1950, in Richmond, Virginia. Attending the inaugural meeting were representatives from state highway departments (as referred to at the time) from Georgia, South Carolina, North Carolina, Virginia, Kentucky, West Virginia, Maryland and Pennsylvania. In addition, a number of federal agencies and universities were represented. A total of nine technical papers were presented.

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

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

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

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

### List of Highway Geology Symposium Meetings

<table>
<thead>
<tr>
<th>No.</th>
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</table>

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee composed of approximately 20-25 engineering geologist and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contribution to the Symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers, except for the treasurer, may only succeed themselves for one additional term.

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

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

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

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples: in Wyoming (1973), the group viewed landslides in the Big Horn Mountains; Florida's trip (1976) included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generation site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; The Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee...
meeting in 1981 provided stops at several repaired landslides in Appalachia regions of East Tennessee.

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

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


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

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

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

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

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

EMERITUS MEMBERS
OF THE STEERING COMMITTEE

Emeritus Status is granted by the Steering Committee

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John Baldwin
David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
A.C. Dodson*
Walter F. Fredericksen
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David L. Royster*
Bill Sherman
Willard L. Sitz
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Sam Thornton
Berke Thompson*
Burrell Whitlow*
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<tr>
<td>Host Coordinator</td>
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Terrestrial Photogrammetric Models for Virtual Structural Mapping Compared to Traditional Geologic Measurements

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Terrestrial photogrammetry was used to develop three-dimensional computer models on two slopes to demonstrate the potential value of this new technology compared to traditional geologic mapping on part of a six-mile-long interstate highway widening project in southwest Virginia. Rock structure measurements made in small areas (window mapping) were compared to the rock structure orientations made with a system developed by 3G Software & Measurement (virtual mapping). The results compared reasonably well. The software also was used to develop detailed topographic profiles of the slopes. The three-dimensional models were constructed in UTM Zone 17 coordinate space using a handheld GPS receiver and manually adjusting the coordinates for correct point spacing. UTM coordinates of distinctive points on two of the main models were used to georeference the same points on more detailed models between GPS locations. Rock structure orientations made with the software are referenced to the model coordinate space; hence, rock structure values exported to ASCII files can be plotted on maps with GIS technology. The field data for two slopes, 210 m and 300 m long and up to 25 m high, were collected during a 4-hour period without restricting traffic flow. A handheld calibrated digital camera, a range pole with distinctive targets for distance, and a distinctive ground target for north rotation were placed in camera view for each model. Rock structure orientations can be made in places that are dangerous for geologists to work, and the models provide lasting records of slope conditions at the time of investigation.
Geotechnical Risk Concerns on a Rural Roadway after a Landslide

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ABSTRACT

Although classified as a rural major collector, the one-mile stretch of State Route 1003 between Templeton and Mahoning in Armstrong County, PA, only saw daily traffic of 600 to 900 vehicles, with a posted speed limit of 35 mph. This roadway hugged a hillside along the Allegheny River valley roughly one third of the way up a 450-foot slope. However because it served as the primary connector between the communities, an emergency project was declared when a landslide took out the southbound traffic lane in July of 2004.

Numerous alternatives were investigated from various repairs of the landslide to complete relocation of the roadway. Because of other problems with the existing roadway including numerous walls of questionable long-term stability, the decision was made to relocate the entire roadway. The roadway was aligned near the base of the slope adjacent to a “rails-to-trails” railroad right-of-way along the bank of the river. This alignment required a cut/fill typical section, with cut into the bedrock and colluvium between the new roadway and the existing roadway, and with fill on alluvium and colluvium.

In the rock cut area, the risk of rockfall hazard was reduced but not completely avoided during design. Instead of applying Modified Ritchie Ditch criteria, the design was based on 2001 OregonDOT / FHWA publication SPR-3(032). Recognizing the safety concerns and budget constraints on this project, a design was developed that allowed containment of most rock falls in a widened-shoulder catchment area. In the areas of colluvium, steeper than normal slopes were designed using the widened-shoulder catchment area in anticipation that some soil movement would occur. In addition, new roadway embankments were analyzed and designed to avoid failures through the trail into the Allegheny River.

Geotechnical details and special provisions were developed and implemented. During construction, additional safety and maintenance concerns required widening of the catchment area below the rock cut. Soil movements have occurred in the colluvium cuts. However, the new roadway is now open to traffic and performing satisfactorily.
The Ferguson Rockslide, Mariposa County, California

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ABSTRACT

This paper is a case history of the events leading up to the Ferguson Rockslide adjacent State Route 140. The site is in Mariposa County, California just west of Yosemite National Park. In 1999 the authors identified topographic features (head scarp, toe bulge, a talus pile below the slide, etc.) that were interpreted to be associated with a dormant slide at this site. At the end of April 2006 there was a small slide in the talus pile. During May 2006 there were several large rockfalls that originated from the toe bulge. By the end of May 2006 the California Department of Transportation (Caltrans) had placed along the highway 600 feet of temporary movable ring net barriers (74 ft-tons energy rating 14 to 20 feet high) to protect the road. From May 25th to May 31st, 2006, almost continuous rockfalls buried 600 feet of the highway. The rockfalls marked the reactivation of the dormant slide that is now known as the “Ferguson Rockslide”.

Immediately after the rockfall buried the highway there was a concern that if the slide continued to accelerate it could possibly dam the river. Caltrans personnel undertook an intensive slide movement-monitoring program starting in June 2006 to predict and anticipate further slide activity. The monitoring methods included estimating the daily rockfall volumes, measuring the slide movement rate with radar, and measuring the displacement of 54 survey monuments on the slide. This was the first time radar had been used in a highway application to measure slide movement.

Continued rockfalls made it impossible to re-establish the roadway on the existing alignment. In August 2006 two temporary bridges and the abandoned rail grade (Incline Road) on the other side of the river were used to construct a one-way signal-controlled bypass around the slide. Five alternatives are being studied to restore two-way traffic through this area.

The authors mapped the slide and the surrounding area to assist in the design of the highway restoration. The reactivated slide occurred in very hard, fractured metamorphic rock (phyllite and chert). The slide mass remaining on the slope is approximately 650 feet wide, 1000 feet long and is estimated to be 90 feet thick. The failure plane is approximately 200 feet above the highway. As the slide moves forward the toe area produces rockfalls (blocks up to 20 feet long). Approximately 90,000 cubic yards of rockfall has accumulated on the roadway creating a large talus pile extending from the river up to the slide plane. The mapping found evidence of several prehistoric episodes of similar slide activity that demonstrates a characteristic creeping slide movement and not a catastrophic failure that could cause Ferguson Rockslide to dam the river.
INTRODUCTION

The section of State Route 140 winds through the Merced River Canyon at approximately 20 to 30 feet above the river level. The canyon has steep sides (slope angles from 35° to 40°) and there is roughly 1600 feet of relief. The slide is about 11 miles west of Yosemite Valley and 12 miles northeast of Mariposa.

The sequence of events leading up to the reactivation of the Ferguson Rockslide began in 1999. On April 29, 1999 a shallow rotational/translational slide occurred in the slope adjacent to State Route 140. The failure was roughly 150 feet wide and 250 feet long. Slide debris, 5,000 to 10,000 cubic yards, blocked both lanes of the highway. At the head of the slide was a boulder and soil deposit. Numerous rockfalls from the head of slide that made it too hazardous to clean up the roadway. The road was closed for a week to observe rockfall activity. The rockfall activity tapered off at which time the slide scar was scaled and the debris was removed from the roadway.

It was expected that erosion of the head scarp area would produce occasional rockfalls up to 6 feet in diameter. A cable net drapery was recommended to protect the highway from rockfalls. Plans and specification were developed and drapery was installed in the winter of 1999.

At that time it was recognized that the failure occurred in the south side of an old talus deposit that had accumulated below a large dormant prehistoric slide (see Figure 1).

Figure 1. This photograph was taken during the winter of 1999 while the cable net drapery was being installed. Some of the pertinent slide features are labeled.
Another shallow rotational/translational slide occurred in the north side of the old talus deposit on April 29, 2006 (see Figure 2). The failure was roughly 130 feet wide, 275 feet long and approximately 6000 cubic yards in volume. Scaling and draping was recommended similar to what was done in 1999. To protect traffic during the construction a temporary k-rail and chain-link low energy rockfall barrier was erected along the edge of the shoulder. By May 10, 2006 the slide had been scaled and the debris had been removed from the roadway. On that day several large, 3 to 5 foot diameter, rockfalls originated from the toe area of the dormant landslide (see Figure 2) approximately 300 feet above the roadway. The low energy barrier was not sufficient to stop rockfalls of that size and energy. It was decided that the low energy barrier would be replaced with a temporary 14-foot-high ring-net barrier that had a 74 ft-ton energy rating. Part of the material for the new barrier was salvaged from a previous barrier that was stored in a Caltrans maintenance yard over hundred miles away. The salvaged materials were loaded and shipped overnight. The limits of the ring-net barrier extended from the northern edge of 2006 slide to the northern edge of the 1999 drapery a length of 400 feet. The barrier was assembled in segments that were welded onto steel trench plates. The assembly area was on the roadway below the 1999 drapery that had not experienced any rockfall up to that time.

Figure 2. This photograph was taken on May 10 after the rockfalls; some of the pertinent features are labeled.
By May 14 2006 most of the barrier had been assembled and many of the barrier segments had been skidded into place. Before daylight that morning there were a series of rockfalls that originated from the toe area of the dormant landslide above the 1999 drapery and bounced on top of the drapery to the roadway. One of those rocks hit the contractor’s pickup truck. The barrier limits were then extended to the southern edge of the ’99 drapery. The rockfalls that crossed the drapery were observed to bounce higher so the barrier height in front of the drapery was increased to 20 feet. Due to the increased frequency of rockfalls it was also decided to move the barrier to centerline to increase the catchment area behind the barrier. On May 25th, 2006 the barrier was in the place and the highway was opened to one-way traffic guided by a pilot car.

Since April 29th 2006 the highway had been closed except for a few brief periods. During that time there was increasing public pressure to get the roadway opened. This was the start of the tourist season for Yosemite Park and the local economy was heavily dependent on tourist dollars. All along it was assumed that the winter rains had triggered the rockfalls and as soon as the slopes dried out the rockfalls would stop.

Shortly after the roadway was opened on May 25th a large rockfall collapsed one of the barrier posts and the road was closed again. That night and the next day rockfall activity continued to increase. On May 26th 2006 there was an attempt made to drag the barrier out of the way of the rockfalls to repair it. The rockfall activity increased to such a level that it was unsafe to work in the area and the repair/salvage attempt was abandoned.

From May 26th to May 31st 2006 the nearly continuous rockfalls filled the catchment area behind the barrier and eventually overtopped it. The rockfalls created a talus pile that buried a 600-foot-long section of the highway and built out 30 feet into the river (see Figure 3). To this day the barrier stands erect buried within the talus. By this time it was obvious that the previously dormant slide had been reactivated. The locals named it the “Ferguson Slide”.
The nearly continuous rockfalls made it impossible to re-establish the roadway on the existing alignment. Caltrans constructed a detour around the slide by erecting two temporary Bailey bridges and paving the abandoned rail grade (Incline Road) on the other side of the river. A narrow 90-degree turn at the downstream bridge required that traffic be restricted to a maximum vehicle length of 28 feet. The one-way signal-controlled detour was opened in mid-August 2006.

SITE GEOLOGY

The Ferguson Rockslide is located in the west-central portion of the Sierra Nevada Geomorphic Province. Metasedimentary rocks of the Calaveras Complex and Shoo Fly Complex intruded by granitic rock underlie the area. These rocks are fractured and folded; the bedding generally trends to northwest and dips to the northeast. Bateman (1992) mapped this area and part of his geologic map is shown below (Figure 4).
Figure 4. This is a portion of Bateman’s 1992 geologic map that includes the Ferguson Rockslide.

West of the slide the rocks are mapped as the Briceburg phyllite unit of the Calaveras Complex (TRb). This phyllite is a dark, fine-grained metamorphic rock that was inferred to be of Triassic Age. The Hite Cove phyllite and chert unit of the Calaveras Complex (TRh) underlies the slide and is also considered to be Triassic Age. East of the slide the rocks are mapped as the Paleozoic Pilot Ridge quartzite. The Pilot Ridge quartzite is considered part of the Shoo Fly Complex. The Calaveras-Shoo Fly Thrust Fault is a shear zone separating the two complexes. North and east of the slide the Hite Cove and Pilot Ridge rocks are intruded by the Bass Lake tonalite, a medium gray, medium grained granitic rock.

SLOPE STABILITY

Immediately after the road was closed the authors began geologic mapping of the area to assist the design of the highway restoration. Plate 1 is a map of the slide area and Plate 2 is a geologic cross section. The slide occurred in very hard metamorphic rock (phyllite) that has been fractured and folded so that the bedding dip is near vertical. The mapped features fit the classification of a rock block slide (Turner, A.K., and Schuster, R.L., 1996). The slide mass remaining on the slope is approximately 650 feet wide, 1000 feet long (see Plate 1) and is estimated to be 90 feet thick (see Plate 2). Approximately 600,000 cubic yards of slide material remains on the slope. The failure plane is approximately 200 feet above the highway (based on the radar monitoring and geologic mapping) and dips out of the slope at about 30 degrees below horizontal (based on measured joint orientations and site topography). The slide appears to be a translational failure; evidence for that failure mechanism is the large closed depression (a tension feature) in the upper portion of the slide. As the slide moves forward the toe area over-steepens and dilates, producing rockfalls. Other observers arrived at a similar conclusion (Wyllie, 2006). All of the observed rockfalls were masses that were 20 cubic yards or less. Approximately 80,000 to 90,000 cubic yards of rockfall (the talus map unit) has accumulated on the slope below the slide and does not appear to
buttress the slide. The talus is a mixture of rock block sizes, the largest being elongated, angular boulders 20 feet in length.

There appears to have been at least 2 or 3 previous prehistoric periods of slide activity. The vegetation patterns on the scarp and changes in scarp morphology are the evidence of the slide episodes. Talus from the current slide activity buried talus that remained on the slope from previous slide events (see Figure 5). Just downstream of the talus, on the left bank (slide side) of the river, is a bar composed of large angular boulders of metamorphic rock, which is interpreted to be older slide talus that the river has moved downstream. There are no deposits of slide material on the opposite side of the canyon even though conditions are favorable for preservation; it is on the inside bend of the river. On the slope above Incline Road and below it are alluvial deposits of rounded boulders and cobbles. The boulders in the alluvium are smaller than 2 feet in diameter and mostly composed of granitic rock with a minor amount of metamorphic rock.

Figure 5. This photograph shows some of slide features and other deposits at the site.

Based on the topography of the surrounding slopes the initial volume of unstable material (prior to the start of sliding) is estimated the 1,500,000 cubic yards (See Plate 2). The current slide volume is estimated at 600,000 cubic yards of rock on the slope that is only 40% of the original slide mass. The slide has lost 60% of its potential energy and considering the amount potential energy remaining to the slide it is reasonable to assume that future events will involve increasingly smaller volumes of material when compared the amount of rockfall during the 2006 event.
GROUNDWATER

It is assumed that the 2006 slide was caused by a rise in ground water. Wyllie (2006) reported seeps in the slide toe area where it was exposed in the slope face. Wyllie also reported a spring at the highway level that flows throughout the year.

The annual precipitation at Mariposa (Western Regional Climate Center) and the monthly precipitation totals for rainfall years 1995, 2005, 2006 and the average monthly totals are tabulated in the graphs below.

Figures 6 and 7. These graphs show the yearly rainfall totals and some selected monthly rainfall totals measured at Mariposa.

The average annual rainfall at Mariposa is almost 30 inches. The totals for the rainfall years 2005 and 2006 were well above the average. However, other rainfall yearly totals (for example 1982 and 1983) were significantly more.

Prior to the slide reactivation some of the monthly rainfall totals (January and March 2005, December and April 2006) were well above the monthly averages. However, monthly totals for other years (January and March 1995) were even greater than the 2005 or 2006 monthly totals. There is little doubt that surface infiltration of precipitation contributes to the groundwater fluctuations in the slide vicinity. However, it is difficult to draw conclusions on how the rainfall totals contributed to triggering the Ferguson Rockslide.

FERGUSON ROCKSLIDE MOVEMENT MONITORING

During the Memorial Day weekend 2006 there was a concern that if the slide continued to accelerate, it could possibly block the river. Caltrans personnel undertook an intensive slide movement-monitoring program to predict and anticipate future slide activity. The monitoring methods included estimating the daily rockfall volumes, measuring the movement rate of the toe of the slide with radar, and measuring the displacement of 54 survey monuments.

Shortly after the slide started moving it was unsafe to access most of the slide to establish monitoring points. Early on it was recognized that the volume of rockfall was a function of slide movement. Starting on June 2, 2006 Caltrans survey crews made reflectorless surveys of the rockfall
(slide talus) accumulation to estimate its volume. The rockfall accumulation surveys were made weekly. Caltrans geotechnical and on-call consultant staff made daily estimates of the rockfall volumes to augment the survey data. The rockfall accumulation survey and daily rockfall volume estimates were discontinued in September 2006 after the rockfall activity tapered off (see Figure 8) and more direct methods of measuring the slide movement were in place.

![Cumulative Rockfall Volume](image)

**Figure 8. A graph of total rockfall volume verses time.**

The rockfall volume estimates were always considered an interim measure and it was understood that additional monitoring methods would be required. A selection criterion was developed for choosing the replacement monitoring methods. The selected method would have to: provide a direct measurement of the slide movement; be remote (it was unsafe to place instruments on the slide); be automated (little if any human input would be required); could collect data continuously (24 hours/day, 7 days/week); and the data could be remotely accessed by telemetry. A method that used radar to measure slope movement was selected. Fifty-four survey monuments were measured monthly to augment the radar data.

The radar monitoring method is called Slope Stability Radar (SSR); it was developed by Ground Probe Inc. and had been used previously in open pit mines to measure slope movement. This was the first time it had been used in a highway application to measure slide movement. The SSR unit contains an onboard computer and software that controls data collection and processing (in other words it is automated). The SSR scanned the toe and talus area of the slide approximately every 15 minutes. The SSR is capable of detecting slope movements as small as 0.08 inches. Some of the advantages of using radar are that it is not affected by darkness, dust, fog, or rain. The major disadvantage of SSR is that it can only collect data from bare rocky surfaces and does not work on slopes covered with vegetation. The SSR unit was installed July 10, 2006 and monitored the movement of the bare rocky toe area of the slide until the end of June 2007. Figure 9 below shows the cumulative average movement of the toe area during that period.

Note that the graph shows a steady decrease in the movement with time. The data was collected and emailed out twice daily by a Ground Probe technician. The SSR was discontinued because slide movement has slowed significantly and the measurements from the survey monuments are a suitable alternative.
Comparing the rockfall volumes surveys and the SSR measurements it is estimated that during the first few days the slide was moving at 80 inches per day. That movement rate is considered slow to moderate (Turner, A.K., and Schuster, R.L., 1996).

![SSR Measured Movement](image)

**Figure 9.** The multi-colored image on the left is a SSR graphic where color represents differing rates of movement; the lighter the color the smaller the movement. The graph on the right is a compilation of the radar data collected from the movement of the Ferguson Slide.

In early August 2006 the slide activity decreased to a point where it became safe to access the upper slide area and 54 survey monuments were established there. The survey monuments allow the measurement of the movement in the upper slide area, where the SSR did not work. Figure 10 below shows the cumulative average movement of the survey monuments during that time period. The current average movement rate is less 1/3 of inch per day. The survey monitoring will continue on a monthly basis until the permanent restoration of the roadway is completed.

![Average Cumulative vs Monthly Displacement](image)

**Figure 10.** A compilation of the survey data collected from the movement of the Ferguson Slide.

**PERMANENT RESTORATION ALTERNATIVES**

Five alternatives are being studied to restore two-way traffic through this area, see Figure 11. The following is a listing of the alternatives in no particular order.

1. Alternative E would remove the slide and stabilize the slope above the existing highway.
2. Alternative R would construct a rock shed to protect the existing roadway from rockfalls generated by the slide.
3. Alternative S would bypass the slide with two bridges and a viaduct that hugs the slope to minimize cuts and fills.
4. Alternative C would bypass the slide with two bridges connected by a cut through the hillside.
5. Alternative T would bypass the slide with two bridges connected by a tunnel.
Figure 11. This is a plan view of the five alternatives currently being considered for the Permanent Restoration of Route 140 in the vicinity of the Ferguson Slide.

CONCLUSIONS

The authors have concluded that there is an extremely low probability that the Ferguson Rockslide will fail catastrophically and in one rapid motion dam the Merced River and bury Incline Road. That conclusion is based on evidence derived from mapping the slide and surrounding terrain. There is evidence that there were a minimum of two other previous prehistoric episodes of slide movement. This is based on the head scarp morphology and vegetation patterns on the scarp. The time between slide episodes is assumed to be on the order of thousands of years. There is no evidence that slide debris from previous events was deposited on the Incline Road side of the river even though the conditions are favorable for preserving such a deposit. Across from the slide Incline Road is on the inside bend of the river. At this location both above Incline Road and in the river channel below the road are cobble and boulder deposits. The cobbles and boulders are rounded, less than 2 feet in diameter, and are composed mostly of granitic rock with a minor amount of metamorphic rock. Slide debris is composed entirely of elongated (up to 20 feet), angular, metamorphic boulders. Some of the debris from previous slide episodes appears to have been dragged down-river by high flows producing a bar composed almost entirely of large, angular metamorphic boulders that stretches at least a ¼ of a mile downstream on the slide side of the river.

The findings of this study conclude that the slide moves at a slow to moderate rate as relatively intact blocks of rock in a translational motion. The authors expect that future movement episodes on the Ferguson Rockslide will be smaller than the 2006 episode. This is due to the loss of potential energy during each slide episode. Slide episodes will add to the existing talus pile further aggrading into the river channel gradually narrowing the channel and forcing flows toward Incline Road and gradually raising river levels.
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Potentially Acid Producing Rock Encapsulation and Permitting - Case Study for SR-28 (US-127) Landslide Project in Fentress County, Tennessee

By: Vanessa Bateman, P.G., P.E., Samuel Williams, P.G., Jim Ozment, P.G. and Leonard Oliver, P.E.
Potentially Acid Producing Rock Encapsulation and Permitting - Case Study for SR-28 (US-127) Landslide Project in Fentress County, Tennessee

By: Vanessa Bateman, P.G., P.E., Samuel Williams, P.G., Jim Ozment, P.G. and Leonard Oliver, P.E.

ABSTRACT

Old coal strip-mining operations in the Upper Cumberland area of Tennessee are known sources of acid drainage which have impaired streams and have negatively affected the environment in Tennessee. A growing awareness among environmental activists and citizens to the potential consequences to potentially acid producing rock have lead to an increased sensitivity and scrutiny of any projects which may disturb or excavate rock formations that can lead to acidic drainage. An awareness of this issue, partially fostered by problems with the Federal Cherahola Skyway project in eastern Tennessee lead the Tennessee Department of Transportation to adopt guidelines for treating and mitigating potentially acid producing rock on project in the early 1990’s. These guidelines were recently revised, but old and new guidelines included encapsulation of potentially acid producing rocks in highway embankments. This is an approach that has proved successful on a number of TDOT projects and has prevented acidic drainage caused by roadway construction on TDOT projects.

The State Route 28 (US 127) widening project where this encapsulation method was recently used was started as a project because of a landslide which required constant maintenance. The affected section of roadway has required regular (sometimes daily) maintenance in order to maintain traffic since the initial slide. This project was delayed by several years due to permitting issues, questions and hearings partially because of the potentially acid producing shale (from the Fentress Formation) that would be disturbed by the roadway construction. The encapsulation details were revised based on input from citizens, environmental advocacy organizations and the Tennessee Division of Environment and Conservation’s Water Pollution Control Division. Construction of the project began in Fall of 2007 and will be complete in 2008.

Context of the Project

State Route 28, located in Fentress County is in the Upper Cumberland Area of Tennessee. Primarily a rural area of the state with a population of 16,625 a per-capita income of $ 21,587 per year, with approximately 23% below the poverty line (US Census, 2008). While much of Tennessee strongly benefitted economically from the development of the Interstate System, Fentress County did not develop at the same pace as the larger metropolitan areas of Tennessee. US-127 (SR-28) is the main US Route connecting Jamestown, the county seat with Interstate I-40. It is also the main route for a large quarry operation located along SR-28 north of Jamestown and is on one of the main bus routes for the local schools located inside Jamestown.
The county has a number of natural and tourist attractions including Sgt. Alvin C. York homesite, Big South Fork National River and Recreation Area, Pickett State Park and cave sites such as Wolf River Cave.

Total relief of the local terrain along this section of the SR-28 corridor is approximately 600 feet. SR-28 travels down an erosional valley cutting a section of the Cumberland Plateau from the Pennsylvanian Sandstones and shale down to the base of St. Louis Limestone. Geological issues in the area include coal seams, landslides, potentially acid producing rock as well as sinkholes, caves and karst.

**History of the Landslide**

A 1 mile segment of SR-28 has been a continual problem for Maintenance forces for over 30 years. This segment has experienced many small and some larger scale slides. The roadway was located on a side-hill fill on top of a colluvial deposit and the Pennington Formation in relatively steep terrain. Over the years a number of corrective actions have been attempted at the site including a rock buttress, continual re-pavement and moving the roadway into the hillside. None of which proved successful in stopping the movement of the road.

Maintaining traffic on the roadway was judged to be critical because there is no realistic detour route. All repair options attempted at the site in the past could not interrupt traffic for any significant period of time. It was this criteria that doomed the buttress repair - as the failure was too deep seated to create a buttress on solid enough ground. Paving, of course, added additional weight to the slide and further increased the driving force. Moving the roadway into the hillside by a traffic lane width proved to be somewhat helpful, but over all was ineffective.

In May of 2003 there was an overnight drop of 8-10 inches and the slide began to accelerate. Eventually obliterating the guardrail and much of the asphalt, at the time of geotechnical exploration there was still 5 feet of asphalt at the road level (Florence and Hutcheson, 2004).
Figure 2. Photographs of Site taken of Slide Progress a) shows formation of crack in roadway, b) accelerated movement has made re-pavement impossible and road was maintained with recycled asphalt gravel. You can see three separate lines of traffic posts that had been placed at roadway level to warn of the shoulder edge.

A preliminary and the full scale geotechnical investigation then took place. In 2003 TDOT faced the following alternatives:
1. Do nothing and attempt to maintain traffic - this approach had worked for a time, but was now untenable.
2. Close the road and construct a massive buttress - again an untenable approach as the detour around this area was far too long and this road is one of the main routes into Jamestown for school busses. It also connects a large quarry to the North of the Site.
3. Attempt some sort of in-place slope reinforcement - this was judged to be unlikely to be successful at the site due the amount and depth of movement. Pennington landslide problems tend to be both complex and deep owing to thick (and irregular) deposits of a clay shale that weathers readily.
4. Re-align portions of SR-28 to avoid the site and locate the roadway on more stable ground. The new roadway would still have to cut through the Pennington Formation, but this could be done in a more stable location.

The Fentress County commission, meanwhile passed a resolution detailing the safety concerns and need for roadway repair at the site. They asked the department to take immediate action on the slide due to the safety concerns at the site (Fentress, 2003).
**TDOT Selects Re-alignment**

TDOT chose a re-alignment project in order to provide relief and started the design process on an emergency basis. The geotechnical investigation for the project was completed in early 2004 and the project was let for contract on April 28, 2006 for a 1.66 mile re-alignment of SR-28 at a cost of $16.4 million. TDOT Maintenance was to maintain traffic on the slide area during construction and traffic was then to be shifted to the new alignment.

**The “Pyrite” Problem Surfaces**

However, there were several problems that needed to be addressed during construction - the most notable and problematic was the potentially acid producing rock (APR) that would be excavated during construction. The Fentress Formation was known to have some potential for acid producing rock problems and contained some pyritic minerals which needed to be handled properly.

Testing on site during exploration revealed a acid based accounting net neutralization potential (NNP) of problematic material between 0 to -5, with some samples testing as high as -50. Many other samples had positive NNP’s indicating that the rock had no potential for producing acid runoff. Acid based accounting was done according to EPA 1994 recommendations with NNP values measuring tons of CaCO$_3$ per 1000 tons of material required to bring this material up to an NNP of at zero. Negative numbers indicate amounts of CaCO$_3$ needed, while positive numbers indicate and excess of CaCO$_3$. TDOT special provisions and guidance from 1990 (written primarily by Harry Moore of TDOT, TDOT 1990) specified that material between 0 to -5 be blended with agricultural lime and placed in highway embankments but that rock with NNP results less than -5 (that is more negative) be encapsulated. This was also recommended in the FHWA manual published near the time TDOT’s special provision was created (Byerly, 1990). It was estimated that approximately 6% of the material to be excavated in the problem cut interval would need to be encapsulated (Florence and Hutcheson, 2004).

**General Geology of the Site**

The geology of the site is fairly typical for an erosional valley in the Upper Cumberland area with steep hilly terrane. The hilltops are bluffs of Pennsylvanian Sandstone and Conglomerate, the existing and proposed roadway then cut down through the relatively level beds (dipping at
Figure 4. Geologic Section of Project area

approximately 10 degrees) through the Pennington Formation (a known landslide former) and into the Mississippian limestones. At the base of the section in the area is the St. Genevieve and the St. Louis limestones. Karst problems including sinkholes can be expected in this zone. The re-alignment project does not pass all the way down section, ending at the top of the Hartselle Formation. Further down section and to the north of the re-alignment project is Wolf River Cave. A large cave managed by the “Wolf River Cave Management Committee” of the Southeast Cave Conservancy. This wild cave is home to a threatened bat species and also has archeological significance.

Permits, Environmental Groups and Competing Stakeholders
As was somewhat typical at the time, final permits had not been acquired for the project prior to letting. A policy change at TDOT specified that all permits were to be obtained by the time of letting. However, this policy was relatively recent and this project, due to the landslide on site was set on the “fast-track” in order to provide relief to the citizens of Jamestown and Fentress County. The project was submitted for final permits in April of 2006. This submittal however, was not to be approved until May 29, 2007 after several public hearings, controversy and meetings with area environmental groups and other stakeholders.
There were several different vocal constituencies with opinions on the project. First there was the Fentress County Commission representing the county residents which supported the road project. Many of the citizens of Fentress County who attended public meetings seemed strongly supportive of the project, and angry that construction had not already begun. Several citizens commented at the TDEC hearing (January 2007) that the existing road was unsafe, that school busses had to regularly drive over the slide and that the new road was needed.

The second group of stakeholders were area environmental organizations. However, the two primary groups did not always have the same agenda. The southeast cave conservancy stated that their concern was the protection of Wolf River cave (located more than a mile from the project). The other group, SOCM (Save our Cumberland Mountains) was primarily interested in the “pyrite” encapsulation. This group originally formed as a coalfield citizen organization has shown strong interest in potentially acid producing rock and handling plans on TDOT projects. Old coal strip mining operations in the Cumberland Mountains area of Tennessee has resulted in acid runoff and impaired streams, so there is some particular sensitivity to APR issues.

The third main stakeholder in the project were the main permitting organizations - primarily the Tennessee Department of Conservation (TDEC) Water Pollution Control as well as the Tennessee Wildlife Resources Agency (TWRA).

Finally there was the contractor, Wright Construction who had bid the project and then suffered construction delays due to lack of permits. Though the project was let in April of 2006, construction did not begin until Fall 2007.

**The “Pyrite” Problem and Permit Delays**

The plan for pyrite encapsulation was to place all of the material in an abandoned quarry (located in the Bangor Limestone) that was along the existing alignment. The submitted plan included alternates for use of a geomembrane or use of a clay layer. The contractor bid using the clay alternate. Graded solid rock was to be placed at the base of the quarry, followed by a layer of agricultural lime. Then potential APR material would be brought in, mixed with lime and finally capped with clay. This is the same basic design set forth in TDOT’s 1990 special provision and has been used successfully on several TDOT projects. This was submitted to TDEC - Water Pollution control and was posted for public comment. Comments were sent in by individual members of the public as well as by representatives from various environmental organizations including SOCM and the Southeast Cave Conservancy.

As public comments were received that were not favorable to the project, TDEC delayed issuing the permits. A meeting was held at TDOT in November of 2006 with representatives from might pass through the clay liner. TDOT and TDEC agreed that these points would be monitored as would any runoff during construction. A retention pond was designed to hold any runoff from the pyrite encapsulation cell was it was being constructed and this too was to receive water quality monitoring. Based on these agreements, permits were issued for the encapsulation.
Other water quality concerns were settled during the same time and permits were finally issued on May 28, 2007.

Figure 7. View of Quarry Wall with small opening and overview of abandoned Quarry before construction.

Construction Begins
The base of the quarry was cleaned out at prepared for the graded solid rock. A five foot layer of graded solid rock was then placed at the base of the quarry, followed by a layer of type IV geotextile and 6 foot of compacted clay. A careful monitoring program was designed for construction with onsite excavated rock tested to determine if it needs to be placed in the encapsulation, if it can be blended or if it is suitable for common fill. A TDOT Geologist is regularly on site to assist with these duties and to monitor the construction.

Figure 8. View of the Quarry a) during preparation of the pyrite encapsulation cell and b) during placement of clay liner at base. Notice the Geotextile raised above the level of the clay, a 10 foot layer of graded solid rock was placed against the quarry wall and the geotextile is used as a separator between the clay and rock.
So far construction is proceeding well and there have been no environmental problems with the encapsulation cell. No water quality testing in the area has revealed any problems and we expect the cell to be completed later this year.

**Figure 9.** Placement of Fentress Formation Rock mixed with Lime inside the encapsulation cell

**Fentress Formation Geology**

One surprising finding during construction, however involved the actual distribution of the pyritic minerals within the Fentress Formation itself. Testing of samples from rock core is very useful for determining potential APR risk. However, core samples do not always provide the same illumination as does large scale excavation of a formation. It was apparent from the core and surrounding geological reconnaissance, that there were small coal layers in the Fentress Shale and that the pyritic minerals occured somewhat irregularly in lenses. What was not clear, was the nature of the distribution of the pyritic minerals in this overall CaCO$_3$ containing shale. Instead of occurring in distinct, if somewhat geographically limited layers, the pyrite is crusted on and in on chert nodules that are distributed irregularly in the formation. The shale, with some limited exceptions (along small coal seams), appears to have positive NNP, it is these nodules of chert, crusted with pyrite (and to a lesser extent marcasite) which are apparently the source of the negative NNP values. This has some implications for both our testing program and policy decision to encapsulate.
The matrix of rock being placed inside the encapsulation cell is at least partially material that has positive NNP - that is this material has neutralization potential. When then mixed with agricultural lime, it should mean that the material placed in the cell should have even less ability to mobilize potential acidity than designed. It adds a level of conservatism to the design that was not originally anticipated.

Acid production and runoff occurs when pyritic minerals are exposed to air, water and have a greatly increased surface area. Excavation of the Fentress Formation is certainly increasing the surface area exposed, but not to the same extent as would be if the pyrite were more distributed throughout the shale. We plan to run some further testing to see if this situation makes any difference to the actual acidic runoff that should be expected during excavation of the Fentress Formation and to see if this has relevance to other Tennessee formations.

CONCLUSIONS
Final conclusions for the project will not be made until the project is complete and monitoring has occurred over time. We will want to assess the success of the encapsulation after construction as well as ensure than the goal of a safe roadway was achieved. Getting to this point has been an educational process for all involved. As a result of some of the experience in addressing public concerns for pyrite encapsulation, TDOT has updated its 1990 special provisions. Many of the recommendations of that project team, including members of TDOT, TDEC and staff from Golder Associates were actually implemented on this project - even though many of the provisions of the new policy were not in place at the time of design and meetings for this project.

Construction of a new project is not as simple as it may have been in the past. Environmental concerns are much more in the forefront of the public mind. The public landscape is different and citizens and citizen groups expect to have input into the process. This can cause significant project delays if time for this commentary is not part of the process.

Figure 9. Chert Nodules in the Fentress Formation
Another issue to be managed is that the public and stakeholders do not always agree on the value or even the goals of a project. The SR-28 project illustrates this well as there were several environmental groups content to have significant delays in project construction until their concerns were addressed and yet there were many citizens in Fentress County who were quite willing to brush those issues aside so that the road could be built. Still others felt the issues were being addressed and wanted to see the focus on road safety.

Projects built for economic development and non-emergency purposes can build in much more time for public comment and contribution. The CSS (Context Sensitive Solution) process is being implemented within Tennessee to better include public comment and contribution to transportation projects. However, it can be very difficult to incorporate all of the provisions of CSS on a project which faces significant time constraints due to safety concerns. Many members of the public can become very frustrated with extended periods of comments when they perceive an emergency need. Balancing all of these stakeholders has to be done on a project by project basis - and any manager needs to be aware that it may be impossible to satisfy everyone who is interested in the project.

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STABILIZATION OF THE TEMPLIN HIGHWAY LANDSLIDE
INTERSTATE 5, LOS ANGELES COUNTY, CALIFORNIA

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ABSTRACT

Interstate 5 is California’s principal north-south highway. Construction of the highway in the 1960s involved excavation of a cut through the hills near the Templin Highway overpass. Since completion of construction, the site has been subject to shallow cut-slope failures and roadbed heaving. Development of a scarp in the hills adjacent to the highway in 2000 led to the recognition of a deep-seated landslide manifesting as a pressure ridge across all lanes of the highway.

From 2000 through 2005, the California Department of Transportation (Caltrans) drilled numerous exploration borings at the site to investigate the subsurface conditions. Many of the borings were completed as slope inclinometers. The results of the investigation suggested that shear movement was occurring subparallel to the axis of the northerly plunging Ridge Valley Syncline towards the free space created by the highway cut.

During the record rainfall of the 2004-2005 season, the landslide’s movement abruptly accelerated, resulting in distress that nearly caused the closure of Interstate 5. In an emergency action and based on the subsurface investigation, Caltrans proceeded with a remedial grading design that included excavation of over 1.2 million cubic meters of earth materials to reduce the slide’s driving forces.

Having full-time observation by geologists during remedial grading operations resulted in the development of detailed geologic mapping that enabled verification of the original geologic model used in the analyses. Full time observation and mapping by geologists also allowed some modifications to the remedial design that yielded significant savings in project cost and time.

INTRODUCTION

The Templin Highway landslide is located along California’s main north-south highway, Interstate 5, at the Templin Highway overcrossing, north of the Castaic area of Los Angeles County, California (Figure 1). The site is approximately 44 miles north-northwest of downtown Los Angeles.
This paper presents a summary of the history of the landslide, the geotechnical investigations, analysis and mitigation design work performed by the California Department of Transportation (Caltrans) and a summary of the construction phase geotechnical services provided by URS Corporation (URS) as a consultant to Caltrans.

BACKGROUND

Interstate 5 was constructed in the late 1960s. Construction of the highway included the excavation of a through-cut which created 1:1.5 (V:H) cut slopes on the east and west sides of the highway. This location had been the site of a series of escalating cut slope and roadbed problems since completion of highway construction. These problems included numerous surficial cut-slope failures, debris flows and heaving of the roadway pavement (Figure 2). A buttress was constructed on the 34.5-meter-high cut slope along the east side of the through cut in the 1960s to arrest a small slide that developed during the construction of the highway. This buttress remains in good condition. However, the majority of the westerly approximately 62-meter-high cut slope deteriorated in the years following construction. A rockslide occurred in 1980 following heavy rains (Foster, 2003). Although the slope was regraded in 1983, the slope condition continued to deteriorate and by 1993 tension cracks had opened on the slope and adjacent areas. The movement broke the concrete-lined drainage ditches, allowing concentrated water runoff to enter the tension cracks and bedrock joints thereby worsening the condition of the slope.

The development of a well-defined scarp west of the westerly cut slope and development of bulges and ridges that crossed all lanes of the highway in 2000 led to a change in paradigm regarding the origin and nature of the recurrent cut slope failures as well as the cause and nature of the pavement heaving along the roadbed----the recognition of the presence of an incipient/emergent, deep-seated landslide, the toe of which appeared to be manifesting as a pressure ridge across all lanes of Interstate 5. The landslide exhibited evidence of increasing instability with time. Slope inclinometers installed by Caltrans yielded data that indicated that the westerly cut slope and the area to the west was moving toward the free space created by the roadway through cut.

A series of sustained storms brought record rainfall to southern California during October of 2004. Slope inclinometer data revealed that the slide mass accelerated during and following
these rains. By November 2004, the shear rate at one instrument was observed to be 46 mm/year with similar rates observed at other instruments. During December of 2004 and January of 2005, the slide’s rate of movement abruptly accelerated. During eight days of heavy, persistent rains on and following January 2, 2005, the slide sheared all of the slope inclinometers installed during the winter and spring of 2004. During and after the storms, artesian springs were observed issuing from grade, slopes and benches located within and adjacent to the slide mass. Ground bulges, ripples and hummocks developed across all lanes of the roadway. Mud and debris flows issued from portions of the slope and pushed concrete K-rail located along the west shoulder into the southbound lanes of the highway. During January 2004 and the first part of February 2005, newly installed slope inclinometers survived less than a week before movement of the slide sheared them off. This implied shear velocities of 2.2 m/year to 5.6 m/year, or greater. The degree of distress was such that the continued full operation of Interstate 5 was threatened.

As an emergency project, Caltrans developed a long-term mitigation design that involved the excavation of over 1.2 million cubic meters of landslide mass to decrease the amount of driving force on the slide. Caltrans completed the remedial grading plans in November 2005. A commercial grading contractor was selected and remedial grading operations began in March 2006. URS Corporation was contracted as a consultant to provide geotechnical support for Caltrans.

SITE TOPOGRAPHY

The site and vicinity exhibits high relief with rugged hills and mountains. The ridge line located at the west side of the project includes Townsend Peak (Figure 2). The summit of Townsend Peak stands approximately 200 m above the highway grade at an elevation of 970 m. Below and to the east of Townsend Peak lies a relatively flat plain which became known as the “Duck Pond” area due to the presence of seasonally very shallow groundwater. To the south of the Duck Pond lies Violin Canyon, a portion of which was filled during the construction of Interstate 5 in the late 1960s. This canyon is referred to as the “Disposal Canyon” in this report and as shown of Figure 2. Total vertical relief across the project site is approximately 300 m.

REGIONAL GEOLOGY

The site is located within the Transverse Ranges Geomorphic Province which is characterized by a series of east-west trending mountain ranges separated by valleys and sub-parallel fault zones.
The Transverse Ranges are the manifestation of a compressional tectonic environment resulting from the “Big Bend” in the San Andreas fault. The area of the subject site is a part of a geologic structure known as the Ridge Basin Syncline, which is a northwesterly trending regional fold developed in Miocene age bedrock (Crowell & Link, 1982).

**SITE GEOLOGY**

The axis of the Ridge Basin Syncline trends northwesterly through the site, subparallel to Interstate 5 (Figure 2). The axis plunges 21 degrees to 27 degrees to the north. At Townsend Peak (on the westerly limb of the syncline), the geologic structure becomes very complex. Bedding becomes contorted, faulted and locally overturned. West of Townsend Peak, the local formations are truncated by the San Gabriel fault, which trends northwest.

The site is predominantly underlain by Miocene age bedrock comprised of the Ridge Basin Group (Crowell & Link, 1982). Rock units encountered at the site include the Paradise Ranch Shale Member of the Peace Valley Formation (Mppr on Figure 2) and the Marple Canyon Sandstone Member of the Ridge Route Formation (Mrm on Figure 2). The Paradise Ranch Shale is comprised of thinly bedded to laminated claystone, mudstone, siltstone, shale and minor sandstone. At the site, the Paradise Ranch Shale is considered a generally weak, incompetent rock mass.

The Marple Canyon Sandstone underlies the Paradise Ranch Shale. The Marple Canyon Sandstone is comprised of thin to thick beds of poorly to well cemented sandstone interbedded with thin beds of conglomerate, mudstone and shale which form a generally competent rock at the site.

**GEOTECHNICAL INVESTIGATION**

**Geotechnical Borings**

From 2000 through 2005, Caltrans drilled numerous vertical borings at the site which were subsequently developed as slope inclinometers, piezometers, 200mm dewatering wells and 610mm mm pilot shafts which facilitated the development of large diameter vertical watering...
wells. The boring depths varied with location from about 15m to 46m at highway grade up to 123m in the upper portions of the landslide. Borings were developed using mud-rotary coring and roller bit as well as bucket auger drilling methods. Cores were recovered, logged and boxed for most of the core borings.

**Geologic Mapping**

The regional Geologic Map of Ridge Basin (Crowel & Link, 1982) was used by to identify formations and general regional geologic structure in the vicinity of the project site. Detailed geologic mapping of the highway alignment, landslide and surrounding areas was performed by Caltrans geologists.

**Geophysical Studies**

Down-hole geophysical logging was completed on four of the borings by Caltrans. The primary reason for using bore-hole geophysical logging was to determine strike and dip of discontinuities and to locate fracture/shear zones. Ancillary benefits of geophysical logging were the ability to evaluate rock quality, lithology and to discern zones producing groundwater. Bore-hole geophysical methods used included:

- The acoustic televiewer (AT) to determine strike and dip of discontinuities (which includes bedding) and to locate cavities and fracture zones.
- The sonic caliper to measure bore-hole diameter and to identify enlarged zones.
- Gamma ray logging to discern shale and clay beds from siltstones and sandstones and to locate shear zones.
- Conductivity logging to facilitate discernment of claystones, shales, clay and shear zones.
- Full wave sonic logging to provide estimates of rock quality and porosity.

The geophysical data were correlated to the boring logs and subsequently plotted on stereonets and rose diagrams to aid in determining the subsurface structure.

**Standardized Materials Testing**
Standardized field and laboratory testing was performed on samples obtained from borings, outcrops and trenches. Some tests were conducted in the field, others in office environments and others by laboratories certified to conduct ASHTO, ASTM and CTM tests.

**Instrumentation**

The site was instrumented by Caltrans with slope inclinometers, time domain reflectometer cable, piezometers and rain gauges. Due to shearing from slide movement, multiple generations of instrumentation have been installed since 2000.

- **Slope Inclinometers:** Slope Inclinometers (SI) are comprised of precision manufactured casing, an inclinometer probe, a control cable and a readout box. Changes in casing inclination can be the result of shear, compression, rotation or translation. Changes in casing attitudes are plotted (both incrementally and cumulatively) versus depth. Interpretation of data plots allows the depths to shear zones (including the slide’s basal shear plane) to be determined and displacement versus time (slide velocity).

- **Time Domain Reflectometry:** Time Domain reflectometry (TDR) is a method of checking cable integrity. In TDR, an electrical impulse is imparted to a coaxial cable. When the impulse encounters a bend, damaged spot in the cable or a break, the impulse generates a reflection that can determine the distance to the damage or deformation location by instrumentation. TDR cables were installed in some of the slope inclinometer and piezometer casings. If slope inclinometers are rendered unreadable due to casing deformation, the TDR cable facilitates detecting shear zones located below elevations where an SI became unreadable.

- **Piezometers:** Both stand-pipe and vibrating wire piezometers were installed at the site during each of the several investigational periods for the purpose of monitoring groundwater levels. Piezometers installed during 2001-2002 were fitted with coaxial cables to facilitate time domain reflectometry readings after they became sheared by slide movement.

- **Rain Gauges:** In January 2005, three rain gauges were deployed across the site.

**CHARACTERIZATION OF SUBSURFACE CONDITIONS**

The principal structure influencing the stability of the site is the northwest-plunging Ridge Basin Syncline (Figure 2). Based on field observations prior and during remedial grading, the majority of the landslide movement appears to have occurred on a tectonic shear that is principally bedding planar within the syncline. During grading, the basal shear was exposed in a cut slope excavated on the easterly flank of Townsend Peak as a fault that cuts across the complex geologic structure in that area. To the south, the fault transitions to a bedding plane shear within...
the synclinal fold. The basal shear extends along bedding to the east across the syncline’s axis and east limb. The trace of the basal shear was observed to be in direct alignment with the southernmost pressure ridge or bulge that traverses the distressed pavement area of the highway. With the exception of the easterly flank of Townsend Peak where the basal shear cuts across complexly folded and faulted Mppr and Mrm, the shear occurs completely within the incompetent mudstone, siltstone and shale of the Mppr, stratigraphically just above the competent sandstone of the Mrm.

Tectonic deformation along this fault/bedding plane shear apparently created a well-developed gouge zone, thus creating a preexisting plane of weakness that subsequently formed the basal shear surface of the landslide. As a combined result of the removal of support created by the excavation of the highway through-cut and an elevated groundwater table, landslide movement occurred on the basal shear surface. Interpretation of the subsurface conditions is shown on As-built Geologic Cross Section A-A’, Figure 3. The approximate location of the section is shown on Figure 2.

Based on Caltrans’ slope inclinometer data and analysis (Caltrans, 2005), the majority of the shear movement in the southwesterly portion of the landslide (in the vicinity of the easterly flank of Townsend Peak and the former “Duck Pond” area) was northeasterly toward the free space created by the highway cut. Shear movement was translated to the north in the easterly portion of the landslide near and beneath the highway. Caltrans concluded that the landslide was being deflected northward by restraining sandstone beds of the underlying Marple Canyon Sandstone member (Mrm). Based on data from boring logs, slope inclinometer and other geophysical data, site observations, etc., Caltrans interpreted discrete shear zones above the basal shear as intraslides.

The rate of movement of the landslide correlated closely to the groundwater levels observed within stand-pipe and vibrating wire piezometers installed and monitored by Caltrans at the site since 2000/2001. The higher the groundwater level, the greater the rate of movement of the slide. Comparison with rainfall records from 2001 to 2005 showed that groundwater levels in the piezometers rose with rainfall; however, it did not lower significantly during the following dryer seasons. Data also show that some sharp, negative spikes in groundwater elevation periodically occur. These spikes occur during periods of increased slide activity and are
interpreted to be the creation of void space (fractures and fissures) as a result of slide movement. However, the former piezometric elevation becomes rapidly reestablished. These rapid recoveries are not always associated with rainfall events. This suggests that recharge, at least in part, is originating from sources outside of the slide mass.

**EMERGENCY LANDSLIDE STABILIZATION**

**Increase in Landslide Movement**

Starting in October 2004, site piezometers revealed that the groundwater elevation was rising fast in response to the above normal rainfall events. By January 10, 2005 water levels observed in piezometers varied from at the at the ground surface on the roadway to 12 meters below the ground surface near the headscarp of the slide located to the west of the highway. These levels were drastically higher than any previously observed levels. By this date, the increased movement on the landslide sheared off all the slope inclinometers installed in 2004. Due to the increased rate of movement of the slide in January of 2005, an emergency dewatering contract was initiated. Initially, Caltrans drilling crews developed two vertical dewatering wells in the central portion of the slide. Subsequently, a contractor was charged with developing additional vertical and horizontal dewatering wells (horizontal drains or hydraugers). Draw-down tests conducted on the vertical wells revealed that vertical dewatering was inefficient and the vertical dewatering efforts were discontinued. During the emergency contract (initiated January 2005), Caltrans and contract drilling crews developed additional geotechnical borings. These borings were ultimately completed as slope inclinometers and piezometers.

Horizontal dewatering exceeded expectations. Initial flow rates from newly installed horizontal drains varied from 2 gal./min. to over 60 gal./min. These rates typically decreased within the first 48 hours the flow rates diminished by approximately 80 percent. By mid-February 2005, more than 500,000 gallons of water per day was issuing from the horizontal drains developed in the hillside. By March 2005, the rate decreased to 200,000 gal./day. By April, 73 horizontal drains totaling in length more than 17,000 meters continued to remove 30,000 gallons of water per day from the hillside.

**Landslide Stability Analyses**

During the months January through April 2005, Caltrans personnel performed geotechnical studies that included slope stability analyses to evaluate the existing condition of the landslide,
the impact of the dewatering effort on the stability and to propose a suitable long term mitigation solution. Several mitigation strategies were evaluated including:

- Dewatering
- Resloping and/or removal of slide mass
- Realigning the highway
- Construction of a cut and cover tunnel
- Buttressing
- Walls and tie-backs
- Shear key
- Reticulated piles
- Soil mixing

Following completion of the stability analyses and consideration of the various mitigation schemes, it was decided that a reduction in driving mass of the landslide accompanied by dewatering raised the factor of safety against sliding to acceptable levels. Analyses indicated that lowering of the groundwater level by 10 meters would result in about a 15 percent increase in the stability of the landslide. However, due to the uncertainty of achieving a permanently lowered groundwater level in the landslide, mitigation measures were considered assuming a relatively minor groundwater level drop from the peak levels observed. After considering the cost and the feasibility, reducing the driving mass by grading was selected as the long term mitigation method.

**Proposed Landslide Mitigation Design**

Analyses by Caltrans indicated that removing the driving mass located in the western portion of the landslide would result in an approximately 50 percent increase in the overall stability of the landslide. This removal required excavation of landslide material from the existing grades to
depths ranging up to 25 meters and 45 meters at the south and north ends of the landslide, respectively.

Analyses indicated that the potential instability of the intraslides could be mitigated by the construction of a 20-meter high, reinforced earth buttress with a 1:2 (V:H) slope face to replace the distressed cut slope along the west side of the highway through-cut. The buttress would be constructed with a 2-meter-deep, 30.5-meter-wide shear key at the base and a 1:1.5 (V:H) back cut. The analysis indicated that the buttress constructed with 18 layers of geosynthetic materials (geogrid) with long term average design strengths (LTADS) ranging from 15 KN/m to 30 KN/m would increase the factor of safety of the intraslides by about 37 percent.

Excavation to reduce the landslide driving mass and construction of the graded buttress would be accompanied by the construction of an additional 13,000 meters of new horizontal drains on the east facing slopes (including the buttress slope) as an added safeguard to supplement the 17,000-meter long system of drains already installed. The intent of the drains is to reduce hydrostatic pressures on the buttress fill exerted by the seepage of water through rock fractures and joints and also to continuously remove groundwater and lower the water level to improve the stability of the landslide with time.

Implementing the aspects of the geotechnical analysis, Caltrans completed the remedial grading plans in November 2005.

**SCOPE OF REMEDIAL GRADING OPERATIONS**

Remedial grading operations commenced in March 2006. Earthwork construction within the subject area consisted primarily of removing the upper portion of the landslide by excavating to grade a portion of the easterly flank of Townsend Peak in the westerly portion of the slide (Figure 2) and the adjacent area to the east formerly referred to as the “Duck Pond” area. The excavation created a northeasterly facing, 80-meter-high cut slope that ranges in inclination from approximately 1:1.5 (V:H) to 1:2 (V:H) and a southerly facing, gently sloping plane that ranges in inclination from approximately 1:5 (V:H) to 1:12 (V:H). Based on field observations made during grading, the geotechnical investigation and design report (Caltrans 2005) and data logged in the new slope inclinometer borings installed within the landslide mass, the remaining thickness of the landslide within the project site ranges from less than a meter to greater than 95 meters.
The material generated during excavation was placed as engineered fill in the disposal canyon at the southerly end of the project site (Figure 2). The east wall of the disposal canyon consists of the engineered fill embankment slope of Interstate 5 and the west wall of the canyon consists of bedrock of the Marple Canyon Sandstone Member. The disposal fill created a southeasterly facing, approximately 62-meter-high fill slope that ranges in inclination from approximately 1:1.5 to 1:3.5 (V:H) with the majority of the slope being constructed at a 1:2 (V:H) ratio.

To mitigate the intraslides, a portion of the existing approximately 1:1.5 (V:H) cut slope that ascends from the westerly side of the highway was reconstructed as a 1:2 (V:H) buttress fill slope. Based on observation and mapping of favorable geological conditions during remedial grading, the overall length of the buttress was able to be shortened to approximately 280 meters from the originally proposed 360 meters.

Remedial grading operations were completed in August 2007. URS Corporation subsequently prepared a report summarizing the results of the earthwork observation and geotechnical testing performed during remedial grading (URS, 2007).

**NEW LANDSLIDE INSTRUMENTATION**

Since most previously installed slope inclinometers were destroyed by landslide movement or by the remedial grading operations performed during removal of the landslide driving mass, a series of 18 temporary survey monuments were installed at strategic locations at the site during the following buttress construction period where the factor of safety of the landslide would be temporarily lowered (i.e. temporary removal of resisting mass during excavation of the buttress back cut, which was performed in a series of designed, sequential slot cuts). These monuments were surveyed beginning on a twice-per-day and later on a once-per-day basis during critical times of the remedial grading to help detect any adverse movement of the slide. The onsite geologist plotted and analyzed the survey data on a daily basis. No adverse movement was detected during buttress construction.

Later, when the progress of remedial grading allowed, four new permanent slope inclinometers and four new piezometers were installed within the remaining slide mass in March 2007. Monitoring of the inclinometers and piezometers began immediately. During the period from March 2007, through the completion of remedial grading in August 2007 and through to the present (April 2008), no movement of the landslide has been recorded in the inclinometers.
CONCLUSION

As data from the new slope inclinometers indicate that the landslide’s movement has been arrested, this points to a successful mitigation design based on Caltran’s geotechnical investigation and stability analyses. Full-time observation by geologists during remedial grading operations resulted in the development of detailed geologic mapping that enabled verification of the original geologic model used in the analyses. An additional benefit of full-time geologic observation and mapping resulted in significant financial savings in the construction of the buttress fill slope (i.e. decreasing the originally proposed length based on verification of favorable geologic conditions).
REFERENCES

Caltrans, 2005  “Geotechnical Design Report for the Mitigation of the Templin Highway Landslide (Final Draft),” 4 April.


Preliminary Results from the National Park Service Retaining Wall Inventory Program

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Abstract
In the past year and a half approximately 2500 retaining walls have been inventoried and assessed according to the National Park Service Retaining Wall Inventory Program (WIP). The WIP represents a standard protocol for classifying retaining wall type and purpose, the condition of wall elements, and recommendations for actions regarding the walls. The WIP is an asset management tool that can be used to plan for future maintenance costs and to prioritize activities within limited budgets.

The results show that 90 percent of the retaining walls support fills, and almost half of those are culvert headwalls. Cut walls and walls for other purposes comprise 10 percent of the inventory. Though 17 wall types were identified, mortared and dry-laid stone walls are predominant. Five to fifteen wall elements were rated for each wall and individual element ratings varied over the full range, from 1 to 10. Final, composite wall ratings varied from approximately 50 to 100 out of a possible range of 10 to 100. Twenty nine percent of the walls received a recommendation for maintenance, repair or replacement. The estimated total replacement cost (value) for the asset was estimated at $300 million and the cost of the recommended actions was $10 million.

The data also reveal the difficulty in using numerical ratings to prioritize activities and predict the investment needed to maintain the asset (the full inventory of retaining walls). The reasons for this include the disparity in the significance of specific wall elements, the wide variation in the size of retaining walls, cultural significance, and the consequences of failure, all of which are addressed in the WIP. The results show that even with consideration of these factors, correlations between ratings and recommended actions (or costs) are approximate. These findings will be of value to owners and agencies considering similar asset management systems.

1.0 INTRODUCTION
The NPS Retaining Wall Inventory Program (WIP) was developed to quantify and characterize wall assets associated with Park roadways in terms of their location, geometry, construction attributes, condition, failure consequence, cultural concerns, apparent design criteria and cost of structure maintenance, repair or replacement. The main intent of the inventory is to determine the backlog of needs associated with Park wall assets, defined as “equipment” ascribed to the “parent” roadway asset currently evaluated under the NPS Road Inventory Program (RIP). Wall inventory condition and repair/replace work order data are provided by the WIP database to the NPS Facility Management Software System (FMSS), the primary asset documentation, management and planning platform resident at each Park. In addition, bridge, culvert and traffic barrier asset data are also provided to FMSS via similar databased inventory programs.

The WIP was commissioned at the request of the NPS Washington Office (WASO), Park Facility Management Division. The program is supported by both NPS WASO personnel and staff from the Federal Lands Highway Division (FLHD) of the Federal Highway Administration (FHWA). Both organizations are equally responsible for the development and management of the WIP; the FLHD has taken the lead for delivery of field inventories, while the NPS is primarily responsible for integration of WIP data within the FMSS asset management system.
Similar to the RIP program, it is the intent of the WIP inventory to periodically reassess retaining wall resources at program Parks to further develop asset management strategies for Park roadways. Although highly focused on the asset management needs and processes unique to the NPS, this inventory and assessment approach should find application within a broader national audience as federal, state and local agencies tackle retaining wall asset issues tied to transportation infrastructure.

The WIP has been developed and initially delivered under three well-defined phases of work. Phase 1 investigated the feasibility of developing and conducting retaining wall inventories for the NPS, ultimately providing specific recommendations for inventory methods and practices supporting the needs of the FMSS asset program.

Following Phase 1 completion in early 2006, Phase 2 undertook developing, refining and testing data collection methods and processes. Program efforts focused on the refinement and definition of approximately 65 wall data attributes, development of field data collection procedures, field forms, and associated field guides and cost information, advancement of FMSS data transfer processes, and the development of an MS Access-based, fully searchable WIP database. Several pilot studies were also conducted during Phase 2.

Data collection, storage and transfer methods and processes were finalized in March 2007, prior to initiating full-scale Park inventories under Phase 3. Program training was also provided at that time to approximately 25 inventory participants, including geotechnical, geological and design disciplinary engineers and support staff from the NPS and the three FLHD division offices (Vancouver, WA; Lakewood, CO; Sterling, VA). Phase 3 field work concluded in February 2008, with inventory teams completing assessments on more than 2,500 retaining walls in 26 National Parks, Monuments and Recreation Areas across the U.S. This initial inventory effort, thought to encompass the majority of retaining wall structures within the Parks system, serves as the basis for updated program developments included in the soon to be released “National Park Service Retaining Wall Inventory Program - Procedures Manual.

2.0 GENERAL INVENTORY/ASSESSMENT PROCESS

Figure 1 identifies the four primary categories of activities comprising the FLHD data collection contribution to the Wall Inventory Program, and lists specific activities under each. This process ultimately results in data transmittal to the NPS FMSS asset program. Providing consistent, high-quality field inventories, and ensuring the long-term security and accessibility of Park wall data, requires all inventory contributors be fully trained on inventory procedures and program delivery expectations.
3.0 INVENTORY CRITERIA AND GUIDELINES

This section presents retaining wall acceptance criteria, overviews the approximate 65 wall attributes and descriptors that are logged, measured, calculated or assessed during field inventories, and provides general guidance on the application and interpretation of inventoried wall elements. Although seemingly straightforward, the apparent simplicity of describing, measuring and evaluating earth retaining structures can be deceiving. For example, in some circumstances it can be very difficult for inventory teams to simply classify the function of a particular wall. Is the wall present on the inside of a switchback curve a fill wall or a cut wall? Is it an integral part of the bridge wingwall, or does it primarily support the bridge approach? Is it a highly battered dry-laid stone wall, or rock inlay slope protection? Should it be considered a wall with a culvert, or a culvert headwall? In fact, this last example prompted the wall program to increase minimum height requirements for culvert headwalls to better capture those significant headwall structures potentially impacting overall road performance, while excluding the thousands of culvert features better managed under a separate asset inventory.
Clearly, opinions will vary from time-to-time as to how the criteria and wall element definitions should be interpreted and applied to field conditions. During the development of this program, inventory teams were often challenged to best describe unique wall conditions, and were occasionally required to exercise judgment beyond available program guidance. Regardless of the situation, inventory teams need to bear in mind that the ultimate goal of the program is to identify retaining structures in need of maintenance, repair or replacement.

3.1 Wall Acceptance Criteria
The following wall acceptance criteria assist inventory teams in determining what constitutes a qualifying retaining wall structure, and whether or not it should be included in the inventory:

(1) The inventory includes retaining walls, together with qualifying culvert headwalls, located on all classes of paved Park roadways and parking areas, as either surveyed under the Park RIP program or identified by Park facilities, maintenance or resource staff.
(2) The retaining wall must reside within the existing roadway or parking area prism, generally defined within the known or assumed construction limits, and must support or protect the roadway or parking area.
(3) The maximum wall height, measuring only that portion of the wall structure intended to actively retain soil and/or rock, must be greater than or equal to four feet. For culverts, maximum headwall heights must be greater than or equal to six feet.
(4) When known or verifiable, wall embedment is considered in determining maximum retaining wall height for wall acceptance; however, embedment is not considered for wall face area dimensioning or condition rating.
(5) Covered or buried retaining structures are included in the inventory when known to meet the aforementioned wall height requirements, and when locations are known or verifiable.
(6) Walls are further defined by an internal wall face angle greater than or equal to 45° ($\geq 1H:1V$ face slope ratio).
(7) When wall acceptance based on the above criteria is marginal or difficult to discern, include the wall in the inventory, particularly where the intent is to support or protect the roadway or parking area and where failure would result in significant impacts, requiring replacement with a similar structure.

In general, the above criteria attempt to qualify walls for the WIP based on association with Park roadways, contribution to roadway stability and safety, and wall geometrics.

3.2 Wall Data Collection
Wall attributes within five general data categories are described, measured, evaluated and/or rated to define and quantify wall assets:

- **Wall Location Data**: Walls are located by Park name, route number/name, side of roadway, RIP wall start and end milepoint, and calculated RIP wall start latitude/longitude.
- **Wall Description Data**: Walls are described by function, type, year built, architectural facings and surface treatments. Measurements are recorded pertaining to wall length,
maximum height, face area, face angle, and vertical and horizontal offsets from the roadway. Photos are also logged for each wall, noting location relative to the roadway, major wall features, and overall element conditions. Figure 2 illustrates several of the key wall measurements recorded in the inventory.

- **Wall Condition Assessment**: Primary and secondary wall element conditions are described relative to extent, severity and urgency of observable distresses, and then numerically rated, giving due consideration to data reliability. Different wall elements and numbers of elements (ranging from 5-15 elements rated per wall) are rated for different wall types and settings, per the Wall Elements listed in Table 1. In addition, the overall performance of the wall system is also evaluated and rated. The wall element ratings and overall performance rating are then weighted and combined to arrive at a final, overall wall condition rating (described in the Procedures Manual).

- **Wall Action Assessment**: Objective consideration is given to (1) the final wall element condition numerical rating, (2) any identified requirements for further site investigations (measure of data reliability), (3) the apparent design criteria employed (e.g., AASHTO), (4) any cultural concerns, and (5) the consequence(s) of failure to determine a recommended action: no action – monitor the wall; conduct maintenance-level work; repair wall elements; replace wall elements; replace the entire wall.

- **Work Order Development**: Brief, yet descriptive work orders are provided when maintenance, repair or replace actions are required. Unit costs for major work items are generated from the WIP Cost Guide, available Park cost data, etc., to arrive at preliminary cost estimates.
Figure 2. Schematic of wall measurements recorded for walls located above or below roadway grade.

Table 1 lists the various wall functions, types, architectural facings, surface treatments and rated wall condition elements included in the WIP inventory. Note that elements are visible parts of a retaining wall. An obvious limitation is that many key components of a retaining wall may not be visible (anchors, for example), so an additional attribute is documented. Performance is rated separate from elements as a simple way of capturing whether or not non-visible components of the wall are functioning adequately…Detailed definitions and applications of each are beyond the scope of this paper, but available in the WIP Procedures Manual.

Wall elements are described in the field via a written “Condition Narrative” – a concise, descriptive narrative of element condition sufficient to characterize types, severity, extent and urgency of element distresses and fully support the element condition rating. Wall conditions are described within four general distress categories: Corrosion/Weathering, Cracking/Breaking, Distortion/Deflection, and Lost Bearing/Missing Elements. Element Condition Ratings are then determined through the application of a 1-10 General Condition Rating Scale, shown in Table 2.

Wall elements are defined, evaluated and rated to the extent practicable, and are generally easy to discern in the field. The inspecting engineer will nonetheless need to rely on their knowledge of wall systems and good judgment when interpreting the intent of each wall element over the wide variety of wall settings and applications to be encountered.
Table 1. Key elements of the WIP program.

<table>
<thead>
<tr>
<th>Wall Function</th>
<th>Wall Type</th>
<th>Architectural Facing</th>
<th>Surface Treatment</th>
<th>Wall Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill Wall</td>
<td>Anchor, Tieback H-Pile</td>
<td>Brick Veneer</td>
<td>Bush Gun</td>
<td>Piles and Shafts</td>
</tr>
<tr>
<td>Cut Wall</td>
<td>Anchor, Micropile</td>
<td>Cementitious Overlay</td>
<td>Color Additive</td>
<td>Lagging</td>
</tr>
<tr>
<td>Head Wall</td>
<td>Anchor, Tieback Sheet Pile</td>
<td>Fractured Fin Conc.</td>
<td>Galvanized</td>
<td>Anchor Heads</td>
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<td>Bridge Wall</td>
<td>Bin, Concrete</td>
<td>Form-lined Concrete</td>
<td>Painted</td>
<td>Wire/Geosyn. Facing</td>
</tr>
<tr>
<td>Slope Protection</td>
<td>Bin, Metal</td>
<td>Plain Concrete</td>
<td>Preservative</td>
<td>Bin or Crib</td>
</tr>
<tr>
<td></td>
<td>Cantilever, Concrete</td>
<td>Planted Face</td>
<td>Silane Sealer</td>
<td>Concrete</td>
</tr>
<tr>
<td></td>
<td>Cantilever, Soldier Pile</td>
<td>Sculpted Shotcrete</td>
<td>Stain</td>
<td>Shotcrete</td>
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<tr>
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<td>Cantilever, Sheet Pile</td>
<td>Shotcrete</td>
<td>Tar Coated</td>
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<td>Crib, Concrete</td>
<td>Steel/Metal</td>
<td>Weathering Steel</td>
<td>Block/Brick</td>
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<td></td>
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<td>Stone</td>
<td>Other</td>
<td>Placed Stone</td>
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<tr>
<td></td>
<td>Crib, Timber</td>
<td>Simulated Stone</td>
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<td>Stone Masonry</td>
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<td>Gravity, Block/Brick</td>
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<td>Gravity, Dry Stone</td>
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<td>Architectural Facing</td>
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<td></td>
<td>Gravity, Gabion</td>
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<td></td>
<td>Traffic Barrier/Fence</td>
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<td></td>
<td>Gravity, Mortared Stone</td>
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<td>Road/Shoulder</td>
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<td></td>
<td>MSE, Geosynthetic Face</td>
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<td>Upslope</td>
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<tr>
<td></td>
<td>MSE, Precast Panel</td>
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<td>Downslope</td>
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<tr>
<td></td>
<td>MSE, Segmental Block</td>
<td></td>
<td></td>
<td>Lateral Slope</td>
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<tr>
<td></td>
<td>MSE, Welded Wire Face</td>
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<td>Vegetation</td>
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<tr>
<td></td>
<td>Soil Nail</td>
<td></td>
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<td>Tangent/Secant Pile</td>
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<td>Curb/Berm/Ditch</td>
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<td>Other</td>
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<td>Overall Performance</td>
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</table>

The requirement for sound engineering judgment in the WIP is most apparent in the manner in which recommended wall actions are determined. Whereas it is a common desire of condition-based inventory systems to directly correlate a rating range to a specific action, the WIP assessment methodology develops a numerical condition rating for applicable wall elements which is then objectively considered relative to other influencing factors to arrive at a recommended action. Other factors include such things as wall performance, the consequences of wall failure, the cultural/historic significance of the structure, and the reliability of the condition assessment data. For a numerical rating to directly tie to a recommended action without the application of engineering judgment, these factors would need to be quantified and compiled in a uniform and fair way. Theoretically, this could be done, but it would be more complex because it would require calibration to judgment, and probably wouldn’t be as accurate because of uncertainty in the calibration.
Table 2. Wall element condition rating criteria.

<table>
<thead>
<tr>
<th>Element Condition Rating</th>
<th>Rating Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>9-10 Excellent</td>
<td>No-to-very-low extent of very low distress. Any defects are minor and are within the normal range for newly constructed or fabricated elements. Defects may include those typically caused from fabrication or construction. Ratings of 9 to 10 are only given to conditions typically seen shortly after wall construction or substantial wall repairs.</td>
</tr>
<tr>
<td>7-8 Good</td>
<td>Low-to-moderate extent of low severity distress. Distress present does not significantly compromise the element function, nor is there significant severe distress to major structural components of an element. Ratings of 7 to 8 indicate highly functioning wall elements that are only beginning to show the first signs of distress or weathering.</td>
</tr>
<tr>
<td>5-6 Fair</td>
<td>High extent of low severity distress and/or low-to-medium extent of medium to high severity distress. Distress present does not compromise element function, but lack of treatment may lead to impaired function and/or elevated risk of element failure in the near term. Ratings of 5 to 6 indicate functioning wall elements with specific distresses that need to be mitigated in the near-term to avoid significant repairs or element replacement in the longer term.</td>
</tr>
<tr>
<td>3-4 Poor</td>
<td>Medium-to-high extent of medium-to-high severity distress. Distress present threatens element function, and strength is obviously compromised and/or structural analysis is warranted. The element condition does not pose an immediate threat to wall stability and closure is not necessary. Ratings of 3 to 4 indicate marginally functioning, severely distressed wall elements in jeopardy of failing without element repair or replacement in the near-term.</td>
</tr>
<tr>
<td>1-2 Critical</td>
<td>Medium-to-high extent of high severity distress. Element is no longer serving intended function. Element performance is threatening overall stability of the wall at the time of inspection. Ratings of 1 to 2 indicate a wall that is no longer functioning as intended, and is in danger of failing catastrophically at any time.</td>
</tr>
</tbody>
</table>

4.0 OBSERVATIONS

The WIP database was queried in several ways to (a) confirm that the process was working and that results were consistent with general expectations, and (b) to learn more about the NPS retaining wall asset – an asset that, until this time, had been undefined in size and condition. As described in greater detail in this section, the results show an expected range and mean in overall ratings and in recommended actions. The query results also show that the NPS retaining wall asset is heavily biased towards certain applications and, though 17 wall types were identified, the asset is dominated by a few wall types.

Figure 3 shows that essentially half of the walls are fill walls, meaning that they are outboard of the road and they retain fill. If culvert headwalls, which are uniquely identified in the inventory, are also considered as a type of fill wall, then nearly 90 percent of all walls are designed and built to retain fill. Cut walls comprise 10 percent of the inventory and a very small percentage of the walls are classified as Slope Protection, Switchback Wall, or Bridge Wall.

Nearly all culvert headwalls, and 50 percent of all walls, are gravity walls of mortared stone, as shown in Figure 4. Gravity walls of dry-laid stone comprise about 25 percent of the inventory.
Fifteen different wall types make up the remaining 25 percent. Concrete gravity and concrete cantilever walls are relatively common, and some walls are extremely rare. The inventory has only a few segmental block MSE walls and metal crib walls, and only one MSE wall with a geosynthetic wrapped face. The distribution of wall types is probably indicative of the setting where the walls are constructed and the relatively narrow time frame during which most were built. Different owners and Departments of Transportation, for example, may find a completely different distribution.

![Figure 3. Distribution of wall functions. Numbers indicate number of walls in each category.](image)

Given the wall definition we used (Section 3.2), a large number of culvert headwalls met the criteria and were inventoried. In fact, more than a third of all walls are headwalls, and this is an important observation and attribute of the NPS retaining wall asset. It leads to the question, however, as owners are starting to inventory and manage their culvert asset as well, as to where the culvert head wall should be included. The results show that culvert headwalls are overwhelmingly small, mortared stone gravity walls, in generally good condition, so the inclusion of these results bias and mask some of the other observations on what could be considered the more traditional retaining walls. Consequently, the observations described in the remainder of this section and presented in Figures 5 through 9 are based on approximately 1500 walls that are not headwalls.
Figure 4. Distribution of wall types.

As noted previously in Section 3.2, the number of elements rated is different for different types of walls. Only the wall foundation element, wall drain element, and lateral slope element - and the performance rating are rated for all walls. Thus, the total number of individual ratings varied from 5 to 15 for a wall, and element scores covered the entire range, from 1 to 10. Nevertheless, when overall ratings are calculated the maximum and mean ratings are consistent between wall types. Figure 5 shows the maximum values for most wall types are 90 to 100 and the mean values are 75 to 85. Minimum values are more variable, but where the population size is significant, the minimum rating is about 50 or less. This distribution of ratings is the first indication that the WIP successfully quantifies wall condition within a reasonable band and with enough variation in scores that some prioritization is possible.

Separate from the numerical rating of individual wall elements and overall wall performance, inspectors also enter a recommendation to take no immediate action and return to monitor the wall again in the future, or they recommend a variety of actions from maintenance of an element to replacing the entire wall. All of these actions have costs associated with them. Figure 6 shows that for most wall types with significant populations about one quarter of the walls have recommendations for some type of action to maintain the wall. The only exception is the welded wire MSE wall type, which only had one wall requiring action. It is possible that this lower percentage of needs is because walls of this type are relatively young members of the NPS wall inventory, or that they are indeed lower maintenance structures. It is beyond the scope of this
paper, but the data exist for these types of questions to be answered, and for the answers to be used in future wall design and construction decisions.

Figure 7 shows that, when considering all walls together, most of those with recommendations for action (29 percent of all walls) have either maintenance or repairing elements as the action. These are relatively low cost endeavors that could be incorporated in a routine maintenance program. Only 3 percent of all walls have recommendations to replace all or part (an element) of the wall, which is a relatively low percentage. Nevertheless, it is shown here that there are more than 30 walls that need replacement now. There are at this time no established criteria to which this outcome can be compared, but it does suggest that the asset as a whole is still in acceptable condition, that a recurring maintenance program would go along way towards keeping it that way, and that there are several walls that need replacing now.

![Figure 7. Distribution of wall ratings. Numbers indicated the number of retaining walls (excluding culvert headwalls) of each type.](image)

One of the intriguing uses of the wall inventory and condition assessment is to use calculated ratings to plan activities and needs. For example, without relying on other judgments or information, a calculated rating of 80 or higher might directly relate to a recommendation of “No Action/Monitor” and a rating of less than 50 to “Replace Wall”. In a sense, this is how many state DOTs use a rockfall hazard rating system; a rating is calculated through a structured
procedure, then sites are prioritized and actions planned based on this rating. The data show, however, that this is not an appropriate use of the WIP, at least in its current form, because the final wall rating does not tell the whole story. The final wall rating is calculated in such a way that it is primarily indicative of observed wall elements that are broken out and considered individually so that, for example, it can be determined if a particular wall element needs addressing on many walls. Because many components of a wall are not observable (they are buried, for example), the additional category of wall ‘Performance’ is rated. Wall performance is included in the final wall rating but it is also interesting to consider it individually.

![Figure 6](image.png)

Figure 6. Percent of walls with recommended maintenance or replacement actions. The number of walls of each type (excluding culvert headwalls) is shown at the top of the figure. See Figure 5 for explanation of wall types.

The three types of information collected; the final wall rating which is based primarily on element conditions, the wall performance rating, and a recommended action based on judgment of these and other considerations, as described in Section 3.2, are presented in Figures 8 and 9. The mean values of wall rating and performance condition score do show the expected trend. The highest mean correlates with ‘No Action/Monitor’ and the lowest with ‘Replace Wall’. The figures also show the band of results one standard deviation above and below the mean – outliers exist but are not shown. Inspection of the figures shows that, while the trend in the mean is as it should be, a given wall or performance rating would typically fall within one standard deviation of the mean in three different recommended action categories. This confirms that a final wall
rating alone cannot be used to predict a recommended action. Only if other considerations, such as design standards, consequence of failure and cultural significance could be realistically included in the numerical score would it be possible to use the rating in this way. In the absence of that, users should use the rating and the recommended action in conjunction to describe the condition and needs of a particular wall.

The WIP includes developing a cost for a work order any time a maintenance, repair or replace action is recommended. Therefore, cost data are available. As expected, maintenance recommendations are most common and least expensive, averaging about $4,000 per wall. Repairing and replacing elements are less common and have average costs of $25,000 to $35,000, and wall replacement is least common. Wall replacement costs average about $150,000, if culvert headwalls are excluded, and slightly less if they are included. The total asset has an estimated value of $300 million based on the quantities and types of walls inventoried and the total of all recommended work is $10 million.

Figure 7. Distribution of actions for the sum of all wall types. Numbers indicate number of walls in each category.

Finally, results show that the level of effort used to inventory and assess the walls was appropriate. The effort was such that 2-person crews were completing 15 to 25 walls per day, and most of this time was spend recording inventory information on size, type and location, rather than on condition assessment and recommendations. We consider it appropriate that most
often the crew was able to make a recommendation for work order needs in this amount of time, and without the need for additional investigation. The results show that (a) when the recommended action was to replace the wall, about 10 out of 30 walls required additional investigation to confirm the recommendation, (b) when considering maintenance, repair or replacing elements, only about 30 out of 300 walls required additional investigation, and (c) if No Action/Monitor was the recommendation, as it most often was, essentially no walls needed additional investigation. Of course, this does not mean that no investigation is needed for the design and construction of replacements and major repairs; it only means that the crews generally had enough time and observations to make recommendations for actions suitable for the WIP and its intended use.

![Figure 8. Relationship of Final Wall Ratings to recommended actions. The mean value and +/- one standard deviation error bars are shown.](image-url)
5.0 USING THE WIP RESULTS

The National Park Service (NPS) has undertaken a large effort to apply asset management processes and business practices to the management of engineered infrastructure within National Parks. It is very important to use limited funds as effectively as possible to protect resources and maintain service to the public and asset management assists with this goal.

Asset management provides tools that allow for the systematic evaluation of the following:
1. Work necessary to maintain facilities in an appropriate condition,
2. The resources necessary to do so,
3. Defining the difference between the resources needed and those available,
4. How to manage the difference.

The wall inventory and assessment focuses on the first and fourth areas for a crucial subcomponent of the NPS transportation network.

**Defining Work to Maintain Facilities**
In order to determine the work necessary, fundamental data that quantify the amount and condition of assets must be determined. The total exposed face area of the different types of retaining walls in the National Parks (those that have been assessed) has been determined. A condition assessment based on observed distresses has also been completed. This information has merit other than defining work directly. Simply providing an objective measure of the
quantity and condition of retaining walls can be used to illustrate to policy makers and funding officials the magnitude of the asset portfolio under the care of park managers. For example, the data show that 29 percent of all walls have an action that is necessary if the asset is to be maintained. The objective nature of these data can provide compelling support when preparing agency or subunit needs documentation.

A critical link that transforms an inventory and condition assessment tool into a true asset management tool is to be able to utilize quantity and condition information to define specific remedial or preventive work and to estimate costs for these actions. This assessment uses a Work Order to document these needs. The Work Order is a line-item list of costed activities needed to correct observed deficiencies. A cost guide specific to the type of work being performed is also needed to generate costed work orders. The cost guide for this effort was prepared by reviewing and analyzing contract awards in National Parks that include wall repair and replacement work elements. Time and equipment items are also included in the cost guide. It should be noted that work orders do not necessarily represent fully accurate project costs, particularly for larger rehabilitation projects. For larger projects, bundled work orders represent an estimate that must be updated based on detailed project development and design activities. This is similar to most other planning level cost estimates.

An example work order is shown below:

Remove and replace last 15 feet of wall. Place large (1/2 cy or larger) foundations stones or CIP concrete foundation beneath replaced wall section.

Foundation: 5cy*$1400/cy structural concrete = $7,000

Remove and replace masonry stone wall: 13cy*$2750/cy = $35,750 Used FHWA unit costs for masonry and concrete.

NOTE: Salvage and re-use existing wall stone. Where new material is needed, use stone consistent with existing historic fabric.

Managing the Gap Between Needs and Resources

Work Orders allow the National Park Service to specifically quantify retaining wall and other infrastructure needs based on objective evaluation and analysis criteria. This information is used for several purposes:

1. Demonstrate a backlog of need associated with maintaining our infrastructure, particularly deferred need (or Deferred Maintenance, DM)
2. Prioritize work on assets given limited funding
3. Prepare work plans and project plans that are responsive to needs and priorities
4. Incorporate transportation information into total asset management plans (Park Asset Management Plans, PAMP’s)
Deferred maintenance is the cost of performing restorative work that was delayed relative to a normal maintenance cycle. Since many retaining structures in the NPS are quite old, and have not had much scheduled maintenance performed (or even scheduled in the first place), most work identified by this assessment (even work that could be considered preventive if done on a regular cycle such as masonry repointing) is considered DM. Deferred maintenance dollar amounts provide a compelling picture of budget requirements that is very useful in preparing funding requests at park, regional and national levels.

The determination of DM also allows for the computation of other condition metrics that can be used to compare condition across asset types. When used in conjunction with the current replacement value of an asset, the Facility Condition Index (FCI) can be computed. The FCI is a comparative indicator of the relative condition of facilities as expressed by the ratio of deferred maintenance to the replacement value. Although originally developed for use with buildings, the FCI may have some applicability to road assets specifically once performance benchmarks (i.e. what value of FCI indicates “good/fair/poor” performance for roads) can be established. FCI is efficient as a high level metric, where it represents complex systems, so the NPS applies the FCI to the entire road asset and includes retaining walls as an associated piece of “equipment”. Still, an inspection of the FCI for the retaining wall component of road assets can give an indication of the overall health of the retaining walls inspected during this effort. For the WIP to date, the approximate FCI (DM/Replacement Value) = $10,000,000/$300,000,000 = 0.03 for the entire inventory. Although performance benchmarks for roads, or retaining walls, themselves, have not been established, it is likely that a low FCI like this indicates facilities in relatively good condition. Even without performance benchmarks, when the FCI concept is combined with an asset priority determination, categories for resource expenditures can be developed in a general sense. The Asset Priority Index is an assessment of how well individual assets (roads) support the function and mission of a particular park unit. Figure 10 shows how asset priority and FCI can guide resource allocation decisions. The quadrants illustrate the type of activities that should be emphasized for assets that fall within them.
Figure 10. The use of Asset Priority Index and Facility Condition Index.

Park Asset Management Plans are created to define what is required to bring a park portfolio up to an acceptable condition and sustain it over time and how to spend limited resources to best achieve this. In order to utilize the work orders discussed above within an overall asset management plan, work orders are stratified and “bundled” by work type to produce maintenance plans and specific rehabilitation project proposals. These plans and projects are compared with asset priorities and other project selection criteria to ensure merit. Plans and project proposals are then compared to estimates of available budgets. Based on this fiscally constrained comparison, high priority projects and plans are selected to be completed within a 10 year planning horizon. This process is performed across asset types (roads, buildings, utility systems, etc). Because the project proposals are based on objective quantity and condition data and have been compared to asset priorities, facility managers can be confident that the program of activities represents an efficient expenditure of limited resources to properly care for park assets.
MSE Wall Backfill Testing:
Preparation of Sample to Accurately Model Field Conditions

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March, 2008
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Brenda Waters
Linda Wicks
Abstract

The quality of MSE (Mechanical Stabilized Earth) Wall Backfill must be checked not only for minimum shear strength requirements, but also for various electrochemical parameters. Electrochemical properties include resistivity, chloride and sulfate concentrations, and pH. The shear strength is checked to assure that the required frictional resistance is available for stability of the wall system, while the material electrochemical properties are monitored to protect the metallic reinforcing elements from corrosion, to help assure longevity of the wall system.

Current PENNDOT requirements for MSE wall backfill set Chloride concentrations at 100 ppm, Sulfates at 200 ppm, resistivity greater than 5000 ohm-centimeters, pH between 6 and 10, and a minimum friction angle of 34 degrees. With these requirements there have been pervasive problems on projects not being able to get material approved for MSE wall backfill. The failures typically reside under the chloride, sulfate and/or resistivity requirements. This forces contractors to consider sources at greater distances from the project, creates material availability problems, and can result in project delays and higher costs.

The test methods developed to measure these parameters were originally developed for soil materials, using material passing the No.10 sieve. MSE backfills required on PENNDOT projects are granular, often consisting of coarse aggregates. An investigation into the problem indicted that the samples being tested in the lab are not consistent in gradation, as when actually placed as wall backfill. A study was conducted to determine the impact of testing only the minus 8 fraction of coarse (up to 1.5 inch) material, and the impact of the inconsistency of the gradation of the material when tested in the lab, as opposed to how it is actually placed in the wall systems.

Results indicated that minor adjustments in sample preparation, limiting the amount of fines in the test sample and better modeling the lab sample to the as-placed gradation, may result in many fewer material rejections, while still providing a relatively conservatively approach to assure backfill quality. The study also indicates the potential for similar problems in other areas of laboratory testing of earth materials.
Introduction

As with other state departments of transportation (DOT’s), the Pennsylvania Department of Transportation (PENNDOT) has in its toolbox, mechanically stabilized earth (MSE) walls as a resource to address the needs of a varied transportation infrastructure. In July 1989, early in the implementation of these systems within the Department, a dramatic failure of a wall occurred. The failure involved a single span adjacent pre-stressed concrete beam bridge with shallow spread footings founded on top of MSE abutments and wing walls. Three panels blew out from the one abutment face, resulting in a significant loss of structural backfill behind and above the lost wall panels (see Figure 1). Approximately 30 percent of the footing had no direct support. The panels were lost due to a series of failed wall connections. The cause of the connection failures is thought to be related to three separate contributing factors. These factors include corrosion of the connections, failure of the concrete panel joint drainage filters, and excessive backfill fines. This failure resulted in a relatively conservative approach in the Department’s policies and specifications defining the use of these systems.

Over time there have been frequent problems with selected backfill materials not meeting specification requirements, particularly the various electrochemical requirements. Backfill material failures have occurred with excessive chloride and sulfate contents, but have been most common with lower than required resistivity values. An investigation was initiated to determine if there was a preventable factor resulting in rejecting proposed backfill materials that may actually be adequate for the intended function, or was there simply a shortage or availability problem of the necessary quality materials. The investigation considered a variety of possibilities including unnecessarily restrictive backfill requirements, material quality deficiencies, and testing-related problems or anomalies.
Backfill Requirements

Pennsylvania’s requirements for MSE wall backfill are indicated in Table A, along with FHWA recommendations and the current material requirements for several other state DOT’s.

<table>
<thead>
<tr>
<th>Specification Component</th>
<th>PENNDOT</th>
<th>FHWA</th>
<th>Alaska DOT</th>
<th>GADOT</th>
<th>NYDOT</th>
<th>NCDOT</th>
<th>WSDOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chlorides (ppm)</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
<td>&lt; 100</td>
</tr>
<tr>
<td>Sulfates (ppm)</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
<td>&lt; 200</td>
</tr>
<tr>
<td>Sulfides (ppm)</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>&lt; 300</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>pH</td>
<td>6 - 10</td>
<td>5 - 10</td>
<td>5 - 10</td>
<td>6 – 9.5</td>
<td>5 - 10</td>
<td>5 - 10</td>
<td>5 - 10</td>
</tr>
<tr>
<td>Resistivity (ohm-cm)</td>
<td>&gt; 5000*</td>
<td>&gt; 3000</td>
<td>NA</td>
<td>&gt; 3000</td>
<td>&gt; 3000</td>
<td>&gt; 3000</td>
<td>&gt; 3000**</td>
</tr>
<tr>
<td>Soundness (max % loss)</td>
<td>20 #</td>
<td>NA</td>
<td>30</td>
<td>15</td>
<td>30 ##</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PI of fines</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>&lt; 5</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

Sieve Size | Gradation Requirements (Percent Passing)  
4 inches | 100  
3 inches | 100  
2 inches | 80 - 100  
¾ inch | 20 - 100  
No. 10 | 20 - 90  
No. 40 | 0 - 60  
No. 200 | 0 - 15  

Collected from agency websites

Notes:

FHWA = Federal Highway Administration  
GADOT = Georgia Department of Transportation  
NYDOT = New York Department of Transportation  
NCDOT = North Carolina Department of Transportation  
WSDOT = Washington State Department of Transportation  
* Between 2000 and 5000 ohm-cm, must meet chloride and sulfate requirements; above 5000 ohm-cm, chloride and sulfate testing requirement waved; below 2000 ohm-cm unacceptable.  
** Above 3000 ohm-cm, chloride and sulfate testing requirements waved.  
# Currently no requirement; value indicated being considered for requirement  
## Use magnesium sulfate soundness test as opposed to sodium sulfate soundness
As seen in Table A, the differences between the various agency requirements are generally small, with chloride, sulfate and pH requirements nearly identical. There is some difference among resistivity requirements. The FHWA and Georgia, New York, North Carolina and Washington State DOT's, all require resistivity values for backfill of greater than 3000 ohm-cm. In addition WSDOT waves chloride and sulfate when resistivity values are above 3000 ohm-cm. PENNDOT allows resistivity values between 2000 and 5000 ohm-cm when chloride and sulfate criteria are met, and waves chloride and sulfate testing when resistivity values are greater than 5000 ohm-cm.

Those agencies that have soundness criteria (Alaska, Georgia and New York), range from maximums of 15 to 30 percent loss, with NYDOT using the magnesium sulfate test as opposed to sodium sulfate. PENNDOT currently has no soundness criteria, but is considering implementing a sodium sulfate soundness criterion of 20 percent maximum loss. For fines content (minus No. 200 material), agencies where criteria were available ranged from a maximum 12 percent for GADOT, to 15 percent for NY and WS DOT's. PENNDOT current requirement is 5 percent maximum fines, but is considering raising that value to a maximum 10 percent fines content.

As can be seen from these results, PENNDOT requirements are somewhat more restrictive than the other agencies indicated, but not to an excessive degree. While some materials that would meet other agency standards would not meet PENNDOT criteria, the number of sources that would be eliminated is likely small. This tends to indicate that overly restrictive backfill criteria is not a major concern for the backfill materials available on PENNDOT projects, and implied that this is not a likely cause of excessive material deficiencies.

**Backfill Testing**

The next factored considered was the possibility of testing anomalies. The test methods used for testing of MSE wall backfill materials for electrochemical parameters are indicated in Table B. The methods cover testing for chloride and sulfate content, pH and resistivity. As indicated by the titles of each of the test methods, the procedures were developed for testing soil materials. Review of the test methods indicates that the prepared test sample is to consist of material finer than the No.10 (2.00 mm) sieve. This requirement is consistent for all four of the electrochemical test methods, and appears to be in at least part a function of the necessary testing apparatuses.

| **Table B** – MSE Wall Backfill Electrochemical Tests |
|---|---|
| Method Designation | Method Title |
| T 288 | Standard Method of Test for Determining Minimum Laboratory Soil Resistivity |
| T 289 | Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing |
| T 290 | Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil |
| T 291 | Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil |
The test methods do not clearly specify if the minus No.10 material shall be proportioned consistent with the gradation of the material as placed, or to simply test the minus No.10 fraction. The standard approach has been to test a non-proportioned, straight minus No.10 sample. As shown in Table A, and also in Figures 2 and 3, the relative percent of No.10 material can range tremendously, and by Figure 2, the theoretically range could be anything from 0 to 100 percent passing the No.10 sieve. From a practical standpoint, in order to meet the maximum 5 percent fines criteria, the minus No.10 material is typically low, as many contractors choose to use an open graded free draining manufactured aggregate, to permit cold weather construction. Considering the generally low fraction of minus No.10 material, the question arises as to whether the test method, as applied, is accurately representing backfill materials as placed for wall construction.

Table B and C, and corresponding Figure 3, show information for two materials meeting PENNDOT gradation requirements for MSE wall backfill materials. One material is a typical PENNDOT No.57 coarse aggregate, and the other represents what would be a dense graded MSE wall backfill. As indicated in Table B for typical No.57 aggregate, the estimated percent of surface area for the fraction passing the No.100 and No.200 sieves is approximately 85 percent of the total sample surface area. The total estimated surface area for one cubic feet of material (assuming 20% porosity) is 1280 ft$^2$. When dealing with just the minus No.10 fraction of the material, the percent of surface area for the fraction passing the No.100 and No.200 sieves is
### Table B: Gradation versus Surface Area – No. 57 Coarse Aggregate

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Sieve Size (in or No.)</th>
<th>Average Particle Size Retained (mm)</th>
<th>Surface Area per Particle (ft²)</th>
<th>Full Gradation</th>
<th>Minus No. 10 Material Only</th>
<th>Coarse Particle Substitution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Percent</td>
<td>Percent</td>
<td>Percent</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Retained</td>
<td>of Total Surface Area</td>
<td>Retained</td>
</tr>
<tr>
<td>37.5</td>
<td>1.5”</td>
<td>43.8</td>
<td>6.47E-02</td>
<td>15</td>
<td>0.78</td>
<td>10</td>
</tr>
<tr>
<td>25.0</td>
<td>1.0”</td>
<td>31.3</td>
<td>3.30E-02</td>
<td>40</td>
<td>2.90</td>
<td>37</td>
</tr>
<tr>
<td>19.0</td>
<td>0.75”</td>
<td>22.0</td>
<td>1.64E-02</td>
<td>15</td>
<td>0.78</td>
<td>10</td>
</tr>
<tr>
<td>12.5</td>
<td>0.75”</td>
<td>15.8</td>
<td>8.39E-03</td>
<td>40</td>
<td>2.90</td>
<td>37</td>
</tr>
<tr>
<td>9.5</td>
<td>3/8”</td>
<td>11.0</td>
<td>4.09E-03</td>
<td>25</td>
<td>2.60</td>
<td>33</td>
</tr>
<tr>
<td>4.75</td>
<td>#4</td>
<td>7.13</td>
<td>1.72E-03</td>
<td>10</td>
<td>1.60</td>
<td>20</td>
</tr>
<tr>
<td>2.36</td>
<td>#8</td>
<td>3.56</td>
<td>4.27E-04</td>
<td>5</td>
<td>1.61</td>
<td>21</td>
</tr>
<tr>
<td>2.0</td>
<td>#10</td>
<td>2.18</td>
<td>1.61E-04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.18</td>
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<td></td>
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<td>1280</td>
<td>100</td>
<td>22,731</td>
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</table>

|                |                        |                                   |                                 | TOTALS         | 100                       | 100                         | 285                          | 73                           |
|                |                        |                                   |                                 |                |                          |                             |                              |                              |

Notes and Assumptions:
1) All spherical particles
2) Assumes 20 percent porosity
3) The “<#200” row is material passing the No. 200 sieve
4) Average particle size is determined by the numerical average of the opening size of the sieve on which the particle was retained, and the previous sieve that the particle passed.
5) Average particle size for material passing the No. 200 sieve, was the numerical average of the No. 200 sieve opening (0.075mm) and a lower bound of 20 microns (0.020mm) for silt particles
6) All material was non-plastic (no colloids)
<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Sieve Size (in or No.)</th>
<th>Average Particle Size Retained (mm)</th>
<th>Surface Area per Particle (ft²)</th>
<th>Full Gradation</th>
<th>Minus No. 10 Material Only</th>
<th>Coarse Particle Substitution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Percent Retained</td>
<td>Percent of Total Surface Area</td>
<td>Surface Area per Cubic Foot</td>
</tr>
<tr>
<td>37.5</td>
<td>1.5”</td>
<td>43.8</td>
<td>6.47E-02</td>
<td>5</td>
<td>0.09</td>
<td>2</td>
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<tr>
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<td>1.0”</td>
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<td>3.30E-02</td>
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<td>0.10</td>
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<td>7</td>
<td>0.24</td>
<td>7</td>
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<tr>
<td>12.5</td>
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<tr>
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<td>4.09E-03</td>
<td>15</td>
<td>1.14</td>
<td>31</td>
</tr>
<tr>
<td>4.75</td>
<td>#4</td>
<td>7.13</td>
<td>1.72E-03</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.36</td>
<td>#8</td>
<td>3.56</td>
<td>4.27E-04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>#10</td>
<td>2.18</td>
<td>1.61E-04</td>
<td>21</td>
<td>5.20</td>
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<td>TOTALS</td>
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<td></td>
<td></td>
<td>100</td>
<td>100</td>
<td>2709</td>
</tr>
</tbody>
</table>

Notes and Assumptions:
1) Dense graded MSE backfill is based upon PENNDOT MSE wall backfill requirements
2) All notes from Table B above also apply
approximately 95 percent of the total sample surface area, and the total estimated surface area for one cubic foot of material is 22,700 ft\(^2\). This represents a nearly eighteen fold increase in surface area when considering only the minus No.10 fraction of the sample. As observed in Table D, while the surface area of the fine particles is very small, the number of particles in just a small percentage by weight is very high. The result is a disproportionately high surface area for a given percentage of fines, versus the same percentage of a coarse particle (say No.16 or larger). In fact as shown in Figure 4, the surface area per unit volume dramatically increases below (smaller than) a No.40 particle.

By substituting the coarser fraction (larger than No.8 particles) with a weight equivalent amount of material passing the No.8 sieve but retained on the No.16 sieve, a sample can be prepared that does not appreciably change the total surface area of the material. This is observed in the last block of Table B (Coarse Particle Substitution). As shown, the percent of total surface area for the minus No.100 and minus No.200 fractions decreases to approximately 60 percent, while the substituted fraction of No.10 material represents 34 percent of the total surface area. The surface area per unit volume is 1784 ft\(^2\), which is higher, but relatively close to the surface area per unit volume (1280 ft\(^2\)) of the full sample. The coarse substitution sample has only 1.4 times the unit volume surface area of the full sample.

Figure 3 - Gradation Curves for Various MSE Wall Backfills
### Table D: Particle Surface Area versus Number of Particles per Volume

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Sieve Size (in or No.)</th>
<th>Average Particle Size Retained (mm)</th>
<th>Surface Area per Particle (ft²)</th>
<th>Number Particles per cu. ft. material (assuming 20% voids)</th>
<th>Total Surface Area per cu. ft. material at 20% Voids (ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5</td>
<td>1.5”</td>
<td>43.8</td>
<td>6.47E-02</td>
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<td>0.75”</td>
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</tr>
<tr>
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</tr>
<tr>
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<td>32,506</td>
<td>133</td>
</tr>
<tr>
<td>4.75</td>
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<td>1.72E-03</td>
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<td>205</td>
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<td>30,801</td>
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</tbody>
</table>

### Figure 4 - Surface Area per Unit Volume versus Particle Size
In this case the coarse fraction of the original sample is replaced and represented in the test sample by a smaller material, but still coarse relative to the fines fraction. Relative to the fines, these intermediate size particles still have comparatively low surface area per unit volume of material. The intermediate size substitution material provides 604 ft$^2$ of surface area per unit volume, as opposed to 100 ft$^2$ of the replaced coarse (plus No.8) material. Note that the surface area of the No.100 and finer material remains the same.

A similar, but much more modest effect is observed with the MSE dense graded backfill (Table C). In this case, because of the better distribution of particle sizes, the difference in unit volume surface area is only a factor of 2.2 (2709 ft$^2$ for the full sample versus 5975 ft$^2$ for the minus No.10 fraction). With the coarse particle substitution, a unit volume surface area of 2906 ft$^2$ is produced – nearly matching the full gradation sample. Using the coarse particle substitution, a theoretically more representative sample of backfill material (relative to material actually placed during construction), can be provided for laboratory testing. The implication is that the disproportionately greater surface area resulting from testing only the minus No.10 fraction of the material, provides a greater concentration of free chloride and sulfate ions, increasing the measured concentrations of chlorides and sulfates, and decreasing the material resistivity.

**Proof of Concept**

In order to determine if coarse particle substitution would provide a viable correction in the measurement of electrochemical properties for MSE backfill, a laboratory study was conducted. The study looked at a variety of current MSE wall backfill sources. The plan included testing each source as per normal procedure, and then testing the materials between successively smaller sieve windows (see Table E). The only difference was that a minus No.8 fraction would be used for the normal sample instead of the usual minus No.10 fraction. No.8 and No.10 particles are 2.36mm and 2.00mm in size, respectively. This would then permit a slightly larger minus No.8 to plus No.16 substitution window, should the study results demonstrate viability. All four required MSE backfill electrochemical tests were included in the study: pH, resistivity, chloride content and sulfate content.

| Table E: Sieve and Particle Size Windows for MSE Backfill Electrochemical Study |
|-----------------------------|------------------|
| Sieve Windows               | Particle Size Range (mm)      |
| Minus No.8 Composite Sample (Normal Procedure) | Less Than 2.36 mm |
| -4/+8                       | < 4.75 mm and > 2.36 mm |
| -8/+16                      | < 2.36 mm and > 1.18 mm |
| -16/+30                     | < 1.18 mm and > 0.600 mm |
| -30/+50                     | < 0.600 mm and > 0.300 mm |
| -50/+100                    | < 0.300 mm and > 0.150 mm |
| -100/+200                   | < 0.150 mm and > 0.075 mm |
| Minus No.200 Material       | Less than 0.075 mm |
The results of the testing are indicated in Figures 5 through 9. A total of 264 individual test increments were run for the study. Figure 5 shows the results for pH testing. No significant change or trend was observed for the materials across various grains sizes. This is not unexpected as the chemistry for any given processed aggregate would not be anticipated to vary with particle size.

The results for chloride content are shown in Figure 6. As clearly observed the chloride concentration increases with decreasing particle size. The greatest change observed was for Source D, increasing from 3 ppm to 32 ppm from the largest to smallest particle size – a total change of 29 ppm. The other three sources had increases in chloride concentration typically in the range of 14 ppm. The minus No.8 composite sample results ranged from 9 to 13 ppm chloride concentration. While none of the sources tested would have produced failing results, the surface area relationship is clear. Of interest is that the chloride concentration of all four sources converge at the -4/+8 particle range. No explanation for this observation is offered.

The results for sulfate content are indicated on Figures 7 and 8. Source D exhibited a sulfate concentration much higher than the other three sources and is shown on Figure 8 on a larger y-axis scale, while Sources A through C are plotted on Figure 7 to a y-axis scale appropriate for the results obtained. A similar, but somewhat more dramatic, trend in increasing sulfate concentration with decreasing particle size is observed, as with the chloride testing. The greatest range was for Source D (Figure 8), from 250 to 1370 ppm from the largest to smallest particle size – a total change of 1120 ppm. Figure 7 indicates that Sources A and C also showed...
significant increase with decreasing particle size, with Source A showing an increase of 77 ppm and Source B an increase of 48 ppm. The change for Source C is negligible and no conclusions are drawn. As with the chloride testing, the sulfate concentrations for the composite minus No.8 samples were typically within the range of the largest to smallest particle size windows. The results again indicate a surface area influence on the measured concentrations.

Figure 9 presents the results for the resistivity testing. The results indicate that resistivity is impacted by particle size even more significantly than chloride and sulfate concentrations. Measured resistivity decreased with decreasing particle size. Source C demonstrated the largest decrease, with resistivity decreasing from 56,600 ohm-cm to 5700 ohm-cm from the largest to smallest particle size – a total change of 50,900 ohm-cm. Source F demonstrated similar results. Sources A, B and E showed less significant, but still substantial, decreases in resistivity. The values obtain for the three sources were very similar. Of greater significance is how much closer the measured resistivity of the composite minus No.8 sample for each source was to its companion minus No.200 sample, as compared to chloride and sulfate results. This result provides the strongest evidence of the surface area influence on the measured values, and supports a coarse particle substitution for sample preparation.
Figure 7 - Sulfate Content versus Particle Size

Figure 8 - Sulfate Content versus Particle Size
Conclusion

The laboratory test results, and the supporting surface area analysis, strongly suggest preparing MSE wall test samples in a manner more consistent with the condition of the material at placement and during service. A combination of the requirements of the current test methods, limitations due to testing apparatus, and the particle size-surface area analysis, suggests a substitution of sample material coarser than the No.8 sieve, with an equal percent by weight of minus No.8 to plus No.16 material. The fractions of all materials finer than No.16 would remain unchanged. The test methods would only be modified in terms of the sample preparation (gradation), and required min/max values for the parameters would remain unchanged.

Testing would have to be conducted on samples prepared under both the current and proposed methods to validate the recommendations. A similar condition may exist with shear strength testing. Preliminary findings indicate that coarse grain substitution may be necessary to produce more accurate results when shear testing granular materials.
References:

Telluride Keystone Hill SH 145
Complex Geological Conditions that Required Innovative Retaining Wall and Foundation Designs

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Richard Andrew, P.G.
Yeh and Associates, Inc.

Joseph Colley, P.E.
Colorado Department of Transportation
ABSTRACT

Yeh and Associates has been involved with evaluation, analysis and design for roadway improvement of a two lane mountainous road known as Keystone Hill, near Telluride Colorado. This section of roadway is approximately one mile in length at a grade of 6 to 7 percent with cut and fills slopes in excess of 500. The roadway was widened from two to three lanes. One section of the project crosses a landslide section which is adjacent to multi-million dollar home structures. Yeh and Associates role on the project was to provide a geologic / geotechnical investigation, slope stability evaluation, and retaining wall and foundation design for the Colorado Department of Transportation (CDOT).

The geologic and geotechnical investigation utilized both truck mounted and helicopter transported drilling rigs. The helicopter transported rigs were used to evaluate subsurface conditions at the proposed wall locations which were not easily accessible. Geologic hazards included landslides, rockfall, and debris flows. The competency of the bedrock in the area was affected by faulting with bedrock at depths in excess of 200 feet in certain sections.

The roadway alignment was designed with both cut and fill retaining walls. Because of the steep terrain, MSE fill wall sections were over 30 feet in height to gain an additional outer roadway lane. The fills walls were designed as a system of mechanically stabilized earth (MSE) walls that were supported by micropile foundations through a landslide section. The micropiles were designed to transfer the vertical loading of 30 foot high MSE walls below critical failure surfaces within the landslide. The cut sections consisted of soil nail walls and draped mesh for rockfall mitigation. Ground anchor tiebacks were not a viable option due to excessively deep bedrock. Inclinometers were installed and monitored for earth movements prior to and during construction.

INTRODUCTION

State Highway 145 in Colorado, is the primary two-lane route between Telluride and the surrounding bedroom communities. The highway follows along the general path of the San Miguel River that has deeply incised into the earth creating steep sided canyons. The project was undertaken to widen a 1-mile section of the existing state highway from two lanes to three lanes, creating two lanes up valley and one lane down valley. Figure 1 illustrates the general location of the project area.
The existing roadway climbs from west to east up Keystone Hill. The total elevation gain is approximately 285 feet over 1 mile. The lower section of the existing roadway consists of cuts in debris fan deposits that are in excess of 30 feet high with cutslopes approaching 0.25H:1V in isolated areas. Embankment fill slopes below the existing roadway are generally constructed at 1.5H:1V. The upper section of the project area consists of fine-grained, low plasticity materials exposed in cutslopes approaching 1H:1V. Embankment fill slopes are generally 1.5H:1V. Typical surficial and subsurface materials at the site consist of shale, sandstone and siltstone bedrock and low to high plasticity clays.

**REGIONAL GEOLOGY**

The underlying geology of the project site consists of steep colluvial slopes, alluvial stream deposits and moderately dipping sandstone bedrock. Major and minor faulting is also evident within the project area. The colluvial materials consist of angular, poorly sorted rock debris embedded in a matrix of silt, sand and clay. Most of the colluvial material is present on steep slopes that are in excess of 1H:1V. A majority of the steep slopes in the project area are comprised of debris fan depositional features. The debris fans appear to be the result of episodic debris flows that formed after intense storm events.
The bedrock in the project area is predominately comprised of the Morrison Formation in the lower sections of the valley and the Dakota and Mancos Shale in the upper sections of the valley. Intrusive volcanic rock comprises most of the elevated peaks surrounding the project area. The volcanic rock generally overlies the Mancos Formation. Intrusive dikes are also common down valley of the project area and above the roadway in isolated areas.

Landslides occur at several locations in the project area. Small slump features are also evident above and below the roadway. Shallow tension cracks approximately 1 to 2 feet deep were observed above the roadway. The tension cracks ranged from 30 to 100 feet in length and were located upslope and downslope of the proposed alignment. Displaced and leaning trees were observed. Groundwater seeps were also observed.

Major faulting also crosses the project. The fault structure appeared has an offset of 150 feet to 200 feet and bedrock units directly adjacent to the fault are heavily fractured.

**GEOLOGIC RELATED ENGINEERING CONSTRAINTS**

Debris flows, existing landslides, relatively loose subsurface materials, and erosion susceptible materials pose the greatest geologic difficulties for design and construction of the project. Based on stability evaluation of the site, many existing slope profiles have a marginal global factor of safety (FS) that ranges from 1.01 to 1.10. The marginally stable slopes are very sensitive to ground and surface water infiltration which acts to destabilize the slopes typically resulting in ground movement (i.e. slump features). Figure 2 is a photograph of the project area looking to the northwest.

![Figure 2 – Pre-construction Project Area with Notable Features.](image-url)
DESIGN OF EARTH RETAINING WALL SYSTEMS

Bidding History of the Project

The project at Keystone Hill required three separate bidding processes to award the project. The climbing lane was originally designed to be 4700 ft long, requiring the construction of 13 retaining wall structures for both the fill and cut slopes. Design began in 2003. The first bid opening was on May 12, 2005, with the two bids received ranging from 174% and 209% of the Engineers Estimate. Both bids were rejected. The project team then reduced the scope of work and revised the drawings. The re-advertised project would be shortened to a 3470 ft climbing lane and contained only 10 retaining wall structures. The fundamental design approach did not change. The second bid opening was on July 28, 2005. CDOT received 2 bids and they were 117% and 126% of the Engineers estimate. Both bids were also rejected. The project team then re-evaluated the project. The roadway was narrowed, drainage improvements were scaled back dramatically, the number of retaining structures was reduced to 7, and the climbing lane was once again shortened to a mere 3,100 feet in length. Yeh and Associates, Inc proposed using a micropile foundation support to reduce the earthwork quantities necessary for the downhill fill MSE retaining walls. On April 20, 2006, CDOT received 3 bids. Unfortunately, they were 116%, 124%, and 130% of the Engineers estimate. Fortunately, additional funding was found to award the project and move forward since justification for bid rejection was again warranted.

Design of Earth Retaining Systems

The two predominating design and construction constraints for the project were:

- Lack of staging area and long transport distances for construction materials.
- Relatively steep slopes in excess of 1H:1V both above and below the proposed roadway section.
- Large landslide section that was associated with a large scale fault with deep bedrock depths.

Roadway and retaining wall design generally uses AASTHO design guidelines. The minimum slope factor of safety based on ASD AASTHO guidelines is typically 1.30 for non-critical structures. A minimum slope factor of safety of 1.50 is defined to be used where abutments, buildings or critical structures are used. Newer LRFD AASTHO guidelines indicated that a resistance factor of 0.75 (i.e. FS greater than 1.33) can be used where the slope does not contain a structural element, otherwise a resistance factor of 0.65 (i.e. FS greater than 1.54) should be used.

When designing retaining wall features in steep mountainous slopes where existing global factors of safety typically range from 1.00 to 1.10, it is difficult to attain a global factor of safety of 1.30 and attaining a factor of safety of 1.50 or greater would likely not be possible or even constructible in landslide areas of this magnitude. In order to proceed forward with the project, it
was necessary to coordinate with various entities that included CDOT Regional Management, CDOT Geotechnical, CDOT Staff Bridge, and coordination with the Design Team. Overall if strict AASTHO guidelines had to be adhered to (i.e. Minimum FS of 1.50 or greater for structural elements) the project would not have gone forward since the project cost would have doubled or even tripled. Therefore the design team and CDOT team consented to a minimum global factor of safety of 1.30.

Inclinometers were installed prior to construction. Homes located above the construction site were visually surveyed and pre-existing damage was documented prior to construction. Inclinometer movement was observed in the inclinometers prior to, and during construction. The background movement was relatively small and was expected given the nature of the landslide and geologic setting. CDOT was aware of the movement of the landslide, and the wall systems were designed to best accommodate long-term minor movements of the landslide area and reduce the additional loading imposed by the wall systems on the slope. At the time of construction, no viable mitigation option was available to mitigate a landslide that extended for hundreds of feet uphill and downhill of the project site given the budget constraints and constructability issues.

**Wall and Foundation Design**

The geotechnical investigation indicated that bedrock was in excess of 200 feet vertically and horizontally in the landslide area mainly due to large scale pre-existing faulting that had affected the area. It was necessary to work with the design team to reduce the cut walls as much as practical. The uphill walls were designed as low design height soil nail walls since it was not possible to anchor tiebacks in bedrock and the bond lengths required in the soft subsurface materials would have been cost prohibitive.

The fill walls on the project were comprised of Mechanically Stabilized Earth walls. In the initial bidding phases of the project the MSE walls were designed with base reinforcement lengths in excess of 70 percent of the wall height to increase the localized bearing capacity and global stability of the wall systems in the landslide and steep slope sections. Due to the nature of the mountainous conditions and scarce availability of structural backfill materials, Yeh and Associates, Inc proposed a micropile foundation support for the MSE wall structures. The micropiles were designed to transfer the vertically loading of the MSE walls below potential global stability failure surfaces. Initially, the costs of proposing deep foundation systems in lieu of shallow foundation systems did not seem feasible, but after reviewing the costs associated with structural excavation and replacement with structural backfill, and the overall lack of an adequate staging area to store materials, the costs associated with the micropile foundation offset excavation and replacement costs to reduce the overall project costs. Figures 3 and 4 depict the preconstruction and final cross sections of the completed project.
**Figure 3 - Preconstruction cross section.**

**Figure 4 - Final Completed Cross Section of Project.**

**Construction of the Project**

The following figures depict various stages of the construction of the project.
Figure 5 - Excavation and Construction of Soil Nail Walls on Fill Side of the Project.

Figure 6 - Excavation and Construction of Soil Nail Walls on Fill side of the Project.
Figure 7 - Constructed Micropile Support Foundations of MSE Walls on Fill side of the Project.

Figure 8 - Construction of MSE Walls on Fill side of the Project.
Figure 9 - Completed Construction of MSE Walls on Fill side of the Project.

Figure 10 - Construction of Soil Nail Walls on Cut side of the Project.
Figure 11 - Complete Construction of Soil Nail Walls on Cut side of the Project.

Figure 12 - Complete Construction of Soil Nail Walls on Cut side of the Project with Overlying Draped Mesh.
Summary

The project was completed over two construction seasons for an approximate cost of $13 million for 3,100 feet of additional outer roadway width. Geologic and geotechnical assessment of the site and geo-structural designs were integral to completion of the project. Additionally, the team approach to defining the project constraints regarding the treatment of minimum global factors of safety was required to move forward with the project.

Overall, the wall systems appear to be 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.
CONSTRUCTION OF TEMPORARY SOIL-NAIL WALLS
IN COARSE-GRAINED EMBANKMENT FILL
I-17/CAREFREE HIGHWAY (SR 74) T.I.
MARICOPA COUNTY, ARIZONA

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Disclaimer

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ABSTRACT

The Interstate 17 (I-17)/Carefree Highway (SR 74) TI in Maricopa County, Arizona currently experiences heavy traffic congestion during peak periods. Considerable growth in the greater north Phoenix area surrounding the interchange, and planned future widening of I-17, demanded a new interchange be designed and constructed. TI improvements involved replacement of the existing SR 74 bridge over I-17 with a new two-span multi-lane bridge supported on spread footings. In order to maintain existing TI operations, it was required that the new bridge be built in phases, with the ultimate eastbound half constructed first, followed by demolition of the existing H-pile supported bridge and construction of the westbound portion. Excavation and construction of the abutment footings necessitated that the existing approach embankment slopes be cut back to vertical to provide sufficient space at the toe for the eastbound abutment footings. An approximately 100-foot long, maximum 28-foot high temporary soil-nail wall was constructed at each existing abutment, extending from 90 feet back of the abutment to the fore-slope above I-17. Nail drillholes were advanced into coarse-grained embankment fill comprised primarily of sands and gravels with cobbles and boulders, and into cemented native soils at the embankment base, on a 5-foot by 5-foot grid. Difficult drilling conditions required several adjustments to the drilling means and methods, and installed soil nail lengths ranged from 22 to 26 feet. Numerous verification and proof tests were performed on installed nails, and full-time inspection of all phases of wall construction was performed, from staged excavation to nail hole drilling, nail installation and grouting, placement of reinforcement, shotcreting, and nail load testing.
INTRODUCTION

The Interstate 17 (I-17)/Carefree Highway (SR 74) TI bridge project is one part of a multi-year I-17 corridor improvement effort by ADOT and the Federal Highway Administration. The $19.6 million I-17/SR 74 reconstruction project will provide a modern interchange with accommodation for additional lanes on I-17. The TI is located at Milepost (MP) 224 on I-17 approximately nine miles north of SR 101L. The TI project area extends from MP 223.4 to MP 224.4, with a total length of approximately one mile along I-17 and 1.5 miles along Carefree Highway. The interchange design consists of construction of two new loop ramps in the northern half of the intersection and new diamond ramps in all four quadrants of the intersection. The new underpass bridge will allow the ultimate I-17 configuration of five lanes plus a High Occupancy Vehicle (HOV) lane in each direction and the loop ramps. Frontage roads extending southward from the TI will be constructed under a separate project by the City of Phoenix. ADOT recognizes that I-17 serves an important role to connect the Valley and northern Arizona, and completion of this project is a major milestone in the series of projects to improve I-17 operations and capacity. In order to maintain traffic on the existing structure and TI ramps, it was required that the new bridge be built in phases, with the ultimate eastbound (south) half constructed initially, followed by demolition of the existing bridge and construction of the westbound (north) half. Carefree Highway traffic was recently moved to the initially constructed south half of the new bridge over I-17, the adjacent existing bridge demolished, and construction of the north half of the new bridge begun. The TI project is scheduled for completion in the fall of 2008.

SUBSURFACE INVESTIGATION, GEOLOGIC SETTING & GEOTECHNICAL PROFILE

Subsurface Investigation
Numerous test borings were performed at the planned bridge location. Three borings were advanced in 2004 to depths of 50 feet using a down-hole percussion (Tubex) drilling system in order to penetrate the coarse-grained materials in the subsurface. Standard penetration testing was performed at approximate five-foot increments in these borings. Six additional borings were performed within the planned TI in 2006 to depths of about 75 to 95 feet using the Tubex system. Standard penetration testing was performed at approximate 2.5-foot increments in the upper five feet of the borings, and at five-foot increments to a depth varying from 25 to 50 feet below existing site grades, and at 10-foot increments below that depth. Representative bulk samples of drill cuttings also were recovered from the Tubex borings to supplement drive samples with small recoveries. Laboratory soils testing consisted of grain-size distribution and plasticity index tests.

Regional Geologic Setting
The TI site is located on the basin floor near the northern edge of the Phoenix Basin in Maricopa County, Arizona (Figure 1). This northern portion of the basin is characterized by a series of northwest- to southeast-trending small bedrock foothills separated by small intervening shallow alluvial basins. The alluvial basins progressively thin northward and ultimately terminate to the
north at the base of the New River Mountains and also to the west on the Hieroglyphic Mountains. The immediate site area lies in Biscuit Flat, which is characterized by a gentle south-to-southwest-sloping alluvial basin floor surface between a series of northwest-trending bedrock hills, including Union Hills to the immediate north to northeast and Deem Hills a few miles to the south to southwest.

**Local Geologic Setting and Geotechnical Profile**

Geologic units exposed at the site and encountered in boreholes completed at the site consist of embankment fill for the existing bridge approaches, and two dominant alluvial deposits: an upper fine-grained unit, and a lower coarse-grained unit (Figure 2). The upper fine-grained unit is exposed throughout the existing surface of the site, except in areas of embankment fill, and was encountered in the bridge borings extending to depths of 1½ to five feet below existing or original grade. This unit is composed of alluvial fan/flat material associated with a combination of floodplain deposits of main through-draining drainages and alluvial fans, primarily derived locally from within the Biscuit Flat basin, and consists of clays derived from decomposition of the volcanic rock, with volcanic and granitic rock fragments with some local reworking and mixing of the underlying coarse-grained materials. The fine-grained material consists of a lenticular deposit of sandy clay and clayey sand with varying amounts of gravel, cobbles and boulders. These upper soils are medium to high in plasticity brown, firm to hard and weakly lime cemented.

The underlying coarse-grained unit is composed of alluvial stream channel deposits associated with an ancient meander or alignment of Skunk Creek. The coarse-grained deposit was encountered at depths below 1½ to five feet in the bridge borings. The coarse-grained deposit consists of silty sand, gravel and cobbles with considerable small boulders associated with stream channel deposits along ancient Skunk Creek, and some lenses of sandy clay, clayey sand and silty sand to sandy silt, which likely represent meandering stream channel and floodbank deposits of the same system. The upper portion of the deposit also contains considerable clay, classifies as clayey sand, gravel and cobbles, and appears to represent a transition to the cleaner underlying materials. The coarse fraction is predominantly subrounded, reflecting the fluvial depositional environment. The deposit varies from weakly lime cemented to moderately lime cemented, with localized lenses of strong lime cementation, is hard to dense, and the fine-grained portion is medium in plasticity where it contains clay and non-plastic in the cleaner zones. The existing and planned bridge spread footings are founded on these very dense to hard, moderately lime cemented coarse-grained soils.

The existing embankment fill is comprised of material from the local underlying coarse-grained unit, consisting of well-graded mixtures of silty to clayey sand and gravel with a trace to some cobbles and boulders, non-plastic to occasionally low in plasticity and firm to hard. The fill soils represent the primary material exposed in the temporary soil-nail wall cut faces, and into which the wall elements were installed. Only the lowermost few feet of the soil-nail walls encountered the native site soils beneath the fill.
Site soils at the time of the field investigation and excavation were slightly moist to moist, and measured soil moisture contents were relatively low, varying from about 3 to 20 percent (of dry weight), with the finer grained, more clayey soils having the higher values. No free groundwater was encountered during the current investigation. The regional depth to groundwater in the site area is more than 250 feet below existing grade.

Based on the subsurface geotechnical data and laboratory test data provided by AMEC, and considering guidance for selection of grout-to-soil bond stress values presented in the *FHWA Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003), the soil parameters selected for use in design of the soil nail walls are presented in Table 1 below:

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>USCS Soil Class.</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (psf)</th>
<th>Estimated Ultimate Bond Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-20 (FILL)</td>
<td>SM/GM</td>
<td>125</td>
<td>36</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>20-22.5 (FILL)</td>
<td>GC/SC</td>
<td>120</td>
<td>32</td>
<td>250</td>
<td>12</td>
</tr>
<tr>
<td>22.5-27.5 (NATIVE)</td>
<td>CH</td>
<td>115</td>
<td>30</td>
<td>500</td>
<td>8</td>
</tr>
<tr>
<td>&gt;27.5 (NATIVE)</td>
<td>SM/GM</td>
<td>125</td>
<td>36</td>
<td>200</td>
<td>20</td>
</tr>
</tbody>
</table>

Notes:  
(1) Unified Soil Classification System classification.  
(2) Allowable bond stress ($Q_{all}$) = ultimate bond stress ($Q_{ult}$)/FS, where FS = 2.0.

**CONSTRUCTION PHASING & TEMPORARY SOIL-NAIL WALLS**

The new bridge is a two-span Type VI Super (“Super VI”) AASHTO girder structure, with span lengths of 143 feet and width of about 120 feet, supported on spread-type footings (Figure 3). Excavation and construction of the abutment footings required that the existing bridge approach embankments be excavated to a vertical plane to provide sufficient space for the eastbound abutment footings (Phase I construction) and approach roadways (Figure 4). A temporary soil-nail wall, approximately 100 feet long and a maximum 28 feet in height, was constructed top-down behind each of the existing abutments, extending from about 90 feet back of the abutment to the existing fore-slope above I-17 (Figure 5). After construction of the abutment footing and wall, a temporary reinforced soil slope consisting of geogrid reinforcements and geotextile facing was constructed in front of the soil nail wall, extending from behind each abutment to the approach embankment.

Phase II of the new bridge construction includes excavation of the embankment behind the existing bridge abutments in order to remove the temporary soil-nail walls, including the shotcrete facing and reinforcing, nail bars and grout columns, except portions of the soil nails which extend below the bottom of footing elevation, which are cut-off at subgrade elevation, followed by construction of the westbound abutments.
SOIL-NAIL WALL DESIGN

General Overview
The soil nail walls were designed following the procedures presented in the FHWA Geotechnical Engineering Circular No. 7 – Soil Nail Walls (FHWA, 2003). Design analysis of the soil nail walls was performed using the Caltrans computer program SNAILWin 3.10 (Caltrans, 1999). The program analyzes internal and external (global) stability using a bi-linear failure wedge approach, which incorporates a limit equilibrium method wherein all forces are balanced and inter-slice forces are included. The selected analysis procedure incorporated factored punching shear, bond stress and yield stress (nail bar steel) values, and the allowable stress design (ASD) approach. Internal and global stability and sliding, both static and pseudo-static, were analyzed for all design cross sections.

In accordance with FHWA (2003), minimum recommended factors of safety used in design of the temporary soil nail walls using the ASD method were as follows:

| External stability (global stability {long-term} and sliding) | 1.5 (static loads) |
| External stability (global stability for excavation) | 1.2 (static loads) |
| Internal stability: |
| nail pullout resistance | 2.0 (static loads) |
| nail bar tensile strength | 1.8 (static loads) |
| Facing strength: |
| facing flexure and punching shear | 1.5 (static loads) |
| headed-stud tensile strength | 2.0 (static loads) |

For analysis of external (global) stability for the temporary excavation condition, for each design cross section, the wall excavation was carried to its full depth and the lowermost row of soil nails was removed, resulting in the lower portion of the cut being unsupported.

Design Cross Sections and Nail Layouts
The proposed layout, profiles and cross sections of the soil nail retaining walls developed by HDR, incorporating the subsurface geotechnical profile provided by AMEC, were analyzed at the maximum height, and the critical wall cross sections were identified, developed and analyzed. The critical cross sections analyzed were based on the maximum wall height. The critical cross sections analyzed are described in Table 2 below:

<table>
<thead>
<tr>
<th>TABLE 2 Critical Design Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil-Nail Retaining Wall Designation</td>
</tr>
<tr>
<td>SN-1</td>
</tr>
<tr>
<td>SN-2</td>
</tr>
</tbody>
</table>
Nail layouts analyzed included 5-foot horizontal and 4.5-feet vertical spacings. All wall sections analyzed included a vertical wall face. The ground surface above each soil nail wall was flat. The depth to the first row of nails was 2.5 feet from the top of wall. The critical design cases were utilized to develop the typical horizontal and vertical nail spacings (Figure 5).

**Nail Bar Steel**

Nail bars were sized by examining the output nail forces from the SNAILWin program. The nail bars used in the analysis of the typical wall design section were No. 9, Grade 75 deformed bars conforming to ASTM A615. The nail bar parameters used in design are presented in Table 3:

<table>
<thead>
<tr>
<th>Bar Size &amp; Type</th>
<th>Tensile Yield Stress (ksi)</th>
<th>Allowable Tensile Stress (ksi)</th>
<th>Bar Diameter (in.)</th>
<th>Steel Area (sq. in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed No. 9</td>
<td>75</td>
<td>41.3</td>
<td>1.128</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: Allowable tensile stress = 0.55* (tensile yield stress) per FHWA (2003).

**Nail Head Strength and Facing Design**

The critical failure mechanisms for a soil nail wall facing and connection system, including facing flexure and punching shear, were checked in accordance with the requirements of FHWA (2003).

The temporary wall facing consisted of 4-inch thick reinforced shotcrete, with a design 28-day compressive strength of 3,000 pounds per square inch (psi). The shotcrete reinforcement consisted of continuous 6x6 – W-4.0xW-4.0 welded wire reinforcement conforming to ASTM A82 and A185, with two No. 4 bars in the horizontal and vertical direction along the nail rows.

The horizontal (waler) bars and the vertical (bearing) bars were each 2-feet 6-inches in length, and centered on the nail head. All reinforcing steel for the wall facing was Grade 60 conforming to ASTM A615. The bearing plate assembly (one per each soil nail) consisted of an 8-inch by 8-inch square, ¾-inch thick, Grade 60 steel bearing plate with center hole.

**Other Design Considerations**

Other design considerations included wall drainage and corrosion protection. Wall drainage consisted of both drainage behind the wall and surface drainage above the wall. Drainage elements behind the temporary wall facing consisted of 18-inch wide prefabricated geocomposite drain strips with separating geotextile wall drain fabric. The geocomposite drain strips were located five feet on-center between alternating vertical rows of soil nails, and connected to four-inch diameter weep holes. The weep hole penetration at the back of the geocomposite drain strip was protected with a layer of geotextile wall drain fabric. Surface water at the crest of all soil nail retaining walls was to be directed away from the soil nail walls.
The soil nail walls were temporary and constructed in non-aggressive soils. The FHWA manual states that for temporary soil nail walls in non-aggressive soils, the soil nail grout is considered to be adequate corrosion protection for the nail bar. Therefore, additional corrosion protection measures were not provided. To ensure that the cement grout cover over the nail bar was of sufficient thickness, all soil nails were be centered in the drillhole using centralizers. Centralizers were spaced at no more than 10 feet on-center and within 1.5 feet of the top and bottom of the drillhole.

SOIL NAIL WALL CONSTRUCTION CONSIDERATIONS

Soil Nail Installation Conditions
Based on the available subsurface geotechnical profile information, it was expected that drillholes for soil nail installation would encounter materials consisting predominantly of fine-grained to coarse-grained cohesionless soils (mixtures of silt, sand and gravel). Stabilization techniques, such as casing or other techniques, were anticipated to be required to advance the soil nail drillholes and permit installation and grouting of the soil nails. In addition, there was the potential to encounter boulders and chunks of concrete and asphalt.

Other Considerations
The soils at the site are predominantly granular soils with minimum cohesion. Therefore, it was anticipated that these soils would slough when cut vertically. The walls were designed with a vertical face and required that the soil be stabilized. The stabilization methods were anticipated to include flash-coating the exposed wall face with shotcrete immediately after trimming and prior to installing the soil nails, installing the soil nails through a temporary berm, or other techniques.

VERIFICATION SOIL-NAIL INSTALLATION: DRILLING, GROUTING & TESTING

Prior to beginning construction of the soil nail wall the contractor was required to complete the construction and testing of verification soil nails on sacrificial nails. The contractor began construction of the west soil nail wall (Wall SN-1) in early July 2007, with excavation of two five-foot deep slot cuts by backhoe at the east and west ends of Wall SN-1, west of the existing I-17 OP bridge, for installation of two verification soil-nails within Row 1. The verification soil-nails were installed within the existing embankment fill (west soil-nail) or possibly the structure backfill zone just back of the abutment backwall (east soil-nail). The Row 1 verification soil-nails were located about four to five feet (at the nail head) below the grade of the existing, immediately adjacent to the SR 74 roadway.

Drilling Verification Soil-Nails
Soil-nail drill-holes were advanced using an Interoc Crawler Drill (Model AN 109B) utilizing percussion-rotary method. Initially, the drill string consisted of a seven-inch diameter hollow-stem auger (ID of 4 ¼ inches) with an eight-inch diameter open bit, with the drill rig operated in rotary-only mode (Figure 6). Typical nail inclination was 15 degrees below horizontal. Three attempts were made at the east verification nail location, and one attempt at the west location.
Auger refusal was encountered, likely on cobbles or possibly boulders, at depths of about 1½ to 3½ feet from the soil cut face. Drilling was stopped, and plans to mobilize percussion bits, drill pipe and possibly casing were formulated by the Contractor.

The second attempt to advance the east verification soil-nail drill-hole utilized a six-inch diameter button-type percussion-rotary bit and 1½-inch diameter drill pipe without casing, and the air-rotary-percussion method (Figure 7). Cobbles or boulders were encountered from depths of four to eight feet from the soil cut face, and the drill-hole was advanced to a depth of nine feet back of the cut face. However, during cleaning of the drill-hole, the drill bit seized-up at a depth of about six feet, and back-hammering in an attempt to free the bit broke the drill stem at the drive head. Repeated attempts to extract the drill pipe and bit were unsuccessful, and the drill pipe ultimately was cut-off at the soil cut face, the pipe and bit left in the ground, and the first verification nail relocated.

On the third day, the crawler drill was relocated to the west verification soil-nail location. The new drill string consisted of four-inch O.D. inner drill pipe, six-inch O.D., five-inch I.D. drill casing, a 4 ¾ inch button-type percussion-rotary drill bit, and utilized the air-rotary-percussion method (Figures 8 and 9). Drill pipe and casing sticks were each 6-foot 6-inches in length. Using this set-up, a 23-foot deep verification nail drill-hole was completed in about three hours, including extraction of the drill stem and bit.

Drilling of the west verification soil-nail was preceded by retrofitting of the four-inch O.D. drill pipe to better permit joining of the threaded sections. The 23-foot deep west verification soil-nail drill-hole was completed in about 1½ hours using the improved set-up, including extraction of the drill stem and bit.

**Grouting Verification Soil-Nails**

The soil-nail assembly consisted of a No. 10 Grade 75 thread-bar with three centralizers (approximately 11 to 12 feet on-center along the bar length) and two ½-inch diameter HDPE grout pipes duct-taped to the nail bar, including a tremie pipe extending to the nail bar tip, and a grout return pipe extending to about five feet from the bar upper end (Figure 10). The grout mix consisted of neat Portland cement (Type I-II-V) and water, mixed in a double-tub motorized skid-mounted mixer (Figure 11). The verification soil-nail was grouted in three stages. For each stage, grout was placed under nominal pressure through the tremie pipe until the top of the grout column came within five feet of the soil cut face (based on the grout return pipe response), then the grout placement stopped, the grout hose disconnected and a stick of drill casing extracted (Figure 12). The grout hose was then reattached and the process repeated. For the last grout stage, the remaining upper 13 feet of drill-hole was grouted and the casing extracted in one stage. The top of the grout column was measured at about five feet from the soil cut face at completion of grouting, and the nail bar projected about 2½ to three feet from the cut face.

Subsequent to grout initial set, the constructed dimensions of the eastern verification soil-nail included a bonded length of 14 feet and unbonded (free) length of nine feet. Shrinkage of the
grout column and wicking of excess mix water into the formation apparently was the cause of the decrease in the bonded length from the placed length of approximately 17 to 18 feet.

The west verification soil-nail, consisting of the nail bar with centralizers and twin grout pipes, was inserted and grouted in four stages. The total grouting time, including mixing, was about 45 minutes. The top of the grout column was measured at five feet from soil cut face, with nail bar projection about three feet from the face.

**Testing Verification Soil-Nails**

After the nail grout obtained the required strength, verification load tests were scheduled for the two Row 1 verification soil-nails. The load test set-up consisted of a 200-ton capacity hollow-ram hydraulic jack, electric hydraulic pump, steel bearing plate, and timber cribbing (Figure 13). Verification load tests were performed to 200 percent of the design test load, corresponding to maximum test loads of 48 and 72 kips), in order to verify the grout-to-soil bond strength for the embankment fill soils utilized in the wall design. Soil nail deformation was measured utilizing two analog dial gauges (0.001 inch graduation, one-inch throw) mounted on a tripod independently supported on the excavated bench grade. Soil-nail test data was recorded by the ADOT inspectors and input to a customized Excel spreadsheet for reduction and analysis (Figure 14). Total deformation of the verification soil-nails in embankment fill ranged from 0.30 to 0.55 inch, with non-recoverable deformation of 0.15 to 0.30 inch. In accordance with special provision requirements, a 60-minute creep test was performed at 150 percent of the design load, and soil-nail creep was checked against permissible movement criteria.

In the course of testing the first verification soil-nail, the dial gauges required re-setting multiple times due to lateral movement of the nail bar/ram/bearing plate caused by the uneven bearing conditions at the soil cut face and bending and breaking of the ends of the timber cribbing (Figure 13). Further, a dial gauge stylus “walked off” the bearing plate at one juncture, requiring repositioning and re-setting. Soil and rock debris sloughing from the cut face also resulted in upset of the gauges, requiring re-setting.

**MATERIALS VERIFICATION**

**Nail Grout**

During construction of the verification soil nails, nail grout and shotcrete were tested in accordance with the special provisions. During grouting of the eastern verification soil-nail the nail grout was sampled by ADOT inspection forces and six 2-inch diameter by 4-inch tall cylinders were cast and stored in a submerged condition in an ADOT curing drum on-site (Figure 15). During the preparation of the cylinders the grout temperature was measured at 106 degrees F, and the mixing water at 108 degrees F. Grout mixing water was obtained by the Contractor from an exposed aluminum pipeline lying on the ground surface and connected to a hydrant source some distance away from the wall site. Water temperature at the supply line tap was measured at 107 degrees F; the Contractor was notified of the unacceptably high temperature of the supply water and of the grout mix and directed to correct the situation. For
subsequent soil-nail installations, the grout mix water was transported by water truck from the source to the mixer location. Furthermore, the grout mix contained more than the proposed quantity of water per grout batch, based on conversations with the Contractor’s grout technician and measurements of the mixing tub, which indicated that additional water (quantity unknown) was added to the mix after initial mixing.

During grouting of the western verification soil-nail the ADOT inspector requested that the Contractor’s grout technician decrease the water volume in the grout mix, in order to obtain the target mix proportions of five gallons per 94-lb sack of cement. Despite the request, the grout mix consisted of approximately 36 gallons water for the five-sack mix or seven gallons per sack of cement, based on water volume in the mixing tubs. Nail grout again was sampled by ADOT inspection staff, and six 2-inch by 4-inch cylinders cast and stored in a submerged condition in the curing drum on-site.

Despite the uncertainty regarding the grout mix proportions, unconfined compression tests on the grout cylinders indicated that the grout attained the minimum required compressive strength of 1,500 psi after a three-day curing period.

**Shotcrete**

Four shotcrete test panels were made to demonstrate suitability of the proposed shotcrete mix and capability of the shotcrete nozzleman (Figure 16). One test panel was reinforced akin to the design wall facing, with welded wire reinforcement and No. 4 reinforcing bars (horizontal and vertical), and the three remaining panels were unreinforced. Coring of the shotcrete test panels was performed after about a 24-hour cure period (Figure 17); observation of the core samples indicated Grade 1 encapsulation of the reinforcement per the ACI 506.2 grading system. A total of eleven core samples were obtained by the ADOT inspectors and provided to the ADOT Materials Central Laboratory for unconfined compression testing.

**SOIL-NAIL WALL CONSTRUCTION**

The construction sequence for each row of soil-nails consisted of excavation of an approximately five-foot deep drop cut; drilling, installing and grouting of the row of soil-nails; placing the geocomposite drain strip, welded-wire reinforcement, vertical bearing bars and horizontal waler bars; placing shotcrete; embedding the bearing plates; and placing washers and nuts. Localized raveling and sloughing of coarse-grained soils exposed in the Wall SN-1 soil cut face occurred during installation of the Row 1 soil-nails (Figure 18); however, the locally unstable areas were not of sufficient extent and depth to impact the existing SR 74 travel lane. A temporary stabilizing berm was placed against the lower portion of the Row 1 face cut after soil-nail installation (Figure 19) and prior to placement of the geocomposite drain strip, facing reinforcement and shotcrete (Figure 20).

Construction of Wall SN-1 progressed in top-down fashion for the remaining soil-nail Rows 2 through 6 to reach finished grade at the toe of the wall (Figures 21 and 22). The lowermost row
of soil-nails was situated immediately above the excavated bottom-of-footing elevation for the new abutment footing (south half) (Figure 22 and 23). A verification test was performed on a Row 6 soil-nail to a maximum test load of 78 kips, in order to verify the grout-to-soil bond strength for the native soils at the base of the wall, along with a 60-minute creep test at 150 percent of design load (Figure 22). Total deformation of the Row 6 verification soil-nail was about 0.70 inch, with non-recoverable deformation of about 0.30 inch.

Construction of Wall SN-1 began on July 6, 2007 with the installation of the verification test nails. Construction was completed on August 24, 2007, for a construction duration of seven weeks. The original estimate for completion of the soil nail wall was three weeks. Construction of Wall SN-2 began on August 23, 2007 and was completed on September 18, 2007, for a construction duration of four weeks.

Proof tests were conducted on production nails. Proof tests were conducted on a minimum of five percent of the installed nails or a minimum of one test per soil nail row. Proof tests are similar to verification tests with the exception that the maximum test load is 150 percent of the design load, and the loading sequence and duration are compressed. All proof-tested soil nails passed.

**CLOSURE**

The installation of the verification nails for Wall SN-1 was very difficult and time consuming. The primary reason for the difficulty was the selection of the appropriate drilling method. Evidence of this can be seen in the construction duration of seven weeks for Wall SN-1 versus the anticipated construction duration of three weeks. Once the appropriate drilling method was selected the construction duration was reduced to four weeks for Wall SN-2, which is significantly closer to the planned duration. The failure to begin construction with the appropriate drilling technique resulted in cost overruns for construction of the soil-nail walls, and more importantly a delay in the overall construction schedule, which was very tight for the project.

**ACKNOWLEDGMENTS**

The authors gratefully acknowledge the assistance provided by Bruce Liepe, C.E.T., Chief Inspector for HDR, including construction progress photographs, and results of soil-nail verification and proof tests.
Figure 2 - Log of Test Boring B-105
Figure 3 - Bridge Elevation

Figure 4 - Temporary Soil-Nail Wall Layout

Figure 5 - Wall SN-1 Elevation View
Figure 6 - Wall SN-1 verification nail drilling (first attempt, hollow-stem auger).

Figure 7 - Wall SN-1 verification nail drilling (second attempt, uncased drill-hole).

Figure 8 - Wall SN-1 verification nail drilling (third attempt, cased drill-hole).
Figure 9 - Button-type percussion-rotary drill bit with casing.

Figure 10 - Wall SN-1 – installing verification soil-nail assembly.

Figure 11 - Soil-nail grout mixing in double-mixer.

Figure 12 - Wall SN-1 verification tub nail grouting.

Figure 13 - Wall SN-1 verification nail load testing.
Figure 14 - Verification nail load test, load vs. movement & creep test plots.
Figure 15 – Wall SN-1 grout sampling and casting of 2x4 cylinders.

Figure 16 - Shotcrete test panels

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Rock Slope Stabilization Measures at the Pali Tunnel
Route 30, Maui, Hawai‘i

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ABSTRACT

On October 15, 2006, two earthquakes with magnitudes of 6.7 and 6.0 struck in close succession just off the northwest coast of the Island of Hawai‘i. Damage caused by the earthquakes has exceeded $100 million; however no deaths and only minor injuries were reported. Numerous rockfalls and landslides occurred in road cuts, embankments and natural slopes. Because of the lack of redundancy in the highway systems of the islands, road closures due to rockfalls or landslides can have debilitating effects on the transportation systems. Most rockfall and landslide events occurred on the Big Island, and on the southeast coast of Maui.

The Hawai‘i Department of Transportation (HIDOT) is implementing a program of rock slope stabilization throughout the islands, including placement of rockfall drapes on the steep rock slopes along Route 30 on the south side of the western end of Maui (which did not suffer damage from the earthquakes). HIDOT implemented these programs long before the October 15, 2006 earthquakes. As part of on-going slope stabilization, the department elected to stabilize the rock slopes above the portals to the Pali Tunnel, located at Milepost 10.4 on Route 30. This two-lane paved roadway can have ADT counts up to 60,000 vehicles per day, as the roadway provides the most direct route for tourists from the Kahului Airport to the resorts in Lahaina, located on the western side of Maui. The roadway also serves as the primary route for emergency and commercial vehicles serving the resort areas. Therefore, HIDOT limited traffic stoppages for rock slope mitigation construction to 20 minutes, and these could only occur during the night.

Differential weathering typical of young volcanic rocks, coupled with precipitation events and ground motions induced by earthquakes, are the principal causes of rockfalls in Hawai‘i. The bedrock at the tunnel consists of thin to medium bedded Wailuku basalt flows, separated by thin to medium bedded, discontinuous beds of clinker and scoria, and thin beds of lateritic soils. Initial construction of the Pali Tunnel in 1951 did not include stabilization of the slopes above the portals, as these were fresh cuts at the time of construction. Weathering has diminished the stability of the slopes above both portals, which required light scaling prior to placement of rockfall drapes. However, the slopes above the west portal rise thousands of feet to the north, and contain large, loose boulders on the verge of toppling and rolling onto the highway. To reduce maintenance associated with preventing rockfalls from reaching the roadway, HIDOT selected an innovative, commercially-developed hybrid rockfall barrier/drape to contain rolling rocks from high on the slope. The hybrid contains elements of a 2,000 kJ rockfall barrier, with ring nets extended as a drape below the barrier. To protect traffic while construction of the hybrid was underway, a lower 1,000 kJ rockfall barrier was constructed on a lower portion of the slope. Hybrid systems have recently been implemented in Washington, California, New Jersey and Colorado, and are undergoing field testing in Italy.
INTRODUCTION

Two earthquakes and numerous aftershocks occurred off the northwest coast of the Island of Hawai‘i on October 15, 2006. These earthquakes are named the Kiholo Bay and Mahukona earthquakes, occurred 7 minutes apart at about 7:00 am local time, and had magnitudes of $M_w$ 6.7 and 6.0, respectively. The latter earthquake has been determined to not be an aftershock of the former, and the Kiholo Bay earthquake mechanism is characterized as occurring on a normal fault. The effects of both earthquakes were felt on all islands in the state, with the most intensity measured on the northwest side of the Island of Hawai‘i. Damage from ground shaking included landslides, and rockfalls in road cuts, embankments and natural slopes. Roadways, bridges and buildings were also damaged. Most damage occurred on the Island of Hawai‘i; however several roadways, rock slopes and a bridge on the east side of Maui were also damaged\(^1\).

Due to a lack of redundancy in the highway system in Hawai‘i, road closures due to rockfalls, landslides and embankment slope stability failures can have a significant effect on initial emergency response and economic recovery efforts\(^1\). The extreme topography of the volcanic mountainous terrain precludes construction of a redundant roadway system, and most roadways in Hawai‘i lie near the coasts of the islands or through the flatter valleys and plains. On the Island of Maui, several rockfalls occurred. County Highway 31 on the southeast coast at Manawainui Gulch was closed due to rockfall instability, as well as below the Kalepa and Allelele slopes southwest of Hana. Undermining of the roadway also occurred, as well as abutment erosion of a bridge at Pa‘ihi. About 500 residents in southeast Maui where affected by these rockfall events. Portions of Route 31 currently remain closed until the cliffs can be stabilized.
The Hawai‘i Department of Transportation (HIDOT) has been actively enacting a program of slope stabilization to protect highways from the effects of rockfall from earthquake or other mechanisms such as erosion and heavy rainfall events. This program includes Route 30 on the southwest coast of Maui (Figure 1). This highway forms a vital link between the airport at Kahului/Wailuku and the tourist resorts at Lahaina on the west side of Maui. This roadway, like many of Hawai‘i’s roadways, forms the only surface travel link between several areas on the island. This roadway is active 24-hours a day, and is used heavily for tourist traffic, buses for volcano bicycle rides, commercial traffic and emergency response vehicles. Much of the southern portion of the highway is characterized by deep rock cuts and steep rock slopes, and is treasured by many as a scenic and challenging roadway. The southern portion of Maui is arid and dry, being in the wind shadow of prevailing northeasterly winds. As such, rainfall is intermittent but intense in the spring, leading to thin soil cover development. The combination of steep rock slopes, intense intermittent rainfall and occasional earthquakes leads to the risk of rockfall affecting this vital highway. Due to this risk, HIDOT has been implementing a program of rockfall protection measures along Route 30, including scaling of loose rock, installation of rockfall drapes/ring nets, and rockfall barriers. The focus of this paper is the conceptual development, design and construction of a hybrid rockfall barrier/drape system HIDOT installed above the west portal of the Pali Tunnel, at Milepost 10.4 on Route 30 (Figure 2).

![Figure 2 - Aerial photo looking to the southeast of the Pali Tunnel project site prior to rockfall mitigation construction, April 2007. Note old highway grade above tunnel portal.](image)

**GEOLOGY**

Maui is a two-volcano island, with a West Volcano (smaller of the two), and an East Volcano, also known as Haleakalā. The project is located on the south coast of the West Maui volcano, within the Wailuku Basalt, the oldest volcanic strata on Maui. The Pleistocene and
Pliocene(?) aged Wailuku Basalt comprises the main mass of West Maui’s shield volcano, which includes thin-bedded flank lava flows, caldera-filling lava flows, dikes and sparse sills. Hawai‘i volcanoes are formed in four eruptive stages: presheild (subaerial phase), shield (subareial and areal), postsheild, and rejuvenated. During the tholeiitic shield phase, the eruption rate is high, with 95-98 percent of the volume of volcanic material erupted\(^3\). Wailuku Basalt is composed mainly of tholeiitic basalt, and the upper part contains alkali basalt and minor hawaiite. Radiometric age dates range from 2 to 1.3 Ma\(^2\). Fresh basalt is blue to gray in color, while eroded beds are dark gray, red, red-violet and brown\(^4\).

The project site lies within thin ‘a‘ā lava flows, which dip gently toward the ocean (moana) at dips of about 20 degrees. ‘A‘ā basalts are characterized by alternating layers and inclusions of massive, very hard and strong basalt, surrounded by various thicknesses of clinker and scoria. The ‘a‘ā lava at the site consists of porphyritic, vesicular, jointed olivine basalt, with phenocrysts of pyroxene (hypersthene), commonly two to 15 feet thick, and occurring in discontinuous, flat masses (Figure 1). The clinker and scoria consists of poorly to loosely welded, irregularly-shaped and rough-surftaced rocks ranging in size from gravel to boulders, in layers two to eight feet thick. Additionally, thin, discontinuous beds of red, orange and purple laterite occur on the upper surfaces of the basalt layers, representing baked soils developed between successive lava flows\(^1\).

![Figure 3 - ‘A‘ā massive basalt between clinker beds (geologic hammer for scale).](image)

There is a significant difference in the mechanical properties of ‘a‘ā clinker and massive basalt. Loose clinker strata are prone to raveling and differential erosion, removing support from overlying massive basalt beds. The massive basalt can form significant cantilevers and overhangs, which can lead to rockfalls when they topple\(^1\). Stressors such as earthquake motions and intense rainfall can trigger such rockfalls. Large boulders of massive basalt (up to 20 feet long) occur upslope from the west portal of the Pali Tunnel, and are precariously perched above the more erodible clinker layers (Figure 4).
ROCKFALL MITIGATION DESIGN

Following the 2006 earthquake and rockfalls, HIDOT undertook repairs and rockfall mitigation measures on Route 30 along slopes that had generated rockfalls or were a concern because of their high traffic volumes and significant corridor/emergency access designation. To date, HIDOT has protected cut slopes along the roadway with wire mesh drapes and more recently with ring net drape installations. HIDOT selected the slopes above the Pali Tunnel and the brows of the tunnel portals for remediation as part of the on-going repairs. Initial scaling proved this decision to be correct. While gaining access to the slope during inspection, rockfall occurred resulting in a rock strike on a vehicle. HIDOT prioritized repairs on the slopes and required active protection of traffic and limiting scaling hours to night work unless rockfall barriers were in place. Repairs were conducted from April to June 2007.

The subject slope represents a lobe of basalt flows between Kamaohi and Mokumana Gulches that extends down to the ocean. The old King Kamehameha Highway follows the topography and was replaced in the 1950’s with the current roadway (see Figures 2 and 5). The Pali Tunnel was constructed in 1951 and is considered a landmark and tourist attraction, as it is the only public highway tunnel on the island. Evidence of past rockfalls generated through
differential weathering of the competent basalt and clinker interbeds is present in the adjacent gulches and on the slopes above the tunnel. Many of the rocks on the slope were unstable and could be sent rolling downhill with little effort. To avoid setting the rocks on the slope in motion during scaling and installation of a drape or other barrier, a temporary rockfall barrier was considered essential to protect the heavy traffic on the roadway below. The initial concept to hold rocks in place or limit their ability to move downslope was a drape system that would extend over most of the slope. To limit the areal extent of the drape, and thus visual impacts, HIDOT selected a hybrid barrier/drape that would not need to extend as far upslope as a drape, and would still allow passage of rocks to a collection area downslope where access for maintenance vehicles was easier.

![Figure 5 - Conditions of west tunnel portal and slope prior to repairs, April 2007.](image)

A hybrid barrier is a passive rockfall protection system consisting of a flexible, woven-wire or cable fabric suspended from a horizontal top support cable raised off a slope surface by posts or suspended by anchors across a chute. Generally internal, side and bottom anchoring of the fabric is limited or eliminated to allow maximum flexibility of the system and attenuation and/or containment of rockfalls at the base of the system. Thus, hybrid barriers address rockfall source areas both underneath and upslope of the installation and control the rock’s descent under the fabric, combining the performance of standard unsecured draperies and flexible rockfall fences. Consequently, they protect more slope area with less coverage than would be required with a full drapery, can be installed higher upslope without increasing maintenance costs and can capture higher energies with less robust fence infrastructure. Because these systems have the combined benefits of effectiveness, cost savings and reduced maintenance, they have become the focus of several studies including review of existing field performance, limited rock rolling tests, and finite-element modeling.
The installation above the Pali Tunnel on Maui incorporated a lower SF100 – 1,000 kiloJoule (kJ) rockfall barrier kit manufactured by Igor Paramassi, and an upper, new SH hybrid barrier developed by Igor Paramassi, based on their successful SF200 – 2,000 kJ rockfall barrier kit. The lower 1,000 kJ rockfall barrier was installed first to protect the roadway and traffic during installation of the hybrid system. This system was installed on the outside edge of the old highway, which forms a natural rockfall collection area. The brows of the tunnel portals and slopes between the old roadway and top of the tunnel were scaled at night, and pinned ring net drapes were installed to secure the slopes above the tunnel portals and the roadway. Design of the barrier and hybrid systems involved topographic survey of the slope, measurement of rocks on the slope and rockfall bounce analyses using Rocfall (Figure 6). Rockfall simulations were calibrated to a known rockfall trajectory that led to a vehicle strike and for which the rockfall pathway could be mapped. The friction angle and coefficients of normal and tangential restitution of the bare basalt bedrock, talus, vegetated talus and pavement were varied in a sensitivity analysis to mimic the known rockfall trajectory. Barrier placement, height and overall capacity contributed to the selection of the barrier and hybrid systems. Barrier installations were checked for the construction condition and long term performance of the installations.

![Figure 6 - Rockfall bounce analysis of upper hybrid rockfall drape/barrier and lower rockfall barrier.](image)

Barrier foundations were partially in soil, and lateral resistance to accommodate loads specified by the manufacturer had to be developed using several factors including base friction, rock anchors in shear, and uphill cable anchors attached to the post base plates. Uphill anchors for the posts of the barrier were also attached horizontally upslope across the old roadway so that a small skid-steer loader could be used to collect and remove rockfall debris from the catchment zone.

Once the lower barrier was in place, construction of the upper hybrid system commenced with post and anchor installation. When the infrastructure comprising the frame and longitudinal wire ropes was in place, the remaining fabric panels and protective plastic-coated, double-twist
wire mesh to contain small rocks (smaller than the aperture of the rings in the ring net) were installed by helicopter lifts (Figure 7). Following installation, several rocks were scaled into the system as a check of the system’s effectiveness.

Figure 7 – Upper 2000 kJ hybrid barrier and lower 1000 kJ rockfall barrier.

The combination of ring nets and black plastic-coated double twist wire mesh fabric blends well with the local topography. It is uncertain whether the circular patterns in the ring nets help the fabric blend or the combination of black “screening” and circular rings facilitates blending. In either event, stakeholders have commented that the ring net systems appear to be less obtrusive than cable net or galvanized wire mesh systems.

HYBRID BARRIERS – STATE OF THE PRACTICE

Many hybrid barrier systems have been constructed in western North America to control rockfalls along highways using both lightweight and high-tensile-steel chain link fabrics, double-twisted hexagonal mesh, and more robust cable nets. Badger⁷ reported systems in Washington to be durable and highly effective in containing rockfalls generated both beneath and upslope of the installation. Observed problems included minor puncture failures near the top horizontal support rope, damage to post supports, debris accumulation above abrupt slope convexities and horizontal support ropes, and maintaining slope coverage with long narrow drapes. Based on these observations, design efforts have sought to avoid restraining the mesh, raise systems to reduce perimeter impacts, minimize vulnerable post supports, diminish effects of slope convexities, reinforce the impact area, and cautiously secure narrow systems. Similar favorable results were reported by Duffy⁸ for systems installed in California using different fabrics and infrastructure, and under varied site and loading conditions. Field testing of hybrid systems is on-going in Italy. Field tests include rolling of simulated rocks downslope into the systems, which are monitored using load cells to measure maximum impact energies, and high-speed
cameras to monitor barrier reactions. Finite-element modeling is being performed for drape and hybrid systems at Washington State University and has used component and field testing results from Italy to develop numerical analytical models for the systems. Hybrid barrier/attenuator systems have been installed in Colorado, Washington, California, Hawai`i, New Jersey, and will soon be installed in Vermont.

CONCLUSIONS

Construction of the hybrid system on Maui was completed between April and June 2007 (see Figure 8). The only impacts to traffic were single lane closures and occasional road closures up to 20 minutes between 9 pm and 6 am for one week in May when the portals were scaled above the roadway and pinned ring net mesh panels were installed. Construction of the lower rockfall barrier allowed the contractor to install the upper hybrid barrier with minimal concern for construction induced rockfalls. The lower rockfall barrier, installed for the construction condition, was left in place to act as a redundant barrier, trapping rockfalls that may exit the upper hybrid barrier system. The systems were designed and installed with periodic maintenance issues in mind, and allow for a cost effective and redundant rockfall retaining system with pleasingly aesthetic qualities.

Figure 8 - West tunnel portal and slope following installation of lower rockfall barrier, upper hybrid barrier, and pinned drape above portal, July 2007 (compare with Figure 5).

The project was constructed rapidly and successfully due to teamwork between HIDOT, their construction inspectors, the rockfall stabilization contractor, the hybrid/barrier manufacturer and the engineer. The hybrid barrier system represents a successful evolution of barrier and
drape technologies to accommodate maintenance and longevity of the systems and provide a safe travelling environment for the public.

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NEW GENERATION OF SPIRAL ROPE NETS
FOR ROCK PROTECTION -
TESTS, DESIGN, APPLICATION

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ABSTRACT

The potential risk from rockfall and small-scale instabilities is increasing. One of the reasons is that environmental influences are becoming constantly more extreme. Another reason is probably also the fact that new infrastructure construction must be adapted to increasingly difficult geological formations or that existing construction has to be adjusted to the changing traffic situation or population development. In response to this, new methods, technologies and systems are being developed to offer economic solutions that are adapted to the situation.

Critical blocks or rock masses must often be secured in overhanging rock faces, which it is not possible to clear without significant risk to people or infrastructure. Up to now, traditional wire rope nets made from stranded ropes have been used for this purpose. When these are used as active protection, these square or rectangular panels require a fixed pattern for the securing points. In addition, their resistance to selective effects and their protection against corrosion is limited.

The development of a new kind of spiral rope net enables a significant improvement in load transmission, handling during installation and corrosion protection. It is now possible to lay the spiral rope net economically in rolls and to join them together in a manner actuated by gravity. There is therefore no longer any need for time-consuming stitching of the panels together. With this protection system, the nails can be arranged irrespective of the size of the net and thus adapted to the specific requirements of the project in an optimal way. The new spiral rope net is also ideally suited for drape systems for passive rock slope protection.

In addition to standard tension tests, as well as tests for the determination of selective load transfer, corresponding model experiments were performed as a basis for the design of the new rock protection system as an active measure. This meant that for the first time, it was possible to investigate the behavior of a flexible net facing in the case of a load from broken-off blocks that have slid off between the anchors. The distribution of forces is determined around the unstable block by means of measuring the loads in the anchors depending on the deformations and deflections. It was possible to transfer the results and findings of the model experiments, including with regard to deformation behavior, directly into theoretical dimensioning models.

Implemented projects show the possibilities for use of this new generation of rock slope protection system.

INTRODUCTION

Conventional systems for stabilizing larger blocks or unstable layers of stratified rock have mainly consisted of diagonal wire rope nets so far where the panel dimensions have dictated the nail pattern. It has been difficult to adapt these systems in terms of technical and economical aspects to the problem areas due to the fixed arrangement of nails.

Compared to conventional cable net systems, the new generation of steel wire rope nets features several enhanced properties. The new design evolved around providing a flexible nail pattern and effective protection from corrosion, thus ensuring that a perfectly adaptable system would reliably and safely satisfy the requirements for the planned duration of use.
A wide-meshed spiral rope net, made of high-tensile, 4 mm in diameter steel wire has been developed for this purpose that allows the arrangement of nails or anchors in any order and thus perfectly adapts to irregular surfaces.

**Historic development**

What started with the use of simple wire mesh (e.g. for gabions, fences, etc.) has been advanced to comprise a top-quality system that is highly effective in terms of withstanding static loads.

The bearing resistance of the ordinary mesh and anchorage is limited, though. It was possible, however, to space the nails farther apart by installing an additional wide-meshed, square or rectangular cable net over the regular mesh. Since the purpose of regular mesh was not to withstand any significant static loads, plastic nets or grating were often used instead.

![Fig. 1: Left: Steep slope covered with regular mesh. Right: Steep slope stabilized by diagonal cable nets (3.3 m x 3.3 m in size with 300 mm x 300 mm mesh openings), PENTIFIX® system installed over a regular mesh](image)

The disadvantages of this method entailed the fixed length of the net which determined the nail spacing and entailed high costs, and time-consuming installation compared to simple mesh cover.

Geobrugg AG, Protection Systems, in Romanshorn (Switzerland) has consequently developed TECCO®, a high-tensile steel wire mesh featuring rhomboidal-shaped openings which permitted the ease of handling of a regular-type mesh while providing the strength of a wire rope net. This innovation has opened up new possibilities, including the:

- optimized nail pattern to permit meeting the local conditions (slope, ground, topography);
- offsetting of nails in horizontal rows to avoid the crossing of pathways in the slope line;
- pretensioning of the system against the ground to be stabilized by pressing (tightening) the spike plates against the mesh and ground.
In the process of development it became clear that the transmission of force to the nails or anchors played an important role in improving the bearing resistance of the slope stabilization system. It was for this reason that the further advancement of flexible slope stabilization systems necessitated the spike plates to be adapted and optimized in terms of size, geometrical layout and bending resistance to meet these new requirements. Originally, mostly square or round, flat steel plates had been utilized. Rhomboidal-shaped, with flange-reinforced spike plates of adequate bending resistance are meanwhile used due to the increased requirements. The slope stabilization system can thus be actively prestressed against the ground.

As a result of the development process and the experience and knowledge gained in high-tensile steel wire meshes over the years, it was possible to design a new wide-meshed spiral rope net to secure individual boulders or unstable layers of stratified rock.

**Objectives for using spiral rope nets as a protection system**

Previous slope stabilization systems prevent the sliding or breaking off of layers of stratified rock prone to erosion. These systems are frequently deployed together with proactive erosion protection measures (revegetation).

With the innovation of the wide-meshed spiral rope net, the scope of application was broadened to include defined boulders prone to break loose in an existing succession of ravines or along a specified slide plane.

It frequently is not possible to clear such boulders or secure them proactively. Consequently, only a type of protection system can be considered where the boulder would be enclosed and retained by a cable net cover.
**Definition of terms**

There are three different types of systems used to protect steep slopes made up of loose or solid rocks. They are distinguished according to their different bearing properties and capacities:

The term **flexible slope stabilization system** (flexible facing) describes slope stabilization measures that employ meshes or netting that exert a calculable and verifiable retention force. These are open systems that can handle static loads which, depending on the ground conditions, allow for revegetation or, if the ground is prone to erosion or decomposition, require such.

**Soft systems** (soft facing) in comparison refer to simple draperies that provide protection from erosion; systems that are not designed to handle static loads.

A **hard system** (hard facing) is the third type of system and it includes shotcrete or concrete structures. The bearing properties feature a stability that is significantly different from the stability of flexible mesh covers. Hard systems are more susceptible to superficial movements of the subsoil which is one of the reasons why nails have to be spaced closer and why higher material strengths are required.
THE SPIRAL ROPE NET INNOVATION

**System elements**

The innovative rock protection system has been developed by Geobrugg AG and in essence consists of the following elements: SPIDER® spiral rope net, nailing, spike plates, shackles, boundary ropes, spiral rope anchors, secondary mesh (optional) and intermediate fixations.

![Image of SPIDER® rock protection system](image1)

The SPIDER® spiral rope net features a rhomboidal-shaped mesh with openings 500 x 292 mm in size (dimensional tolerance: +/- 5%). The spiral rope used for this application consists of three twisted together high-tensile steel wires, each 4 mm in diameter, with a yield stress of at least 1,770 N/mm². Similar to the TECCO® high-tensile steel wire mesh, this spiral rope is first crisscrossed to form the spiral shape and then twisted together to form a net. The ends of the spiral cables are tied to one another to permit the full transmission of force to the adjoining panels. Basic protection from corrosion consists of a coating of 95% zinc and 5% aluminum. The spiral rope net can also be made of stainless steel if exacting requirements concerning the protection from corrosion have to be met. The basic dimensions of the net rolls are 3.5 x 20 m; one roll weighs approx. 190 kg.

![Diagram of system elements](image2)
Commercially available GEWI or TITAN nails can be used for fixing the net cover, which has to fulfill the static requirements. Raw nails are usually used and grouted with at least 20 mm of mortar. With permanent protection measures, an allowance for corrosion of 4.0 mm in reference to the static nail diameter is often taken into account.

Contrary to earlier cable net covers where so-called ear heads were utilized for fastening the cable nets to the nails, a system of rhomboidal-shaped spike plates of type P33 are now used with which the spiral rope net can be simply tensioned against the ground. The geometrical layout, size and bending resistance have been optimized based on various puncturing and bending tests and adapted to the system requirements. For the force-locked connection of the net panels, 3/8“ shackles are used normally. The loss due to overlapping is kept to a minimum.

In order to achieve an ideal load transfer in adjoining areas and to reinforce the boundaries, boundary ropes, 14 mm in diameter, should be used all the way around, and they should be braced against the spiral rope anchors laterally. The boundary ropes can be pulled directly through the mesh openings from the top, bottom or sides. Seam ropes, boundary shackles or compression claws to attach the net to the boundary ropes are thus not needed. The shackles may be fixed with glue to prevent possible vandalism. In the event of overhangs, it may be wise to attach additional cables under the overhangs to optimize the bearing behaviour of the system.

As an option it is possible to install a secondary steel wire mesh underneath the spiral rope net if there is a risk of rocks coming loose that might fall through the mesh openings. Intermediate fixations is often provided to ensure the protective measure will be adequately braced against the ground. A simple spike plate will do the job.

**Bearing resistance of the system**

Extensive tensile tests have been conducted with the SPIDER® spiral rope net under the supervision of the LGA Nuremberg. The bearing resistance to tensile stress in the main bearing direction of SPIDER® S4-230 is 220 kN/m. The bearing resistance to a localized force on the area around the knot is 60 kN. This value is an important datum for the design of the protective system to secure rocks from coming loose and sliding off.

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Fig. 4: Left: Standard test for the determination of tensile strength per running meter. Right: Determination of the bearing resistance to local force transmission.
Application areas

The SPIDER® rock protection system has been designed to secure rock slopes where the ground is hardly to insignificantly prone to decomposition, where the surface is irregular and where rocks that come loose tend to be large. There are currently two concepts regarding the potential risks and maintenance requirements:

- Concept (I):
  If the endangered area is to be secured in a proactive manner and the maintenance work is to be kept to a minimum, it would be wise to utilize all-round nailing in combination with a net cover system of spike plates. The type and arrangement of nails as well as its lengths are to be adapted to meet the requirements for static loads.

- Concept (II):
  Should it not be possible to drill through the critical areas or should the requirements regarding deformation and maintenance be less than mentioned under (I), then the nails could be arranged around the critical area (e.g. around the unstable boulder). The protective measure in this instance is rather passive. Larger deformations must be anticipated should pieces of rocks or even a mass come loose under the protection of the net drapery. (II) can be applied to limited areas only.

The application areas involve three different cases (Fig. 5):

(A) Protection of single boulders
(B) Protection of a mass of rock, rock pile
(C) Reinforcement of a system that is (too) weak
Advantages compared to ordinary cable nets

The advancement of ordinary and customary cable nets to spiral rope net application has yielded several advantages for the client, project manager and the contractor. Compared to ordinary cable net systems such as the PENTIFIX® system, which works with individual 3.3 x 3.3 m panels, the SPIDER® spiral rope net is delivered in 3.5 x 20 m rolls. Rather than spending the time sewing together the individual panels, two shackles per meter are used to connect the panels in a force-locked manner and in no time flat. The reduced number and the optimized quality of joints permits an efficient installation of the spiral rope net.

In terms of static loads, one of the decisive advantages lies in the fact that the arrangement of nails no longer depends on the size of the individual cable net panels, but can be perfectly adapted to match the project-specific requirements. The advanced system of spike plates ensures that the system is braced against the ground as securely as possible.

The corrosion protection was further enhanced by no longer needing cross-clips and by using the significantly larger wire diameter of 4 mm, compared to the 0.9 mm diameter of twisted wire, in conjunction with an aluminum/zinc coating.

Fig. 5: Application areas (A), (B) and (C)

Fig. 6: Left: Ordinary cable net system with cross-clip. Middle: Link of SPIDER® spiral rope strands with steel wires of diameter 4.0 mm. Right: Ordinary wire rope with its individual wires of diameter of about 0.9 mm.
Fig. 7: 1:1 endurance test near the coast in the north of Spain. Left: steel wires of diameter 3 – 4 mm 95% Zn / 5% Al coated (GEOBRUGG SUPERCOATING®). Right: 100% Zn coated cable nets with wire diameter of about 0.9 mm

EXAMPLES OF IMPLEMENTED PROJECTS

The following projects present some examples of how the SPIDER® rock protection system has been used as active measure in Europe.

Fig. 8: Left: Taubenloch Canyon near Biel, Switzerland, protection of overhang above a hiking trail with secondary mesh. Right: Test site of the Eidgenössischen Forschungsanstalt für Wald, Schnee und Landschaft (WSL) (Swiss Research Institute for Forest, Snow and Landscape) in Lochezen quarry, Walenstadt, Switzerland. Protection of various up to 12 cubic meter large boulders
CONSIDERATIONS ON THEORETICAL ASPECTS OF THE STATIC SYSTEM

General considerations

The possibility of failure of a block-shaped boulder or a layer of stratified rock from a given succession of ravines is presented in the figure below. The retention forces required to prevent a boulder from toppling are relatively low, the event of a rockslide, however, produces significantly greater forces and stresses, which the following sets out to examine more closely.
Fig. 10: Schematic failure mechanisms of block-shaped boulders or a layer of stratified rock

**Design approach**

In order to secure an individual boulder at risk of coming loose, a external stabilizing force (P) is required that acts to hold the boulder against the stable ground.

This force depends predominantly on the:

- dead weight (G) of the block-shaped boulder
- inclination of the sliding surface to horizontal (β)
- friction angle (φ) between the stable ground and the block
- cohesion (c) or interlocking along the slide plane and its size A
- direction (θ_o) and (θ_u) of the forces (Z_o) and (Z_u) in the net cover
Fig. 11: Retention forces based on stabilization considerations
The retaining force (P) can be calculated as follows based on taking into account the stabilization issues relevant to an individual block-shaped boulder as well as the model uncertainty correction value $\gamma_{\text{mod}}$.

$$P \ [\text{kN}] = \frac{G \times \gamma_{\text{mod}} \times \sin \beta \times \cos \beta \times \tan \varphi - c \times A}{\gamma_{\text{mod}} \times \cos (\beta - \omega) + \sin (\beta - \omega) \times \tan \varphi} \ [1]$$

The vector $\vec{P}$ applied in a two-dimensional model can be divided into the vectors $\vec{Z}_o$ and $\vec{Z}_u$ which will be transferred from the net onto the nails or anchors and therewith into the stable subsoil. The direction ($\omega$) of the force (P) to horizontal (upwards = positive) or the relation factor $\eta$, respectively, depends on various factors such as the interlocking action and/or friction between the surface of the block and the net restraint, the surface irregularities / roughness of the block, and more.

$$\vec{P} = \vec{Z}_o + \vec{Z}_u \ [2]$$

$$Z_u \ [\text{kN}] = \eta \times Z_o \ [3]$$

$$Z_o \ [\text{kN}] = \frac{P}{\cos (\theta_o - \omega) + \eta \times \cos (\theta_o + \omega)} \ [4]$$

The stronger the interlocking action between the net cover and the boulder is, the more favorable is the direction of action of the resultant force (P) and thus the smaller is the tensile force on the lower restraint. In general, the force at the lower restraint is always smaller or in maximum equal than the force at the upper restraint thus.

Fig. 12: Vectors of $\vec{P}$, $\vec{Z}_o$ und $\vec{Z}_u$, opening angles $\varnothing = \varnothing_o + \varnothing_u$ and its influence on $P$

The forces ($Z_o$) and ($Z_u$) significantly depend on its orientations to each other. If the opening angle ($\varnothing = \varnothing_o + \varnothing_u$) tends towards 180 degrees, the forces ($Z_o$) and ($Z_u$) tends theoretically towards infinite then keeping (P) constant and $\eta \neq 0$. The figure on the right hand side as well as the following diagram clarifies this. The
arrangement of the spiral rope net on the slope plays thus an important role in securing a block-shaped boulder.

The diagram below shows that the forces in the upper restraint \((Z_0)\) can rise disproportionately with an increasing opening angle \((\vartheta)\). \((Z_0)\) can even become a multiple value of the boulder weight \((G)\). The influence of the friction angle \((\varphi_G)\) between the boulder and the restraint is insignificant in comparison.

![Diagram showing retention force of the upper anchor \((Z_0)\) dependent on the opening angle \((\vartheta)\), with \(\varphi = 30^\circ, \beta = 60^\circ\).](image)

**Fig. 13:** Retention force of the upper anchor \((Z_0)\) dependent on the opening angle \((\vartheta)\), with \(\varphi = 30^\circ, \beta = 60^\circ\)

Since the rock protection system features a certain degree of elasticity, it is unavoidable for the boulder to be displaced in the slide face in the event of a failure. The stress on the restraint is reduced as a result of this boulder movement. The opening angle \((\vartheta = \vartheta_0 + \vartheta_u)\) becomes narrower with an increasing displacement and the upper and lower retention forces consequently decrease. Fig. 14 shows the qualitative presentation of the parameter interdependence.
MODEL EXPERIMENTS

Objectives

The model experiments for the protection of a boulder with the SPIDER® system were conducted on a scale of 1:3.5. Objectives included the implementation of the theoretical basic considerations described in the previous chapter, the comparison under real-life conditions, and the determination of the distribution of forces in a three-dimensional system.

Test setup

The test setup basically consisted of a blue steel frame to which the rope and the model net was fastened and a slide face red colored in between. The frame was 1.5 m wide and 2.5 m long. The angle between the slide face and the frame was kept constant at 36°. Strain gauges were used to measure the forces acting on the rope, net and directly on the sliding body. A potentiometer was used to measure the displacement of the block-shaped boulder. A wooden block that weighed 58 kg (= 570 N) was used as a sliding body.

Tests on static and kinetic friction

The purpose of the first series of tests was to determine the static and kinetic friction of the block on the painted red slide face (steel plate). The slope of the slide face was raised until the block started to slide. The block was fixated to a cable in the second phase. The slope was increased from initially 25° to 40°, 45° and 50° and the retention force required to stabilize the block was determined in...
parallel to the slide face. Then the cable was undone, so that the block could smoothly slide down, whereby the resultant forces acting on the cable were measured. The results of both tests indicated a friction angle range from $33^\circ$ to $36^\circ$, whereby no significant discrepancy was indicated between the static and kinetic friction.

**Model experiment with cable restraint**

The purpose of this series of tests was to examine the congruence with the theoretical two-dimensional model. The forces acting on the cable leading upwards ($Z_0$) and leading downwards ($Z_u$) and the respective angles in relation to the slide plane were measured for three different inclinations of the slide plane ($\beta = 40^\circ$, $45^\circ$ and $50^\circ$).

![Fig. 16: Left: Schematic presentation of model experiment with cable restraint. Right: Test results with an inclination of the sliding plane of 50 degrees to horizontal.](image)

Fig. 16 depicts the test setup for the model experiment with cable restraint and shows a diagram of the test results with an inclination of $50^\circ$ of the slide face to horizontal, as an example. At the beginning of the test, the block was kept in place in a distance of about 210 mm away from the rope by the force ($R$). When let smoothly sliding the block into the cable, the force ($R$) decreased to zero and the forces in the rope increased correspondingly.

The cable was relatively tense and thus the deformation was small until the cable held the block. This resulted in a wide opening angle ($\theta$) from $172^\circ$ to $174^\circ$ in the direction of the restraint leading upwards or downwards, which according to the theoretical model results in great forces. Even with a flat slide face angled at $40^\circ$, this force resulted in a retention force of 44% of the block weight, according to the experiments, and angled at $50^\circ$ to more than 100%.
The friction angles derived from the retention forces were slightly above the effective friction angle of the slide face, which might possibly be attributed to the fact that restraining effects tend to increase the friction slightly.

The influence of friction between the net and the block was relatively insignificant which is congruent with the findings of the theoretical model.

**Model experiment with net cover**

Next to one restraint at the top as well as one at the bottom, two restraints on each side, one at 75% of the frame height and the other at 40% of the frame height (viewed from bottom), held the model net in place. In addition of measuring the forces in the slope line ($Z_o$) and ($Z_u$) and laterally ($S_1$) and ($S_2$), also the corresponding angles ($\theta$) and ($\delta$) were measured.

![Model experiment with net cover](image)

**Fig. 17:** Left: Schematic presentation of model experiment with net cover. Right: Test results with an inclination of the sliding plane of 45 degrees to horizontal.

Fig. 17 depicts the test setup, the force measurement arrangement and the corresponding directions of the net restraint (top, bottom, sides). On the right-hand side, the corresponding loads in the restraints are presented for a test with an inclination of the sliding plane of 45 degrees.

The net is less tense than the cable and thus, until an equilibrium is established, the displacement is greater and consequently the opening angle ($\theta$) is narrower (from approx. 149° - 155°) towards the main direction (in slope line), which instantly affects the retention forces. The retention forces still range from 14% to 30% of the block weight, depending on the inclination of the slide face. The influence of the lateral restraints is taken into account with 22% - 25%. If this percentage of lateral forces were missing, the longitudinal retention forces would be greater accordingly and would make up approx. 18% to 38% of the block weight.
Since the angle of the lateral restraints to horizontal ($\delta$) is relatively small, with 10.0° to 13° at the bottom and 2° at the top, forces of up to 40% of the upper retention force ($Z_o$) are being mobilized in the lower restraint ($S_1$).

The friction angles derived from the retention forces are slightly above the effective friction angle of the slide face. This tendency increases the steeper the slide faces are and thus, the retention forces are also greater, which can be attributed to restraining effects which increase the friction slightly.

In a further test serie with a similar test setup described just above, the lateral lower restraints ($S_1$) were moved down all the way, and the lateral upper restraints ($S_2$) were positioned at approx. 50% of the frame height. Unlike to first arrangement restraints with the net cover, greater forces ($S_2$) were measured in the lateral upper restraint, which achieved approx. 55% of the longitudinal upper retention force ($Z_o$). The proportion of lateral forces rises accordingly for the effective retention force ($P$) to almost 30% with an increasing slope of the slide face. This exerts a relatively small influence on the size of the upper retention force compared to the block weight. The friction angles derived from the retention forces lie within the range of the effective friction angle of the slide face and thus are somewhat lower than with the first arrangement, which could possibly be attributed to the slightly different pressing force of the block on the substructure (direction of the retention force).

**Findings from model experiments**

The forces calculated by means of the two-dimensional model were in general congruent with those measured as a result of the experiments. For the purpose of practical application this means that the two-dimensional model can be applied with satisfactory accuracy to determine the forces in the main direction.

The opening angle ($\delta$) from the directions of the restraint in longitudinal direction exerts the decisive influence on the forces.

The influence of friction between the net and the block is small in comparison.

The influence of the lateral restraints on the retention force depends on the position of the restraint and the forces themselves depend on the angle ($\delta$) from the directions of the lateral restraints to horizontal.

Based on the model experiments, the factor ($\eta$) can be determined whereby further model tests are aimed at as a wider base.

**PROCEDURE FOR DIMENSIONING**

First, the most important input quantities have to be determined:

- Weight, geometrical dimension of the block-shaped boulder
- Inclination of the sliding surface ($\beta$)
Shear parameters along the sliding surface (friction angle and possibly cohesion)
- Angle of the net restraint to horizontal (\(\theta_o\)) on top of the boulder
- Angle of the net restraint to horizontal (\(\theta_u\)) at the bottom of the boulder
- Angle of the lateral net restraint to horizontal (\(\delta\))
- Accelerations due to earthquake horizontal (\(\varepsilon_h\)) and vertical (\(\varepsilon_v\))

These input quantities are applied in the described formulae to calculate the retention force on top (\(Z_o\)) and at the bottom (\(Z_u\)) of the boulder to be protected, whereby the geotechnical parameters are to be reduced by means of the typically applicable partial safety coefficients. In addition, a coefficient of model uncertainty (\(\gamma_{mod}\)) can be introduced for the model. The influence of kinetic friction between the net and the block-shaped boulder is usually negligible.

The experiments conducted on models so far allow the following qualitative conclusions in terms of the distribution of forces. These conclusions will have to be refined by means of different anchorage arrangements and by utilizing different block-shaped boulders.

- The friction between the net and the block-shaped boulder can increase the calculated upper retention force by 10% - 20% and reduce the lower retention force accordingly.
  \[ \eta = 0.80 - 1.00 \]
- The influence of the lateral retention forces may reduce the longitudinal retention forces by approx. 15% - 30%.
- The lateral retention forces may exceed 50% of the upper retention force, depending on the arrangement and deflection of the net in the restrained section.
  \[ \nu = \frac{S}{Z_o} = 0.30 - 0.50 \quad \text{if} \quad \delta \geq 5^\circ, \quad S = \sum_{i=1}^{n} S_i, \quad n = \text{number of lateral restraints on one side} \]

As an example, the following proof of bearing safety of the net to local force transmission on the top of a boulder have to be fulfilled. The proof of bearing safety of the net to local transmission at the bottom as well as laterally can be done correspondingly. The number of nails (\(n_0\)), its strength and length needs to be chosen or adjusted in that way, the occurring forces can be safely transferred from the net via the nails into the stable subsoil.

**Proof of bearing safety of the net to local force transmission on top of the boulder**

Maximum retention force in the net on top of boulder, on dimensioning level: \(Z_{od} \ [kN] = \ldots\)
Bearing resistance of the net to local force transmission longitudinally: \(Z_R \ [kN] = 60\)
Resistance correction value for local force transmission in line of slope: \(\gamma_{ZR} \ [-] = 1.50\)
Dimensioning value of the bearing resistance of the net to local force transmission on top:

Number of static relevant nails or anchors on top of the boulder:

Total bearing resistance of the net to force transmission on top:

Proof of bearing safety:

\[ Z_{Rd} = \frac{Z_R}{\gamma_Z} \text{ [kN]} \]

\[ n_o [-] = \ldots \]

\[ Z_{Rd,tot} = Z_{Rd} \times n_o \text{ [kN]} \]

\[ Z_{od} \leq Z_{Rd,tot} \]
CONCLUSION AND FUTURE WORK

Spiral rope nets, fabricated like mesh cover, provide new possibilities for securing unstable boulders prone to come loose on steep slopes due to their high longitudinal and transverse tensile strength and their high knot strength, which is important if the anchorage is subjected to a point force.

The forces measured in model experiments scaled 1 : 3.5 in real-life scenarios were congruent with the results derived from the simple two-dimensional theoretical model.

The model experiments also provided data relevant to the distribution of forces on the upper, lower and lateral anchorage and restraints. The upper retention force calculated by applying the two-dimensional model served as the measuring quantity, which was primarily dependent on the opening angle (θ) of the directions of the upper and lower restraints. The flatter the net rests on the surface, the wider the angle and the greater the forces in the net itself and the cable restraints. The friction between the net and the boulder exerts only a secondary influence on the forces and thus is usually negligible.

Depending on the arrangement, the lateral retention forces achieve up to or even more than 50% of the upper retention forces (Z₀) required according to calculations. The upper retention forces (Z₀) can well exceed the weight of the boulder to be secured in a situation with steep slide faces and wide opening angles. Deformation within the context of boulder movement decreases the opening angle and reduces the effective forces, which relieves the system. A state of sliding hence does not result in an increase of forces leading to failure.

The presented results from the model experiments are the beginning of a series that is to be continued to determine the influence of further restraint arrangements including the insertion of boundary cables, and also the influence of different block-shaped boulders and different slides faces (friction values). The findings from these model experiments shall also be compared with the results gained from completed objects.
SUGGESTIONS FOR FURTHER READING


CLOSE QUARTERS ROCKCUT STABILIZATION

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INTRODUCTION

The project site, located in an industrial section of Bergen County, New Jersey, was cut into a sedimentary ridge that defines the western boundary of former glacial lake Hackensack, which formed during the last glacial retreat about 30,000 years ago; at the base of the ridge, fill overlies relatively thick sequences of varved clay and dense glacial till before sedimentary rocks of the Brunswick Formation are encountered at depths greater than 100 feet. These same rocks outcrop in the ridge along the west side of the site.

In order to accommodate a planned 300,000 ft$^2$ warehouse and office complex, relatively deep cuts in the rock ridge were required; residential properties line the top of the ridge. The result of excavation was a north trending, east facing 70° to 80° cut varying from 80 feet in height at the far south end to 50 feet at the far north end. A 34-foot-wide access easement exists between the curb at the toe of the cut and the warehouse. A 30 foot wide service road provides access to property to the north of the warehouse; within this road are situated nine-foot-wide parking spaces adjacent to the warehouse. The distance from the toe of the cut to the curb ranges from about 2 to 4 feet, providing virtually no catchment area for rockfall. A 4 foot sidewalk abuts the warehouse.

BACKGROUND

Limited subsurface geotechnical investigation was performed prior to site development because of topographic and space constraints; two borings were drilled on residential properties at the top of the ridge, however, no rock mass characterization (i.e., collection of planar data or Rock Mass Rating) was performed. The initial geotechnical engineering report indicated the rock mass could support 70° cuts (i.e., 6V:1H), and that a qualified geotechnical engineer should inspect the cut to make any supplemental recommendations. During mass excavation by mechanical means a 120 cubic yard section of rock fell from the lower reaches of the cut following an intense rain fall event; this event undermined a 40-foot-long section of stronger rock by as much as six (6) feet. At that time, the developer’s inspecting geotechnical engineer recommended rock bolts be installed in the cut face to provide resistance to rockfall, and that steel netting be installed to restrain small blocks and shale spall. Evidently steel dowels were installed in lieu of rock bolts, but steel netting was installed to mantle the entire cut face. The dowels consisted of No. 6, 8 and 11 steel reinforcing bars that were grouted with minimum embedment lengths of 4 to 8 feet, and were left protruding from the rock face roughly 6 to 8 inches.
About four years after site development was completed, the property transferred ownership. Maintenance records indicated localized periodic rock fall occurred after completion of the developer’s stabilization and protection measures. The dowelling appeared to be insufficient as shale was observed to have spalled around the dowel, exposing as much as 3 feet of the steel elements. The steel netting yielded mixed results; some rockfall was restrained, other rockfall tore through the netting and spilled over the curb. At that time, the current property owners sought a solution. Our work included detailed inspection of the subject slope, rock mass characterization, and development of a stabilization remedy.

**INVESTIGATION**

We reviewed available information, including aerial photographs, topographic maps, local and regional geologic maps, the initial site geotechnical engineering study, other geologic references, site details and development plans, and the developer’s stabilization and rockfall protection plan. Subsequently, we developed and implemented a field investigation program to characterize the rock mass; we performed a detailed line survey of the 1,400 ft long rock cut. In addition, we observed and measured spall rate of the shale and monitored rockfall events as a means of predicting standup time of different sections of the rock cut.

**Rock Mass Characterization**

Horizontal control along the toe of the rock cut was established by a series of marker stations (0+00 to 14+00) painted at 50 foot intervals on the curbing, beginning at the south end of the property and extending northward for approximately 1,400 feet. The cut face is slightly irregular with an orientation of 205°. The warehouse is situated directly opposite the cut between stations 2+20 and 12+68. A standard 25-ft. fiberglass surveyor's rod was utilized to measure vertical distances on the rock face. For greater heights, a weighted tape measure was lowered from a telescoping boom lift to the curb. Orientation measurements (i.e., strike and dip) of pervasive bedding, joints and/or fractures (i.e., structural discontinuities) in the rock were collected using a standard Brunton compass. A composite photographic mosaic of the entire rock cut was produced and used to aid in the analysis of the exposure. At each station a scaled cross-section of the rock cut was generated, indicating the lithology of each successive rock layer and the orientation and dip of structural discontinuities.

**Stratum**

The rock mass can be divided into an Upper Zone and a Lower Zone based upon the predominance of sedimentary rock types observed in each zone. The Upper Zone is characterized by 1 to 2 foot thick fine to medium grained, blocky sandstone layers interbedded with 8 to 10 inch thick layers of shale and fine sandy siltstone. The Lower
Zone consists predominantly of finely laminated siltstones, shaly siltstones and shales with occasional layers of fine-grained sandstone and sandy siltstones. The shales and shaly siltstones of the Lower Zone are fissile; that is they tend to split or disintegrate on exposed surfaces subject to the weathering effects of the atmosphere. Overall, the Upper Zone is more resistant to weathering than the Lower Zone. A 2-foot-thick buff colored sandstone bed forms the lowest unit within the Upper Zone. The height above curb to the buff sandstone bed is approximately 34 feet at the south end of the site and approximately 22 feet at the north end. The Lower Zone gradually thins to the north, and its character begins to change north of Station 10+25, where occasional thin lenses of quartz pebble sandstone were observed. Although the rock remains fissile, the rock face appears to be more competent north of station 10+25, as individual rock layers coarsen in texture from shaly to fine sandy siltstone.

Distinct structural discontinuities were observed in the Lower Zone; see photo. The discontinuities terminate beneath the buff sandstone member of the Upper Zone. The structural discontinuity data indicated two structural trends; each having a potential for general raveling failure.

**Rockfall Events, Erosion, and Maintenance**

During our inspections, disintegrating shale fragments were observed “raining” down the slope from various heights, but primarily within the Lower Zone along its entire length. Accumulations of talus had formed at the toe of the cut and at different rates along the entire cut. Some talus had spilled onto the service road, and tabular-shaped blocks of shale and siltstone ranging from one to three feet in maximum dimension were observed between the slope and netting at various locations.
Isolated 2 to 3 cubic yard (cy) rock falls from within the Lower Zone occurred between stations 9+00 and 9+40 during our initial observation period. The netting was damaged, but the rockfall was restrained behind the curb. In addition, upstream storm water was observed causing erosion of soil and rock in this same vicinity. This section of the rock cut was also the area of greatest spall accumulation. The rockfall and talus that had accumulated at the toe of the slope were periodically removed and the netting was repaired.

**EVALUATION**

Weathering and rockfall were occurring by two mechanisms; both associated with water. Water was observed seeping out of bedding beneath the buff sandstone bed at the contact between the two zones, and also along discontinuities at several locations. Water percolating through structural discontinuities reduces frictional resistance. When ambient temperatures drop below the freezing point, water in joints near the surface expands upon freezing thus widening the aperture causing jacking.

The disintegration (spall) of the shale is caused by repeated exposure to freeze/thaw cycles; water trapped in the interstices on the shale surface begins to expand as the air temperature drops below the freezing point of water but cannot solidify. This condition creates enormous osmotic pressure between the individual grains in the rock mass. Subsequent rise in air temperature releases this stress; this cyclic stressing and unloading causes the shale to break apart.

Initial spall monitoring indicated spall volume was greatest between Stations 5+02 and 11+38, indicating the rock is generally more durable both to the north and south. A review of climatic data indicated that precipitation had been below normal during the typically colder months, while temperatures were above normal. Since it was observed that rockfall would occur, as expected, after periods of above normal precipitation and below normal temperature, greater amounts of rockfall and spall and increased frequency could occur during prolonged more severe periods.

The durability of the shale was controlled by changes in moisture content (freeze/thaw, wet/dry, hot/cold) and stress release. Daily, monthly, annual and cyclic variations in temp/precipitation result in constant stresses that weaken the shale, resulting in disintegration of the rock into gravel, as well as causing failure of rock blocks. The northeast slope aspect is uninsulated and subject to maximum variations.

The variable composition of the Lower Zone, combined with unfavorable orientation of certain discontinuities resulted in a rock slope with variable competency. Thus, differential rates of ravelling and weathering were to be expected. The majority of the talus and larger rockfalls observed accumulating at the toe of the slope originated from the Lower Zone.
Continued rockfall from the Lower Zone was expected to result in undermining the Upper Zone, a condition that existed between stations 9+00 and 9+40 during site development. Over time, such overhanging zones were expected to become unstable.

Several solutions were identified and evaluated to determine an appropriate solution for the ongoing erosion of the slope. The objective was to minimize the exposure of the Lower Zone to the atmosphere and provide confining pressure to control weathering and raveling, all with minimal impact to the warehouse operations. Because of the lack of catchment area at the toe of the cut and the undermined conditions, the section of rock cut between Stations 5+02 to 11+38 were targeted for immediate stabilization. The prefer solution was one that can be constructed between the toe of the slope and the curb line.

**STABILIZATION**

A soldier pile and pre-cast concrete lagging retaining wall backfilled with clean stone was constructed between the toe of the cut and the curb between stations 5+02 and 11+38 to address undermined sections of the slope in need of immediate stabilization. Steel soldier piles were socketed 5 ft into rock on 6 vertical to 1 horizontal batter at 12 ft spacing. A leveling course of concrete was constructed between piles to provide a level base for the pre-cast concrete panels. Water collected in the backfill stone was allowed to drain through weep holes in the leveling course. A 1-3/8 inch double corrosion protected steel rock anchor was grouted into the rock behind each soldier pile; the design load of 145 kips was locked off on a 2 ft by 2 ft steel plate bearing against the shale; see photo. A similar sized steel bar was then connected to the anchor and extended through the soldier pile. Precast concrete panels 4 feet in height were inserted between each set of adjacent piles to heights of 24 to 32 feet; see photo. One inch diameter trap rock was used to backfill the wall to apply a small confining pressure against the rock. Concrete closure panels were constructed at each end of the wall.
Post-Construction Monitoring

The cut was subsequently monitored over a 3-year period to verify our initial findings related to standup time, and to determine where additional stabilization would be needed. In particular, the section of rock between Stations 3+02 and 4+10 experienced the greatest increase in spall rates (267%). This section of slope was predicted to be the next section of cut requiring stabilization and was subsequently stabilized.
POLYURETHANE RESIN (PUR) INJECTION FOR ROCK MASS AND STRUCTURE STABILIZATION

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ABSTRACT

The Federal Lands Highway Division (FLH), FHWA, is currently investigating the application of polyurethane resin (PUR) injection as a rapidly deployed, cost-effective ground stabilization measure providing superior stabilization performance, while achieving aesthetics objectives. Most recently, in cooperation with the Colorado Department of Transportation (CDOT), FLH completed full-scale PUR demonstration projects at a historic tunnel located along SH 14 in the scenic Poudre Canyon west of Ft. Collins, CO, and at a dry-laid stone masonry wall supporting SH 149 along the Rio Grande River west of South Fork, CO. The Poudre Canyon demonstration involved the “gluing” of a previously bolted section of the western tunnel portal where annual freeze/thaw cycles and rock mass creep toward the adjacent Cache La Poudre River were contributing to rock mass instability. The South Fork demonstration involved PUR injection within a highly-porous, actively failing and culturally-sensitive dry-laid stone masonry wall – a type of retaining structure commonly encountered throughout federal park and forest lands. Based on these investigations, application guidance is being developed for the selection of polyurethane resin products and injection methods when (1) stabilizing failing groundmasses (e.g., rock slopes, unique rock promontories, escarpments), and (2) preserving aging and/or deteriorating man-made structures (e.g., historic retaining walls, archeologic structures).

INTRODUCTION

The Federal Lands Highway Division (FLH) of FHWA is responsible for the construction and rehabilitation of scenic roadways in America’s most environmentally and culturally sensitive settings. As good stewards of U.S. public lands roadway projects, preservation of unique natural features and historic and archeologic structures is central to the FLH “Lightly on the Land” construction philosophy. To further support preservation of our public lands resources, FLH has sought, through its Technology Deployment Program, ground stabilization technologies that…

(1) Provide superior stabilization and preservation of natural, archeologic, and historic structures subject to environmental and roadway construction damage;
(2) Produce aesthetically pleasing results in context-sensitive settings (particularly technologies that are virtually invisible to the public); and
(3) Provide cost-effective alternatives to traditional blasting-scaling-bolting operations – which are often expensive, time consuming, environmentally invasive, publicly adverse, and which may result in less-than-desirable constructed/excavated features requiring follow-on aesthetic treatment.
Polyurethane resin (PUR) injection, or “rock gluing”, a long-established method for rapidly stabilizing weak, actively failing ground in the underground mining industry, is one such technology readily transferable to FLH highway projects (sample shown in Fig. 1).

Fig. 1. Rock fragments permanently bonded within hardened polyurethane resin (PUR).

This simple, two-part, polymer resin is easily transported and stored, readily pumped into fractured rock and/or porous manmade structures, provides superior stabilization/sealing with very short set and cure times, is environmentally friendly when set, and results in aesthetically pleasing site conditions. Although technology transfer to the civil transportation sector has been slow compared to more conventional ground stabilization methods (e.g. rock bolting, cementitious grout), this technology becomes quite cost-effective when addressing the aesthetic requirements common to FLH roadway projects – where external rock and structure rehabilitation fixtures cannot be tolerated, and where applications cover relatively confined, limited areas.

The FLH Technology Deployment Program is currently investigating and documenting applications of the PUR technology as a rapidly deployed, cost-effective ground stabilization measure providing superior stabilization performance, while achieving aesthetics objectives. During the summer of 2006, in cooperation with the Colorado Department of Transportation (CDOT), FLH completed a full-scale PUR demonstration project at a historic tunnel located along SH 14 in the scenic Poudre Canyon west of Ft. Collins, CO (Fig. 2). The demonstration involved the “gluing” of a previously bolted section of the western tunnel portal where annual freeze/thaw cycles and rock mass creep toward the adjacent Cache La Poudre River were contributing to portal instability. Over the course of six days, a three-man crew working out of a lift drilled sixteen, 10-12 ft deep holes above the western portal and outboard tunnel abutment, through which 5,000 lbs of PUR were successfully injected. The PUR infused throughout the
rock mass, evidenced by small amounts of resin dripping from surface joints and fractures, effectively stabilizing and sealing the portal area.

Fig. 2. PUR injection at the west portal of the Poudre Canyon Tunnel, along the Cache La Poudre River on SH 14.

In addition to the Poudre Canyon demonstration, FLH has recently completed a second PUR injection demonstration involving the stabilization of a culturally-sensitive, dry-laid stone masonry retaining wall supporting SH 149 west of South Fork, CO, adjacent to the Rio Grande River (Fig.3). Whereas the Poudre Canyon Tunnel demonstration involved PUR injection throughout a relatively large volume of moderately jointed and fractured rock, the South Fork retaining wall project focused on evaluating injection methods within a highly porous, highly unstable structure. Of particular interest to this investigation was whether PUR could be successfully delivered to target zones within the wall mass, if resin could be pumped without further damaging the wall or initiating failure, and if PUR outflows along the face could be effectively managed to minimize required cleanup and aesthetics impacts.
Fig.3  Drilling prior to PUR injection behind a dry-laid stone masonry retaining wall along SH 149 west of South Fork, CO.

Although it was not possible to implement full-scale performance/proof testing at these demonstration sites, qualitative observations coupled with years of rock mass stabilization experience in the underground mining industry suggest significant gains in rock mass and structure stabilization were achieved. Both demonstrations resulted in a number of “lessons learned” which will serve as guidance for future applications on FLH projects.

POLYMER PRODUCTS AND PUR APPLICATIONS

Although polyurethane resins encompass a broad spectrum of product specifications, they represent a fraction of the even broader range of polymer products available for sealing, bonding, stabilizing, and consolidating porous materials. With this in mind, the FLH Technology Deployment Program has focused on those product specifications most suitable for rock mass and historic structure stabilization – paying particular attention to product performance attributes and operating constraints, system delivery methods, product cost, potential environmental impacts, and the technical benefits compared to more traditional stabilization options. In light of program findings to date, this section provides a brief overview of polymer products, specific attributes of polyurethane resins deemed beneficial to rock and structure stabilization, the range of current applications in the civil and mining industries, a comparison with traditional cementitious grout applications, and an overview of potential environmental issues.
Overview of Polymer Products

There exist literally tens of thousands of different mix designs comprising the family of polymer products inclusive of polyurethanes (PU), polyurethane resins (PUR), and epoxy resins (EP). Although sometimes difficult to distinguish one product from another, as terminology is often used interchangeably to describe these products, they can be broadly defined by several key characteristics: density, strength, reactivity with water, expansion/elongation, shrinkage, number of mixing stages, and relative product cost. Of these characteristics, water interaction is of principal interest when selecting the proper polyurethane product for ground/structure stabilization – a fact illustrated by the demonstration projects described later in this paper.

The following briefly overviews the aforementioned characteristics of PU, PUR and EP products. Due to the wide range of products available, application and material property information should be obtained from suppliers and carefully considered prior to final product selection.

Polyurethane (PU). Polyurethanes are extremely versatile plastics, and are found in a variety of forms: flexible or rigid foams, solid elastomers (or rubbers), coatings, adhesives and sealants. Although generally considered thermoset plastics, those that permanently harden upon heating/curing, there are grades of polyurethane elastomers that are thermoplastic – softening upon heating and then hardening once cooled without appreciable change in chemical composition.

As with all urethanes, foams are produced when reacting two principal components, polyols and isocyanates. In practice, this product can be stored fully mixed and injected in a single-stage process, greatly simplifying PU application. PU reaction set-times can be varied from as little as 15 seconds to several hours, depending on accelerant additives. Table 1 lists additional relative PU properties when compared to PUR and EP products.

Table 1. Relative properties of polyurethane foams (FHWA, 2007).
PU’s are generally considered to be hydrophillic, aggressively interacting with water to foam upwards of 3,000% of the original volume, and may elongate as much as 500%. PU’s can also shrink in excess of 10% if allowed to thoroughly dry. Because this polymer type incorporates water within its chemical structure, PU will shrink and swell indefinitely with groundwater fluctuations. In addition, as density decreases with product expansion, shear strength also greatly decreases. For these reasons, PU’s are typically used in water sealing applications, and not relied upon for high adhesion strength or groundmass consolidation in high load settings. However, when PU is injected under confined conditions significant expansion pressures can be generated extending the use of this product to a variety of structure jacking applications.

**Polyurethane Resin (PUR).** Polyurethane resins differ from polyurethane foams primarily in terms of their strength, two-stage mixing requirements, and reactivity with water. PUR is significantly stronger than PU, attaining compressive/tensile strengths exceeding weak-to-moderate intact rock strengths while exhibiting very high adhesion. In fact, removing this product from most rock surfaces following initial set typically requires hammering or grinding, often taking a veneer of the rock in the process. As a result of the high adhesion strength of this “glue” product, PUR has been used to stabilize failing groundmasses in underground mining environments throughout the U.S. since the mid-1970’s.

PUR generally consists of a two-stage, 1:1 mixing/injection system. “A” and “B” components, each with a viscosity similar to a light oil, are pumped in separate lines until the point of injection, where mixing is facilitated by spiral inserts within the injection nozzle. Reaction set times vary from less than a minute to several hours, and are greatly influence by line and ground temperature. For example, the PUR product used during the demonstrations described later in this paper had an effective working injection temperature range from 50°F to approximately 95°F, resulting in set times ranging from several minutes to 15-20 seconds, respectively. Although initial set times can be very quick, with 90% strength achieved in less than 1 hr, full cure is commonly specified at 24-48 hours. It is, therefore, important to carefully consider the application environment when selecting an appropriate PUR product to avoid the need for heating or cooling injection lines and to ensure proper resin set.

PUR is often described as a hydrophobic polymer material, but many products do nonetheless foam in the presence of water (Fig. 4). Water-induced expansion is much less than the aforementioned PU products, generally ranging from 25% to 250%. The moderate hydrophillic nature of PUR aides in the uptake of this product within finely fractured rock masses whenever moisture is present. In wet settings, significant foaming occurs with an associated loss in density and compressive, tensile and shear strength. However, the water-activated product sets as a stiff foam with moderate adhesion making it a good application when void filling and rock mass consolidation require a stronger product than the highly-expansive PU foams. In dry conditions, PUR sets as a hard, dense resin, much like an epoxy glue, exhibiting excellent bond strengths.
Fig. 4. Non-foamed (brown resin) and foamed (light tan) PUR material.

Due to the two-part transport and delivery system requirements, PUR is somewhat more expensive than PU foaming products. Average total costs for PUR injection in rock masses (including the retaining wall study described in this paper) range from 4$/lb to 7$/lb depending on site access, drilling constraints, traffic control requirements, and clean-up requirements. In view of the cost per unit volume of ground treated, particularly in dry conditions where very little resin expansion may occur, consideration should be given to the potential for filling large voids on a project before selecting PUR as the primary rock mass stabilization product.

Like most polymeric plastics, PUR is highly reactive to ultraviolet radiation. Although this property does not affect the performance of the product confined within a rock mass or similar structure, it is beneficial in expediting the weathering of surficial spillage and injection overruns. Even though overruns are largely removed at the time of injection (and most easily removed prior to resin set), coatings and thin veneers of resin are often left behind on exposed surfaces. Within a few months, these final remnants of the injection project are often fully weathered and no longer visible. Table 2 lists PUR properties when compared to PU and EP products.

**Epoxy Resin (EP).** Epoxy resin products are similar to PUR in terms of strength and product delivery methods, yet exhibit no shrinkage or expansion in the presence of water – a true hydrophobic polymer material. As a result, EP products do not as readily permeate finely fractured rock masses, having to displace water during injection. A dense, non-expanding product, EP is by far the most expensive of the adhesive polymers considered for rock mass applications, limiting usage to low-volume applications requiring a high-strength, non-foaming, high-adhesion resin glue (properties given in Table 3).
Polyurethane Resins (PUR)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Injection Type</td>
<td>Grouts</td>
</tr>
<tr>
<td>Density</td>
<td>20 to 70 pcf</td>
</tr>
<tr>
<td>Comp./Tensile Strength</td>
<td>15 to 20,000 psi</td>
</tr>
<tr>
<td>Component Mixing</td>
<td>Generally Two-Stage</td>
</tr>
<tr>
<td>Injection Pressure</td>
<td>100 to 3,000 psi</td>
</tr>
<tr>
<td>Expansion</td>
<td>25% to 200%</td>
</tr>
<tr>
<td>Elongation</td>
<td>10% to 25%</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>0% to 3%</td>
</tr>
<tr>
<td>UV Reactivity</td>
<td>High</td>
</tr>
<tr>
<td>Relative Cost</td>
<td>Medium to High</td>
</tr>
<tr>
<td>Water Reactivity</td>
<td>Hydrophobic/Hydrophillic</td>
</tr>
</tbody>
</table>

Table 3. Relative properties of epoxy resins (FHWA, 2007).

Epoxy Resins (EP)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Injection Type</td>
<td>Grouts</td>
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<tr>
<td>Density</td>
<td>5 to 60 pcf</td>
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<tr>
<td>Comp./Tensile Strength</td>
<td>5,000 to 20,000 psi</td>
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<tr>
<td>Component Mixing</td>
<td>Two-Stage</td>
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<tr>
<td>Injection Pressure</td>
<td>30 to 800 psi</td>
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<tr>
<td>Expansion</td>
<td>Minimal</td>
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<tr>
<td>Elongation</td>
<td>Minimal</td>
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<tr>
<td>Shrinkage</td>
<td>Minimal</td>
</tr>
<tr>
<td>UV Reactivity</td>
<td>Moderate</td>
</tr>
<tr>
<td>Relative Cost</td>
<td>High</td>
</tr>
<tr>
<td>Water Reactivity</td>
<td>None</td>
</tr>
</tbody>
</table>

Civil and Mining PUR Applications

Polyurethane foaming products and epoxy resins have long been used in the civil construction industry. General applications include crack sealing, establishing water/gas barriers, void filling, structure jacking and material bonding. More specifically, the following types of examples may be found in use today:

- **PU Spray-On Membranes:** Spray-on polymers have been used successfully in the tunneling industry as both temporary and permanent measures to support loose, raveling ground. Comparisons to conventional shotcrete applications indicate that spray-on polymers exhibit 2 to 10 times the tensile strength of shotcrete at half the application thickness. Although not
commonly used in place of shotcrete, thin coatings of spray-on polymers are often used immediately following excavation in soft ground to control both rock ravel and water seepage prior to initial girder-mesh-shotcrete support.

- **PU Void Filling:** Due to the aggressive hydrophillic nature of polyurethane products, PU is often used to fill suspected or known voids behind permanent tunnel lining systems and foundations – particularly those involving water seepage. The PU foams and sets quickly, minimizing product loss in flowing water conditions and quickly sealing seeping voids.

- **PU Subgrade Improvement/Slab Jacking:** Two-component, highly expansive PU products have been used extensively in the U.S. to fill voids beneath pavement and to raise slabs to correct joint faulting and/or slab settlement. This one of the more common uses of PU in the civil industry today.

- **EP Structural Foundation Sealing and Repair:** EP has long been used in the U.S. to repair and seal cracked structural foundations where a low-viscosity, high-strength product is required in relatively small volumes of application. These applications generally do not involve significant water seepage, and do not require void filling or structure consolidation.

Polyurethane resin products have been used in the U.S. since at least the mid-1970’s; however, their application has largely been limited to stabilizing weak and failing ground masses within the underground mining industry. Several million pounds of PUR are injected annually in U.S. underground coal mining operations alone – serving to reinforce, consolidate and seal large volumes of overhead rock. For many of the largest U.S. mining companies, PUR injection has become a staple technology for rehabilitating critical roof fall areas, stabilizing weak roof strata during longwall ground support recovery operations, stabilizing/sealing geologic anomalies (e.g., fault and shear zones, ancient sand channels), and managing/mitigating water inflows (Fig. 5). In all cases, successful PUR applications in mining are dependent on carefully considering several key attributes of the setting:

- **Site Accessibility:** Site access considers geometric constraints, required progression of PUR injection, and the potential need for primary and/or supplemental ground support installation.

- **Presence/Absence of Groundwater:** Groundwater inflows may require the use of a hydrophillic PUR product for rock mass sealing; however, consideration should be given to the potential for creating hydrostatic heads sufficient to destabilize the rock mass. Minor groundwater conditions lend themselves to hydrophobic/mildly-hydrophillic PUR products with greater installed densities and strength.

- **Rock Mass Permeability:** The location, extent and character of rock mass discontinuities and bedding planes determines how far the resin will transport through the rock mass, what volume of PUR may be needed, and what extent drilling may be required to ensure resin permeates critical support zones. For example, PUR may readily travel along bed separations in delaminating sedimentary rock masses, but may not migrate throughout layered strata without extensive cross-measure drilling.

- **Air/Rock Temperature:** Rock temperatures are relatively stable and within operating ranges for PUR injection in most underground operations. However, air temperatures within the mine can fluctuate greatly depending on the time of year and mine ventilation requirements.
Fig. 5. PUR injected into coal mine roof and rib to stabilize failing ground conditions (note small-diameter injection rod extending from corner of opening).

Typically, PUR is pumped under fairly high pressures in underground settings to minimize drilling requirements, expedite PUR installation in time-sensitive settings, and ensure migration throughout the rock mass within 10-20 ft of the injection hole. The low viscosity of many PUR products allows permeation through crack apertures as narrow as 0.04 mm. Staged pumping allows filling of larger discontinuities first, with latter stages permeating the finer fractures.

**PUR Versus Cementitious Grout**

PUR and cementitious grouts are best compared on the basis of density, viscosity, strength, set-up time, and installed cost:

- **Density:** Polymer products can be customized to achieve a much broader range of installed densities than cement or silica grouts. However, predicting and controlling resin expansion in variable moisture conditions with mildly-hydrophilic products is difficult. This is an important consideration when attempting to stabilize failing structures that cannot withstand even small deformations associated with PUR expansion.

- **Viscosity:** Polymer products generally have much lower viscosities than cement or silica grouts, allowing permeation into fine fractures. Fast set times are used to constrain PUR migration from the injection point, and staged pumping is used to direct the product throughout the rock mass.

- **Strength:** The strength of fully cured cement grouts ranges from 2,500 to 5,000 psi; silica grouts are substantially weaker, ranging in strength from 100 to 1,000 psi. Conversely, PUR strengths typically range from 10,000 to 20,000 psi, with much higher bond adhesion strengths.
Set-Up Time: Initial set for cement and silica grouts ranges from hours to days, whereas polyurethane resins can be customized to set within seconds to several minutes – generally achieving 90% strength in about an hour. PUR’s are temperature sensitive, with large fluctuations resulting in widely varying set times. Care must be taken to ensure line and ground temperatures are within the manufacturers specifications.

Installed Cost: Generally, cement and silica grout installed costs are substantially cheaper than PUR per unit volume ($15-$30/cuft installed for cement grouts versus $120-$150/cuft installed for PUR). However, equal volumes of these products may not be applied to a given setting to achieve the same results. For example, large voids in a dry-laid retaining structure may be filled with a low-strength cement grout to help consolidate the rock mass. In dry conditions, a much smaller volume of PUR may be injected to coat the internal rock structure and increase bond at rock contacts without filling an appreciable portion of the void volume. In this case, the installed cost may be similar, but greater strength gains may be realized with the PUR.

In general, cement/silica grouts are used where high-volume, low-to-moderate strength, lower cost grouting is required. PUR’s are generally more applicable when high-strength, lower volume, broader transport, and faster set time conditions are warranted.

Environmental Issues

PUR products, in the thermoset cured form, are generally inert and chemically stable, and are commonly used in potable water containment and food preparation/storage applications. However, the isocyanate component and solvents used to control set times in the polyol resin component possess varying degrees of toxicity depending on mix formulation, and may contribute pollutants to groundwater in their component form. In general, both components are considered mildly to moderately toxic, and are easily containable on project sites within clearly labeled 55-gal drums connected to a closed pumping system.

Some resin mixtures are highly flammable both before and after set. Although most applications are well protected within natural rock or man-made structures, FLH projects have given consideration to the effects forest fires may have on near-surface PU slab-jacking installations.

As previously mentioned, ultraviolet light (UV) degradation does impact polymer products. There are currently no environmental pollutant concerns identified with UV degradation of cured PUR. In practice, very small quantities of inert PUR surficial overrun (in a cured thermoset plastic state) are left to degrade within the environment, ultimately resulting in a non-visible application with no environmental impact. Biodegradation from microbial or fungal attack has also been documented in instances involving specific polyester-based PUR products.

PUR Demonstration Projects

As previously mentioned, over the past couple of years FLH has undertaken two cooperative PUR demonstration projects with the Colorado Department of Transportation (CDOT). The first
was conducted at a historic tunnel located along SH 14 in Poudre Canyon, just west of Ft. Collins, CO. This site was selected due to its similarity to traditional mining PUR applications (jointed rock mass injection), and because it represented a historic rock mass structure that might easily be found within the domain of an FLH partner agency (e.g., National Park Service, U.S. Forest Service). The second project involved the stabilization of a culturally-sensitive, potentially historic, dry-laid stone masonry wall supporting SH 149 just west of South Fork, CO, along the Rio Grande River valley. This site was selected in response to numerous requests from FLH partner agencies regarding the ability of PUR to stabilize historic and/or archeologic structures. In fact, stabilization of historic retaining wall assets may well turn out to be the major application of PUR injection within the FLH roads program.

**Poudre Canyon Tunnel**

In June 2006, FLH demonstrated the application of PUR injection for rock mass stabilization within the western portal of the Poudre Canyon Tunnel, located on SH 14 along the scenic Cache La Poudre River in northern Colorado (Fig. 6). The tunnel is a very short, 75-ft-long, drill-and-blast, two-lane rectangular excavation through a vertically foliated gneiss and metabasalt. Widely spaced random jointing occurs within the rock mass; however, discontinuities and foliation are favorably aligned relative to the tunnel drivage, requiring no artificial support or lining within the tunnel. However, the vertical foliation does create freeze/thaw rockfall problems at either portal (foliation-defined rock “plates” peel from above the portal), requiring the implementation of a spot bolting program within the overlying western portal rock mass and along the outboard portal abutment in 2001 (Fig. 7). It was felt that this test location would greatly benefit from additional ground reinforcement and fracture sealing, and would be somewhat protected from injection-induced rockfall by the existing tension-bolt installations.
FLH procured PUR injection services from Micon Mining, Grand Junction, CO. Micon is the leading provider of PUR injection services to the underground mining industry, and has over 30 years experience with resin injection and rock mass stabilization in a wide range of rock types and application settings. Their RokLok 70 PUR product was selected based on its strength, viscosity, mild-hydrophillic nature, and broad operating temperature range. Table 4 lists some of the pertinent physical properties of the RokLok 70 product.

Table 4. Properties of Micon RokLok 70 polyurethane resin.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Set Time</td>
<td>2 min.</td>
</tr>
<tr>
<td>90% Strength</td>
<td>1 hr.</td>
</tr>
<tr>
<td>Full Cure</td>
<td>48 hrs.</td>
</tr>
<tr>
<td>Density</td>
<td>70 pcf</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>10,200 psi (viscous yield)</td>
</tr>
<tr>
<td>Compressive Modulus</td>
<td>92,000 psi</td>
</tr>
<tr>
<td>Flexural Strength</td>
<td>10,900 psi</td>
</tr>
<tr>
<td>Flexural Modulus</td>
<td>313,000 psi</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>3,850 psi</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>530 psi</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>7,100 psi</td>
</tr>
<tr>
<td>% Elongation</td>
<td>~17%</td>
</tr>
</tbody>
</table>

Fig. 7. Close-up of the foliation joint-defined blocks above the western portal.
Pertinent details and findings of the project include the following:

- Sixteen 1.5-in-diameter holes were drilled 10-12 ft deep within the portal outboard abutment (bounded by the Cache La Poudre River) and overlying rock mass (Fig. 8). Drilling and PUR injection was completed in six working days.

![Fig. 8. PUR injection hole locations in the west portal of the Poudre Canyon Tunnel.](image)

- All drilling was accomplished with a pneumatic rotary-percussive, hand-operated jackleg drill, operated from a man-lift. Holes were generally completed in 20-25 minutes, resulting in minimal traffic delays.
- All holes were injected within 24-hours following drilling to eliminate the possibility for cross-contaminating pre-drilled holes, and allowing hole-by-hole results to dictate the ultimate injection layout.
- 200 to 700 lbs of PUR was injected in each hole, for a total of 5,000 lbs of PUR used on the project. Each 55-gallon barrel contains 500 lbs of component product, therefore requiring approximately 12 total barrels of A/B components to complete the job (Fig. 8).
- Approximately 850 sqft of portal area was treated to an estimated average depth of 10 ft, for a total approximate treated volume of 8,500 cuft (~0.75 lbs/cuft of rock mass treated).
- Coupled, 3-ft-long hollow injection rods, with a short packer/mixing assembly attached at the resin delivery end, were inserted to within a few feet of the back of the hole (approximate 6-8 ft depth). Packers were generally seated fairly tightly during installation, but can
accommodate up to 2-in-diameter holes during pumping, if needed. The innermost rod and attached packer assembly were resin-anchored within the hole by the conclusion of the injection process, and were abandoned in the hole by disconnecting at the coupler.

![Component “A” and “B” barrels and two-sided pump.](image)

- Relatively small volumes were pumped (1-4 gpm) under low pressure (<50 psi) until PUR overrun was observed. Pumping was then suspended for approximately 1 minute, allowing the PUR to begin to set prior to resuming pumping. Staging the pumping in this manner allows cracks to be sealed, thereby pushing the next volume of PUR delivered along other fracture and joint paths.
- Work progressed from bottom-to-top. Initial PUR injection would flow down through the rock mass until the rapid set effectively sealed the lower portion of the rock mass. Continued pumping would then cause the PUR to work its way upward within the rock mass above the installation hole (Fig. 9). In most cases, PUR migration was confined to an approximate 4-8 ft radius around the installation hole. However, more persistent discontinuities with wide apertures could easily convey resin 10-15 ft prior to initial set.
- A majority of the rock mass discontinuities appeared to be filled with hard, non-expanded, dense resin. Foamed resin was seen coming from rock mass discontinuities located near the overlying slope surface and beneath slope vegetation – areas with higher moisture contents (Fig. 10).
- Despite the volume of resin pumped within the portal area, no rockfall occurred during or following PUR injection as a result of injection pressures or resin expansion in wet zones.
- Traffic was stopped during all drilling and injection operations, with average delays running about 30 minutes. Vehicles were kept well back from the injection operation to avoid fine
PUR “strands” occasionally squeezing from fine cracks during pumping from landing on and permanently affixing to car exteriors.

Fig. 9. Typical migration of PUR injection from below the injection point, upwards through the rock mass. Note that some of the resin is foaming due to moisture in the surface fractures.

Fig. 10. Hard, dense, high-strength resin fully filling major rock mass discontinuity.

- No significant overruns were encountered. Cleanup involved rapidly peeling PUR drips and runs from the rock mass prior to set, or chipping hardened overruns from the rock surface with hand tools (Fig. 11). Injection holes were completed with dark-colored grout. A few
months after the project was completed it was nearly impossible to see that any work had been done at the site.

- The total cost of the project, less traffic control provided by CDOT, was ~$42K, or about $8 per installed lb of PUR.

![Image](image.jpg)

**Fig. 11.** Overruns are relatively easy to remove if tackled before initial set is complete.

No verification drilling was conducted to determine what level of volumetric coverage may have been attained or the nature of the resin product within discontinuities (hard resin or foamed resin). Resin set time tests on rock samples at the site, coupled with visual observation of the progression of the resin throughout the rock mass (and out several of the supposedly fully-grouted bolt installation holes) indicated that a substantial volume of the rock mass was securely reinforced. This empirical performance assessment was sufficient for CDOT to recommend the use of this product on other state highway projects during the summer of 2007.

**South Fork Retaining Wall**

In September 2007, FLH evaluated the potential application of PUR injection for stabilizing dry-laid stone masonry retaining walls. As previously noted, this particular type of wall construction is common within the managed lands of FLH partner agencies, representing nearly 25% of all retaining walls found in U.S. National Parks. Unlike typical rock mass applications, non-mortared rock retaining walls are highly porous, generally ranging from 5% to 30% void space depending on the size of stone placed in the structure, degree of masonry performed, and the overall quality of construction. The non-uniform, high void character of these structures can significantly complicate planned PUR delivery within targeted wall volumes. These decades-old
structures, many of which are in serious disrepair and/or varying states of failure, are also highly sensitive to injection pressures, potentially limiting the use of hydrophillic resin in wet environments. In addition, the often culturally-sensitive nature of these structures further requires that evidence of repair be kept to a minimum – placing considerable emphasis on managing PUR overruns and cleanup. These and other factors combine to make this application far more challenging than traditional rock mass injection, requiring vigilant project management and inspection.

The South Fork demonstration project involved a short section of an approximate 600-ft-long dry-laid stone masonry wall presumed to have been constructed approximately 60 years ago. The wall varies in height from 3-12 ft and is in serious disrepair, indicated by localized failed sections (repaired with timber lagging and gabions), rotating/bulging sections, missing foundation elements, and settlement/piping cavities along the top of the wall. Several years ago, in an effort to forestall eminent wall failure, approximately 300 ft of the eastern section of the wall was reinforced with an “A-frame” micropile installation drilled along the back of the structure and a shotcrete, mesh and tie-back system installed along the face. The PUR demonstration project focused on an equally unstable, approximate 60-ft-long section of the dry-laid wall immediately west of the micropile section (Fig. 12). This wall section ranges in height from 6-12 ft and is in a state of pending major failure evidenced by wall face rotation/bulging (approaching negative batter) and numerous sinkholes/depressions just behind the top of the wall.

Fig. 12. Looking west along the test section.
Micon Mining was again retained to provide PUR injection services, and the RokLok 70 product used at the previously described Poudre Canyon Tunnel demonstration was once again selected for its strength and mild hydrophilic properties. Pertinent details and findings of the project include the following:

- Injection work began along the top of the wall, sequentially injecting several holes drilled with a 3-in-diameter auger and cased with 2-in ID PVC casing. Holes were advanced on 5-ft centers to the estimated bottom of the wall (8-12 ft), 3-5 ft behind the wall face. Little or no wall rock was encountered during drilling, suggesting wall construction consisted of a near-uniform-thickness coursing of roughly masoned stones (as opposed to more conventional trapezoidal gravity wall construction techniques). The auger method resulted in oversized holes, requiring a crude annulus packer of rags and PUR be formed near the collar of the hole to contain resin during injection. The weight of the drill rig, down-pressure on the auger and drilling vibrations combined to seriously distort the upper wall rock courses. This approach was quickly abandoned to avoid distressing the already unstable wall prior to injection.

- PUR injection began at the site following several days of intermittent rain and periods of steady drizzle. As a result, resin injected to the back toe of the wall foamed substantially, fully filling voids in the lower wall structure within 2-4 ft of the injection hole (Fig. 13). Staged pumping (1-2 gpm at <25 psi) resulted in the upward migration of PUR into the wall mass, similar to the manner in which PUR migrated through the rock mass at the Poudre Canyon site. However, once the lower wall voids were filled, PUR expansion due to high moisture in the wall created sufficient back-pressure to literally jack the wall out from the injection hole. Minor wall deformations were observed, and in one instance half-moon cracking developed at the top of the wall radiating several feet out from the injection hole and parallel to the face. This prompted a different approach to injection management.

Fig. 13. Foamed PUR pouring from the wall toe during injection.
Small-diameter hollow injection “jam” rods were then manually driven on intervening 5-ft centers within 3 ft of the wall face to an approximate mid-wall-height depth (Fig. 14). PUR injection proceeded as before, with steady, small volumes injected over the course of several minutes. PUR flowed down through the wall mass, first appearing in the face at the wall foundation. Continued pumping filled the back of the wall up to the estimated rod tip depth, at which time pumping was stopped to avoid over-pressuring the wall. This approach allowed fast insertion of the injection rods (~5 minutes each), delivered PUR to targeted zones within the wall, and improved injection pressure management in the wet conditions.

The upper 3-5 ft of wall was then injected by simply hand-placing of the injection rod within the openings between capstones. PUR flowed downward several feet before setting and causing subsequent pumping to flow out the face. This work was done one day later when the upper facing stones were mostly dry, so very little resin foaming occurred. Visual inspection indicated that the dense resin actually coated the interior rock surfaces and rock-on-rock contact points, rather than fill the voids. This method resulted in minor overruns through the face which can be easily removed during injection.

Injection directly into the face was also evaluated using a short 18-in injection “gun”. This method can very quickly deliver resin throughout the wall mass, but resulted in significant face drips and overruns as the injection gun was moved from one placement to the next. Improvements to the injection tooling could overcome much of this problem (Fig. 15).

Fig. 14. “Jam” rod being driven just behind settlement zone. This method was fast and sufficiently tight to inhibit resin from traveling up the outside of the rod.
Fig. 15. PUR overrun experienced during face injection.

- Over the course of three days, 60 feet of wall, averaging 9 ft in height, was injected with 4,000 lbs of PUR. It is estimated that approximately 2,000 cuft of wall structure was treated. Of this volume, approximately 400 cuft was void space. 60 cuft of non-foamed resin was delivered, likely filling somewhere between 20-25% of the wall void volume (Fig. 16).

Fig. 16. Wall interior showing foamed/non-foamed resin coverage.
• Confirmation core drilling confirmed PUR void filling in the back of the wall. Follow-on geophysical investigations, including 3-D seismic tomography and ground penetrating radar surveys before and after PUR injection, are still pending results and will be described in the FHWA final project report.
• Wall cleanup required vigilance during resin injection to quickly locate and remove PUR overruns, to the extent possible. The hard, non-foamed resin could be seen as drips, runs and small areal coatings over a significant portion of the wall face. It is anticipated that this material will quickly weather away due to the strong southern exposure of the wall face. The foamed PUR was easier to remove, but left a visual impact along the wall where it fully filled face voids. Overall, the PUR overruns are only visible when standing directly in front of the wall. No signs of the injection program can be seen from below the wall along the Rio Grande River or from nearby pedestrian visual access points.
• Based on the lessons learned during the demonstration, this section of wall could have been treated in less than two days – with work progressing at about 5 ft/hr. The total cost of the project, less traffic control provided by CDOT, was ~$32K, or about $6.50/lb installed.

Again, no performance testing was conducted to confirm the strength gains provided by the injected resin. However, post-injection core drilling conducted immediately behind the wall face did not distort the upper rock courses, suggesting the wall rock was behaving more as a consolidated mass – capable of resisting greater applied loads. This site will be visually monitored over the next few years to document wall stability and to determine how long it will take to fully weather face overruns.

SUMMARY OF “LESSONS LEARNED”

Throughout the course of this FLH Technology Deployment Program project, a number of key lessons have been learned that will greatly improve future applications of PUR injection on Federal Lands projects. The following summarizes these key findings:

(1) Proper polyurethane product selection is highly dependent on (1) project requirements (ground consolidation? void filling? rock mass reinforcement/stabilization? water sealing?) and (2) setting conditions – particularly structure permeability, ambient operating temperatures and water conditions.

(2) Pre-injection volume estimation can be difficult, particularly in wet/damp conditions where a little PUR can go along way to filling cracks and voids. In general, an estimate of 300 lbs per injection hole/jam rod installation should be used for preliminary estimates, regardless of the application. For rock applications, where drilling is required, approximately 1,000 lbs of resin can be injected per day. For retaining walls with good face access, and where drilling is not required, upwards of 2,000 lbs of PUR can be injected daily.

(3) Until PUR is more fully evaluated for mitigating rockfall problems, it should NOT be used in lieu of bolting. However, PUR can be effectively used to minimize the amount of bolting that may be required, and may mitigate the need for other types of surface treatments (e.g. plates, straps, mesh).
Planning the efficient progression of work is essential to an optimal installation. On rock slopes, work should progress from the bottom up. This ensures that staged pumping is always working against a well-filled and sealed volume of rock as the PUR migrates upward through the rock mass. For rock retaining structures, it is recommended to treat the top of the wall first to stabilize loose, unconfined blocks before proceeding with interior wall injection. Injection rods placed several feet behind the wall face, on approximate 5-ft centers along the wall, and to within 5-6 ft of the bottom of the wall, should then be injected, taking care not to create conditions within the wall where expanding resin is pressuring against prior sealed sections of the structure. Finally, direct face injection should be done to stabilize facing rock. It does not appear that drilling is required for most rock retaining wall PUR applications – the jam rod technology is sufficient for effective PUR delivery to the wall mass.

There does not appear to be a need for drainage pipe installation when treating porous retaining walls. PUR coverage is neither continuous within the wall mass or sufficient to fill entire voids. The same can be said for rock mass installations as well. Although only a fraction of the existing void space may be filled, the strength increase achieved by bonding wall elements together and/or consolidating wet sections with foaming PUR appears to greatly enhance wall stability.

Staged pumping of relatively small volumes of PUR at very low pump pressures appears to work well for the progressive stabilization of both rock and retaining structures. Higher volume, high-pressure pumping should be limited to the mining industry where isolated rock failure during injection (hydrofracturing of the rock mass) can be tolerated. Staged pumping, coupled with fast set times, ensures that loading from hydrostatic injection pressures are isolated and of short duration.

The majority of the cleanup effort should be done within 1-2 minutes of PUR overrun, before it has a chance to set. Hand tools are effective at chipping and peeling drips and runs from rock surfaces, but cannot remove all of the resin overrun. In truth, the resin product’s dark brown color blends well with most surfaces, making it difficult to see from more than 10-15 feet away. The foaming product is a much lighter color, and is readily visible from a distance. Fortunately, foamed PUR is much easier to remove than dense, non-foamed PUR, limiting its visibility on most projects.

In addition to the two case histories presented in this paper, FLH has also used PUR to stabilize a sagging sandstone tunnel brow in Colorado National Monument, near Grand Junction, CO, and CDOT has used the product to enhance the stability of previously bolted rock slopes subject to large planar and wedge failures along U.S. Hwy 6 just west of Golden, CO. In all cases, the PUR applications appear to have met FLH’s program goals: application of a rapidly deployed, cost-effective, superior ground stabilization method that meets aesthetics objectives of context-sensitive settings.
ACKNOWLEDGEMENTS

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ROCK GLUE FOR AN UNSTABLE ROCK SLOPE:
Rockfall Mitigation Design with Polyurethane Resin Grout

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ABSTRACT

Rockfall mitigation schemes for transportation projects typically include design elements that dramatically alter the visual appearance of rock slopes. For example, slope profile reconfiguration and installation of rockfall netting and control barriers are commonly used to reduce rockfall hazards. Projects for which preservation of existing aesthetic qualities of the rock slope is a high priority, design elements that severely impact the rock slope’s appearance are less desirable than elements that are hidden or have a limited surface expression. The use of polyurethane resin (PUR) grout in lieu of traditional rock bolts and rock anchors may be a practical solution to reduce the rockfall hazard potential while preserving slope aesthetics. The method of injecting PUR grout into the rock mass for the purpose of bonding individual blocks into a bigger, more stable, continuous mass is called “rock gluing.” Rock gluing has been used in the tunneling and mining industries primarily to control water seeping into underground spaces and stabilize the crown. Rock gluing is relatively new as an above ground technique, but has been used successfully in transportation projects to improve the structural integrity and slope stability of rock masses. A comparison of two design concepts for mitigating potential rock slope failure modes within a 240-ft long, 35-ft tall rock slope on the George Washington Memorial Parkway in Arlington County, Virginia shows that incorporating rock gluing into the mitigation design in lieu of traditional patterned rock anchors provides an effective way to reduce rockfall hazards while preserving slope aesthetics.

INTRODUCTION

Methods used to stabilize highway rock slopes include passive and/or tensioned rock bolts used to anchor portions of the rock mass within the slope, rockfall netting, and rock slope excavation to a safe slope ratio. Although effective, these stabilization methods tend to drastically alter the natural visual appearance of the slopes they protect. In order to provide a context sensitive design that meets the National Park Service (NPS) and Federal Highway Administration (FHWA) Eastern Federal Lands Highway Division (EFLHD) objectives, an innovative design approach was sought. A technique borrowed from the mining industry, called rock gluing, was thought to offer promising results. Herein, we present a rock slope stability design case study involving rock gluing as part of the stability solution.

Rock gluing has been used in the tunneling and mining industries since the 1960’s, primarily to control water seeping into underground spaces. It also provides a structural function of improving the structural integrity and slope stability of the rock mass. Rock gluing for above-ground rock slope stabilization is relatively new, but it has been used successfully on at least two highway projects associated with FHWA Central Federal Lands Highway Division (CFLHD). In 2005, rock gluing was used to stabilize an overhanging rock slope along the General Hitchcock Highway between Tucson, Arizona and the Mt. Lemon Ski Area (Carder, 2006). In 2006, rock gluing was used to stabilize the rock mass at a tunnel entrance in Poudre Canyon, Colorado (Carder, 2006).
**ROCK GLUING**

Rock gluing is the method of injecting polyurethane resin (PUR) grout (i.e., “rock glue”) into a rock mass to bind individual rock blocks into a bigger, more stable, continuous mass. Rock gluing is used to seal off discontinuities to control planar sliding and toppling instabilities, and to reduce long-term impacts of weathering and freeze-thaw effects. Rock glue is injected under pressure and the glue is forced into rock mass discontinuities where it hardens and adheres to the rock. The hardened glue increases the tensile and shear strength characteristics of the discontinuities and thereby reduces the risk of sliding failure along those discontinuities. It also reduces the pore volume and thus the potential for freeze thaw and opening of new discontinuities. It improves the strength and stability of the rock mass internally without any surface expression that would alter the aesthetics.

There are many types of PUR grouts, and it is difficult to properly characterize the entire group. However, in general PUR grouts have several advantages when used as rock glue for rock slope stabilization. Because PUR comes in a variety of viscosity, set times, permeability, and strength, it can be tailored for a specific project application. Certain types of PUR have high strength that is comparable to cement grouts, but are less brittle than cement grouts. PUR also can have high-strength adhesion, comparable to intact rock. PUR is almost a pure liquid allowing it to permeate fine cracks within a rock mass. Fast setting times allow for fast grouting operation and a fast slope stabilization program, which is important for transportation projects. In addition to the grouting pressure, pressure is generated by foaming of the grout, which increases its ability to penetrate into rock discontinuities. PUR also has water sealing properties.

PUR is expected to be highly durable for rock slope applications. Soils grouted with PUR do not display signs of long-term degradation (Oshita et al., 1991). However, some degradation may be expected in highly acidic or alkalic environments. Most PUR material is non-toxic and has even been approved for potable water applications.

Rock gluing also has several disadvantages for rock slope stabilization. Production design standards have not yet been established for slope stability applications and there is no direct verification technique for discontinuity coverage except for performing cores after PUR has been installed. Longevity of PUR grout may be affected by acidity or alkalinity of groundwater. PUR can not be installed in very hot or very cold conditions.

**Installation Method**

In general, the following rock gluing procedure is followed for slope stabilization:

1. Establish the injector hole pattern, typically 10-20 ft apart.
2. Drill first injector hole, typically 1.5 inch diameter.
3. Inject PUR using packers to seal the hole.
4. Pressurize and pump until termination criteria are met. Termination criteria may include pressure, grout take, setting time, and/or observed rock glue flow from slope face.
5. Hold for setting time.
6. Pump for a second stage.
7. Fill hole with grout and top off with colored grout to hide hole.
8. Move to next (adjacent) injector hole and repeat injection process.
9. Verify with intermediate hole, if necessary.
10. Proceed and adjust spacing as necessary.
11. Finally, clean up excess surface hardened glue.

GEORGE WASHINGTON MEMORIAL PARKWAY, VIRGINIA

Background

The George Washington Memorial Parkway (GWMP) was established by congress in 1930 as a memorial to the first president of the United States. The Parkway runs parallel to the western shore of the Potomac River from Interstate 495 at the north end to its southern terminus at George Washington’s Mount Vernon Estate and Gardens. The park setting and the scenic character of the roadway preserve the natural environment along the river, and travel along the Parkway allows for sweeping views of historic vistas and the nations capitol. Management and maintenance of the Parkway falls under NPS jurisdiction.

A rockfall event in 2002 released several large rock blocks with an approximate maximum diameter of 10 ft onto the northbound lane of the GWMP in the vicinity of the Francis Scott Key Bridge in Arlington County, Virginia (Figure 1). Although no one was injured, the rockfall damaged curb and pavement sections and temporarily disrupted traffic along this busy commuter corridor. Cleanup activities after the rockfall event included limited scaling of the rock slope with a backhoe to remove the most critically unstable blocks. However, there were concerns that potentially unstable rock blocks remained as a rockfall hazard. Therefore, the FHWA-EFLHD contracted with Schnabel Engineering, LLC to perform a slope stabilization design study to assess the stability of the slope, develop concepts for mitigating the potential slope instability and rockfall hazards, and prepare design plans and specifications for the chosen concept option.

Preserving the aesthetic quality of this scenic roadway was an important design consideration. The slope stabilization design needed to mitigate the rockfall hazards using construction methods and hardware that would minimize the visual impact to the existing rock slope. Rock gluing was considered as an
alternative to traditional slope stabilization elements such as rock anchors and wire mesh drapes that would drastically alter the slope’s appearance.

Schnabel Engineering personnel collected geologic rock structure mapping data in the field, and used the data to perform kinematic and limit equilibrium analyses to evaluate and characterize the rock slope hazards as a basis for developing the mitigation design concepts. Two remediation design concepts with engineering cost estimates were developed to compare a traditional mitigation design including patterned rock anchors with a rock gluing solution.

Field Measurements and Design Considerations

The purpose of the site investigation was to gather information in support of the stability analysis and rockfall hazard assessment. Data was collected with respect to intact rock and rock mass properties, groundwater conditions, and discontinuity characteristics. Additionally, the size and shape of typical rockfall blocks expected to be shed from the slope were measured in support of the rockfall hazard assessment.

The rock exposed in the cut slope is brownish-gray, moderately- to strongly-foliated, quartzofeldspathic metamorphic rock of the lower Cambrian-aged Sykesville Formation (Fleming et al., 1994). The geologic structure of the Sykesville Formation is characterized on a local scale by foliation and joints. Based on rock hardness index testing, the intact rock is defined as “strong” with an expected unconfined compressive strength of about 10,000 psi. The rock mass is described as “moderately weathered” indicating that significant portions of the rock are discolored and may be significantly weaker than in the fresh state. The overall fracture spacing of the rock mass observed in the slope face ranged from about 2 to >6 ft between individual discontinuities.

The character of rock mass discontinuities behind the slope face is the most important controlling factor for rock slope stability. Schnabel Engineering conducted geologic rock structure mapping to identify and characterize discontinuities exposed within the rock face. Data was collected to characterize individual discontinuities including the following:

1. Location (i.e., station),
2. Discontinuity type,
3. Orientation (i.e., dip and dip direction),
4. Estimate of the Joint Wall Compressive Strength (JCS) based on hardness index testing,
5. Joint Roughness Coefficient (JRC, following Barton and Choubey, 1977),
6. Infilling type and coverage,
7. Water condition, and
8. Persistence and termination.
A total of 66 individual discontinuities were inspected during the rock structure survey. The orientations of these discontinuities were plotted and contoured using the computer program DIPS produced by Rocscience Inc. The DIPS stereonet contour plot in Figure 2 shows the discontinuity orientation measurements. The contours represent statistical concentrations of discontinuity orientation vectors, and help to visualize the clustering of orientation data. Each data cluster shown represents a discontinuity set, such as foliation and joint sets, considered in subsequent kinematic analysis. The orientations of the planes associated with the major contour peaks are considered to represent the average orientation of each discontinuity set. The scatter of the individual discontinuity points around each contour peak indicates the variability of the orientation of discontinuities associated with each set. The following major discontinuity sets to consider in the analyses were identified:

**Plane 1:** Joint set with an average orientation of 80°/200° (dip/dip direction). Plane 1 joints tend to be very persistent. Some Plane 1 joints were traceable for over 100 ft along strike. These joints form the majority of the rock slope face. A lineation feature observed on Plane 1 joint surfaces is defined by the intersection of Plane 1 joints with Plane 3 joints. The JCS is estimated to be 4,000 psi. The average JRC is about 7. Iron staining was observed on the majority of the Plane 1 joint surfaces.

**Plane 2:** Joint set with an average orientation of 62°/174° (dip/dip direction). Plane 2 joints do not tend to be very persistent. Plane 2 joints are typically traceable for less than 10 ft. The JCS of these joints is estimated to be 4,000 psi. The average JRC is about 7. Iron staining was observed on many of these joint surfaces.

**Plane 3:** Joint set with an average orientation of 28°/003° (dip/dip direction). Plane 3 joints are relatively persistent as they are traceable for up to about 30 ft along strike. These joints were identified as possibly a secondary foliation feature. The intersection of Plane 3 joints with the Plane 1 joints are identified by a lineation observed on Plane 1 joint surfaces. The JCS is estimated to be 4,000 psi. The average JRC is about 11. Plane 3 joints are particularly rough along strike. Some iron staining was observed on a few of the Plane 3 joint surfaces.
Plane 4: Joint set with an average orientation of 70°/063° (dip/dip direction). Plane 4 joints do not tend to be very persistent as they are typically traceable for less than 10 ft. Plane 4 joints intersect with Plane 6 foliation to form an adversely oriented wedge sliding surface. Wedge sliding along Planes 4 and 6 appear to have resulted in several localized rockfall events. The JCS is estimated to be 4,000 psi. The average JRC is about 11. Iron staining was observed on some of the Plane 3 joint surfaces.

Plane 5: Joint set with an average orientation of 33°/153° (dip/dip direction). Plane 5 joints tend to be very persistent and traceable for up to 60 ft. They have an average spacing of between about 15 ft and 20 ft. The JCS is estimated to be 1,000 psi, on average, with an average JRC of about 6. Plane 5 joints were observed to exhibit iron staining.

Plane 6: Foliation with an average orientation of 74°/280° (dip/dip direction). Plane 6 discontinuities are separations along foliation planes, and tend to be very persistent and traceable for up to 50 feet. Plane 6 foliation separations intersect with Plane 4 foliation to form adversely oriented wedge sliding surfaces. Wedge sliding along Planes 4 and 6 appear to have resulted in several localized rockfall events. The JCS is estimated to be 1,000 psi, on average, with an average JRC of about 10. Plane 6 foliation separations were observed to exhibit iron staining.

Groundwater is a very important consideration for slope stability analyses because groundwater uplift pressures in a rock mass can be a major driving force for slope failures. Numerous localized seepage points were observed coming from discontinuities daylighting from the slope (e.g., Figure 3) indicating that groundwater uplift pressures should be accounted for in the geological engineering analyses. Without reliable groundwater level data from piezometers installed behind the slope face, it is unclear whether the observed seepage represents stabilized groundwater conditions within the rock mass or if it reflects transient and localized surface drainage pathways. The slope stability analyses were based on the possibility of high groundwater levels behind the slope face. Albeit a conservative approach, it was considered that rock mass discontinuities may be filled with water during extended periods of heavy rain.

Typical rockfall block sizes for the rockfall hazard analysis were measured from rock blocks lying on the ground at the base of the existing slope. These rocks

Figure 3: Groundwater seepage point observed near the base of cut slope.
were observed to be generally rectangular to irregularly shaped, and are typically up to approximately 3 ft across in the maximum dimension. However, it was acknowledged that 10 ft rock blocks are possible based on the blocks that were shed from the slope during the 2002 rockfall event.

**Engineering Analyses and Design Considerations**

Based on the geometric relationships of the 66 discontinuities observed and measured at the site (Figure 2), kinematic stability analysis was used to evaluate potential rock slope failure modes controlled by planar rock mass discontinuities. Potential rock slope failure modes include sliding of rock blocks along a single planar discontinuity, sliding of wedges formed in the slope by intersecting discontinuity planes, and toppling of rock blocks. The computer program RockPack III for Windows, produced by RockWare Inc., was used for the kinematic stability analysis. Inputs to the program include the orientations of measured rock mass discontinuities within the rock mass, the orientation of the rock slope, and the estimated rock mass discontinuity friction angle. Interface friction angle calculations were performed for each of the six major discontinuity sets based on the Barton (1976) method. For the kinematic analysis, an interface friction angle of 33 degrees was selected to check for sliding potential since it was the lowest friction angle calculated for each of the six discontinuity sets.

A RockPack stereonet plot is presented as Figure 4 for the typical rock cut slope configuration to illustrate the results of our kinematic analysis. The results of the kinematic stability analysis are summarized as follows:

1. Potential wedge sliding failures are indicated based on the average orientations of Plane 4 and Plane 6 discontinuities.

2. Although the dip direction of the Plane 3 discontinuity set is adversely oriented with respect to potential planar sliding (i.e., the plane dips towards the slope face), potential planar sliding is not indicated based on the average dip of the Plane 3
discontinuity set since the dip of the plane is less than the interface friction angle assumed in our analysis. However, it was recognized that groundwater pressures acting on Plane 3 discontinuities may trigger planar sliding.

3. A potential for toppling failure is indicated. Based on the kinematic analysis, toppling may occur along Planes 1 and 2. Toppling is considered to be a local, and not a global stability hazard.

Where potential failure modes were identified in the kinematic analysis, more detailed limit equilibrium analyses were performed to calculate a factor of safety (FS) against sliding based on slope geometry, measured discontinuity orientations, estimated groundwater conditions, calculated discontinuity interface friction angles, and estimated or measured material properties of the rock mass. Global (i.e., large, slope-scale) and local (i.e., small scale) failures were analyzed. For the global case, a slope height of 35 ft with a failure surface (i.e., plane or wedge intersection) extending through the toe of the slope was considered as a worse-case scenario. A 10-ft high rock block was assumed for the local case based on the size of the 2002 rockfall block. Discontinuity interface friction angles of 47°, 63°, and 49° were used for Planes 3, 4 and 6, respectively, based on calculations following Barton (1973). Zero cohesion was assumed. A tension crack was assumed to occur 50 ft from the slope crest for analysis of global planar failure (based on geometrical considerations), and 5 ft and 10 ft from the slope crest during local planar failure. For the local wedge failure analyses, tension cracks were assumed to occur 5 ft and 9 ft from the slope crest. A tension crack was not assumed for the global wedge failure condition. Both “dry” and “wet” conditions were considered to account for potential groundwater uplift pressures acting on sliding and toppling blocks. For the global failure analyses, it was considered that the sliding surfaces and tension crack could be 0% and 100% filled with water. For the local failure analyses, it was considered that tension cracks could be 0% and 100% filled with water.

The limit equilibrium analyses were performed using RockPack III, which calculates the FS for a potential planar or wedge slide geometry as a function of driving and resisting forces. The program also calculates the sum of moments for a given toppling scenario. The output for the toppling evaluation provides the normal force required of a rock anchor to hold the toppling block in place. The limit equilibrium analysis was used to evaluate existing conditions (i.e., without reinforcement), as well as the force required to achieve a FS of at least 1.5. A FS of 1.5 is typically considered acceptable for permanent slopes (FHWA, 1989). Therefore, for cases where the FS was found to be less than 1.5, the horizontal force required to obtain a FS of 1.5 was also calculated. In these cases, the horizontal force required to obtain a FS of 1.5 was used to evaluate potential reinforcement mechanisms including rock gluing and rock anchoring.

Table 1 provides a summary of the limit equilibrium analysis results for planar and wedge sliding scenarios. The results indicate a FS greater than 1.5 against potential global (i.e., 35 ft high failure blocks) and local (i.e., 10 ft high failure blocks) planar sliding along Plane 3 discontinuities assuming a dry condition for the failure surface.
### Table 1: Summary of Limit Equilibrium Analysis Results

<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Controlling Discontinuity Plane(s)</th>
<th>Block Height (ft)</th>
<th>Tension Crack Location - Distance from Slope Crest (ft)</th>
<th>Percent of Tension Crack Filled with Water</th>
<th>Factor of Safety</th>
<th>Horizontal Force (lb) Required to Achieve FS ≥ 1.5</th>
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<tr>
<td>Global Planar Sliding</td>
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<td>35</td>
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<td>100</td>
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<tr>
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<td>5</td>
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</table>

However, the FS against planar sliding is less than 1.5 for both the global and local cases if the tension crack is considered to be partially filled with water. This can occur when surface run-off during a heavy rainstorm fills up the crack before it drains, or if the tension crack is filled due to groundwater conditions. Therefore, the mitigation design concepts must consider reducing groundwater uplift pressure for the global- and local-scale planar sliding condition, or otherwise provide a means to overcome potential groundwater uplift pressures. The analysis indicates a FS greater than 1.5 against potential wedge sliding along Planes 4 and 6 discontinuities for global- and local-scale failures assuming both dry and wet conditions. Limit equilibrium analysis indicates that rock blocks bounded by Plane 2 discontinuities have the potential for toppling failure. Toppling failure is considered a local, and not a global slope stability hazard. The tension force on a rock anchor installed in the middle of the block required to stabilize (i.e., FS = 1.5) a 10 ft tall block is 336 lbs and 5,018 lbs for dry and wet conditions, respectively.

Site conditions soon after the December 13, 2002 rockfall event are illustrated in Figure 1. The rockfall zone is approximately 20 ft tall by 10 ft wide, and thus represents a local failure condition. Discontinuity Planes 2, 3 and 4 provided the release surfaces and define the boundaries of the rockfall block. Based on the geometry of the release surfaces, it was interpreted that this rockfall event was controlled by planar sliding along the Plane 3 joint surface at the bottom of the rockfall block. It is believed that the major contributing factor in this rockfall event was the relatively small surface area of the Plane 3 boundary surface compared with the relatively large size of the rock block. The small surface area would provide limited...
shear resistance to stabilize the large rock block it supported. The results of the FS analyses indicate that the potential for planar sliding along Plane 3 joints is greater with groundwater uplift pressures along the release surfaces acting to increase the driving force. Therefore, water pressures may have also influenced the rockfall event based on the seepage markings that appear from many discontinuities in the rockfall area. A third contributing factor may have been ice wedging that acted along the discontinuity planes bounding the failed rock block over time.

Based on our kinematic and limit equilibrium analyses, and consideration of the likely cause of the 2002 rockfall event, it was concluded that there is a risk of future rockfall events within the rock slope area. It is believed that the following failure modes are possible if no stabilization measures are performed: 1) global planar sliding along Plane 3 discontinuities, 2) local planar sliding along Plane 3 discontinuities, and 3) local toppling failure along Plane 2 discontinuities. The rock slope stabilization measures must account for these failure modes. Rock slope stabilization design options addressing the three failure modes identified above are described in the succeeding sections of this paper.

**Slope Stabilization Design Concepts**

Two design concepts were developed to reduce the rockfall hazard potential for global- and local-scale planar sliding and local-scale toppling failure mechanisms indicated by the limit equilibrium analyses. Both concept designs include elements of limited scaling, minimum of spot bolting and installation of drains. However, the difference between the designs is the use of either rock anchors or rock glue as the primary reinforcement mechanism.

Scaling, also referred to as slushing, involves removal of loose or potentially dangerous rock blocks from the slope face by manual scaling (with pry bars), drag scaling (with something heavy such as a dozer track), or mechanical scaling (with hydraulic or pneumatic splitters). Scaling does not involve blasting. Scaling, by itself, was determined to be an impractical long term stabilization solution since natural weathering processes and freeze-thaw cycles will act to open discontinuities and destabilize rock blocks on a local scale over time. However, scaling was recommended as the first step to prepare the rock face for further reinforcement, and as a safety precaution to protect workers by reducing the potential for localized rockfall during reinforcement construction activities.

In addition to scaling, some larger rock blocks could be individually bolted (“spot bolted”) to the slope face using tensioned rock anchors or passive rock dowels if it is concluded that further attempts at scaling may destabilize the slope. Spot bolting was not intended to be a major work scope for either stabilization scheme, and may not be required at all. However, both design concepts include provisions for spot bolting if needed.

Both design concepts also include installation of horizontal drains. The potential for global planar sliding along Plane 3 discontinuities can be reduced to an acceptable level by reducing groundwater levels in the rock mass, and the groundwater levels in the rock mass can be lowered and controlled by installing “horizontal drains” in the rock slope face. The lowest row of drains
were designed to be 6 ft above the ground surface at the base of the slope and spaced horizontally at 20 ft. A second row of drains was planned 10 ft above the first row, spaced 20 ft apart horizontally, and staggered between the first row of drains. The drains were designed to penetrate at least 10 ft beyond a theoretical planar failure surface daylighting at the base of the slope such that the bottom row of drains will be about 50 ft long while the top drains will extend 25 ft into the rock mass.

With the risk of global planar sliding adequately reduced by installing horizontal drains, the local planar sliding and toppling failure modes are addressed by installation of rock anchors or rock gluing. Figures 5 and 6 illustrate the design concepts including rock anchors and rock gluing, respectively.

**Rock Anchor Design**

The rock anchor concept includes installation of 1-inch diameter, grade 75 bars placed within a 4-inch diameter bore hole and grouted. The bars would be protected from corrosion (e.g., epoxy coated and surrounded by cement grout). Because the largest rock block failures within the slope have been about 10 ft in size, the anchors would be a minimum of 20 ft long with a minimum 10 ft-long bond zone, and will be spaced 10 ft apart on center (vertical and horizontal). The anchors are designed to be tensioned to achieve a capacity of 50 kips. The anchors would be inclined a maximum of 15 degrees from horizontal. Based on the limit equilibrium analyses, this rock anchor pattern and anchor capacity should adequately stabilize both planar sliding and toppling rock blocks.

Tensioned rock anchors have several distinct advantages over rock gluing. First, the use of rock anchors is well established with a proven track record. The engineering behavior of the rock anchors is relatively well understood. The capacity of the tensioned rock anchors can be verified by conducting tension load tests (i.e., pull tests). Also, the rock anchors cost less than rock gluing for this project. Tensioned rock anchors also have several disadvantages. First, the exposed plate and nut may be highly visible on a contrasting-colored rock surface and thus will be aesthetically unattractive. However, rock-colored paints can be used to help the plate and nut “blend in” with the rock exposure. Alternatively, the anchor nut and plate can be recessed into the rock face, and rock-colored shotcrete can be used to cover the plate and nut to help them blend in. Second, the bond between grout and rock may weaken and deteriorate with time. The rate of deterioration is based on the chemical composition of groundwater. Permanent rock anchors are expected to last a long period of time and therefore are installed with corrosion protection.
Rock Gluing Design

The limit equilibrium analyses for potential planar and toppling failures indicate PUR grouting will effectively raise the (FS) for local-scale planar sliding and toppling above 1.5. To analyze the effectiveness of rock glue for stabilization of local planar and toppling blocks, the total resisting force (i.e., sum of tensile and cohesive strength contributed by the rock glue) required to raise the FS for local-scale planar sliding and toppling above 1.5 was calculated. The force required to raise the FS for local-scale planar sliding and toppling above 1.5 is based upon the horizontal force required to obtain a FS of 1.5 for the potential local-scale planar sliding and toppling conditions considered in our limit equilibrium analysis. It was assumed that the rock glue would effectively cover at least 50% of discontinuity and tension crack surfaces in the rock mass.

The results indicate that the minimum grout tensile strength and cohesion values of 5 psi and 6 psi for planar sliding and toppling, respectively, are required to achieve a total resisting force greater than the horizontal force required to obtain a factor of safety of 1.5 for the potential planar sliding and toppling conditions. These tensile strength and cohesion values should be easily achieved since tensile strength and cohesion for PUR grout reported by the manufacturer vary from 150 psi to 5,000 psi. Because the largest rock blocks that have fallen from the slope are about 10 ft in size, the glue injection holes would be spaced 10 ft apart on center. The

Figure 5: Slope stabilization design concept including rock anchors.
injection holes would be drilled 15 ft deep behind the rock face. The analyses indicate this pattern can adequately stabilize both 5 ft and 10 ft deep planar rock blocks.

![Figure 6: Slope stabilization design concepts including rock gluing.](image)

**Cost Estimates**

Various contractors provided cost estimates for scaling, rock gluing, spot bolting, pattern anchors, and installation of the drains. These estimates were used to prepare cost estimates for the recommended design concepts for rock anchors and rock glue. The rock gluing cost estimate is based on information provided to us by contractors, as well as cost information derived from the Poudre Canyon Project in Colorado with a 30% allowance for contingencies related to the estimated degree of fracturing within the rock mass. It is believed that there is some uncertainty in the construction costs for the rock gluing compared to rock anchors. The actual costs of rock gluing may vary from estimates depending on the actual degree of fracturing of the rock mass.

**SUMMARY**

Rock gluing may be a practical solution to mitigate rockfall hazards while preserving slope aesthetics for some rock slope stabilization design projects. It provides a viable alternative to traditional methods of rock slope stabilization when aesthetics is an important design consideration. Rock gluing was the preferred stabilization method for an unstable rock slope along the George Washington Memorial Parkway.
REFERENCES


Debris Flow Mitigation Using Flexible Barriers

by

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Disclaimer

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ABSTRACT

During October 2003 and June 2005 California suffered several major wild fires. The fires denuded slopes and baked the soils. Immediately following the fires the California Department of Transportation (Caltrans) began assessing the damage to the roadway infrastructure and implementing repairs. In many locations steeply incised drainages were classified as having a high potential for debris flow. Each location was investigated resulting in the installation of flexible debris flow barriers. Barrier selection was based upon site characteristics and governing equations for analyzing debris flow energy and flow thickness.

Flexible barriers have been widely used to catch and contain rockfalls. But barrier capacity for debris flow containment has had some uncertainties. In recent years testing and cases histories has led to development of engineered flexible debris flow barriers with predictable capacities of up to 370 ft-tons (1000 kilojoules). Caltrans elected to install these barriers at several locations in San Bernardino and Santa Barbara counties. In all 13 barriers were installed varying in capacity from 74 to 370 ft-tons (200 to 1000 kilojoules). In one location an experimental extra large barrier, “the Whale Net”, was installed to catch and contain flows on the order of three thousand cubic yards of material.

Seasonal winter rains following the fires caused debris flows to occur at several locations where debris flow barriers were installed. In some cases the barriers were impacted multiple times. Caltrans was tasked with maintaining the systems and assessing system performance. Actual debris flow impacts were compared to predicted impacts and system integrity was measured for repairs, cleaning operations and overall maintenance suitability.

System performance proved satisfactory. Following the initial debris flow impacts some cleaning and repairs were required. Based on the damage design modifications were implemented to improve system performance and maintenance. These new designs were subsequently impacted and performed as expected. Flexible debris flow barriers have proven to be fully capable of stopping and containing impacting debris flows with acceptable levels of maintenance and cleaning methods.
INTRODUCTION

During October 2003 and June 2005 Southern California suffered several major wild fires. Hundreds of thousands of acres of land were burnt in San Bernardino, San Diego, Santa Barbara, Ventura and Los Angeles counties. The fire impacted Forest Service lands, rural communities, county roads and state highways. The results of the fires were denuded slopes and baked the soils. In the aftermath of this disaster, county and state officials quickly set out to assay the damage and implement repairs. The principle concern for transportation personnel was the impact the denuded slopes could have on slope stability above and below transportation corridors. Immediately following the fires the Caltrans began assessing the damage to the roadway infrastructure and implementing repairs. With the potential for heavy winter rains much of the effort was focused on hillside erosion in the form of debris flows and mudflows within the steeply incised drainages.

SITE HISTORY

SAN BERNARDINO COUNTY

On State Route 18 following the fires in October 2003 Caltrans began work preparing for the winter and the possibility debris flows. Catchment ditches were cleaned to original condition, culverts were cleaned, and drainage ditches were restored. By mid December the roadway was re-opened. Then on December 25, 2003 a storm passed through the area and deposited nearly 6 inches of rainfall in a 24-hour period (See Figure 1). An E-mail from Maintenance in the area sums up the event: “As you probably have heard we got our butts kicked Xmas day. Six inches of rain that day made all the burn areas above the highways cut loose. The 18 four lane was up to five feet deep in mud and rock in several spots. The narrows got narrower when we lost the up bound lane at the "low wall. 18 below Rim Forest was closed and even 138 by Silverwood was under mud for awhile. Now we have opened all except for parts of the #2 lanes on the 18, (the narrows will be closed "indefinitely"), but all the drainage is plugged solid. The weather Service has got us with two significant rain/snow events Mon night thru Wed then again Fri thru Sunday. Snow level to start at seven then to five thousand feet at the end of the storms. So.........we are working only on the lanes and only the BM night crew of four at that. If these storms hit as advertised kiss it goodbye. The toll rd may be the only way in and out of arrowhead. If I can get my hands on the digital camera I’ll send some pic's. Have fun,”

Figure1: Rainfall Totals for San Bernardino Mountains December 2003
Most of the roadway on California State Route 18 between Mile Post 17.7 and 18.9 was covered with debris flow deposits (See Figure 2) and was only passable with four-wheel drive vehicles. Culverts were plugged; ditches filled to capacity and overflowed so that all drainage systems were overwhelmed with debris flows and surface water flow. The material passed across the roadway and spilled over several embankments scouring away the roadway embankment and destabilizing the roadway. Although the existing infrastructure had served the area adequately for more than 40 years this combination of intense wildfires and extreme rainfall was “the perfect storm.”

**Figure 2: Debris and Flood Damage on the Narrows**

**SANTA BARBARA COUNTY**

June 2005 fires raged through the coastal mountains where California State Route 101 emerges from an inland corridor to a coastal corridor through the Gaviota Pass. The Pass is notoriously windy as inland and coastal temperature differences create a nearly constant flow of wind fueling the fires. Early winters rains were above normal and the combination caused significant debris flows in several drainages. In the Janet Creek location a large debris flow, two thousand cubic yards, discharged onto the highway pushing a RV off the roadway (See Figure 3) into the guardrail just above the raging Gaviota Creek. Debris flowed down the highway several hundred feet through the Gaviota Tunnel closing Highway 101 north bound lanes. The same storm then moved eastward and caused closure of Interstate 5 both northbound and southbound. These closures caused by debris flows and snow severed the two only major north south highways in the State.

**Figure 3: Debris Flow Damage in the Gaviota Pass**
INVESTIGATION
Initially all pertinent Geologic data available was reviewed. Areas with a history of instability were identified as well as areas underlain by known unstable geologic units. Following this review geologic personnel did a ground survey of each site. Each area was mapped, unstable sections were delineated, and hazard potential was assigned. Dozens of areas were identified with potential instabilities and each location prioritized. Thirteen areas were identified as having a high potential for repetitive debris flow activity. The debris flow locations were concentrated in two areas: California State Route 18 in San Bernardino County and California State Route 101 in Santa Barbara County (See Table 1).

<table>
<thead>
<tr>
<th>Location</th>
<th>Highway</th>
<th>Mile Posts</th>
<th>Location Name</th>
<th>Location Identification</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Bernardino County</td>
<td>California State Route 18</td>
<td>17.65 to 19.0</td>
<td>The Narrows</td>
<td>1 through 10</td>
</tr>
<tr>
<td>Santa Barbara County</td>
<td>California State Route 101</td>
<td>46.0 to 47.8</td>
<td>Gaviota Pass</td>
<td>South Tunnel, North Tunnel, and Janet Creek</td>
</tr>
</tbody>
</table>

Each location was a natural drainage that discharged into the roadway corridor. The investigation concentrated on the lower portion of each drainage. Debris flow volumes and particle sizes were determined from maintenance records of past events. Channel dimensions were measured; they included width, depth and gradient. Debris flows were classified as either fine or coarse flows. Fine grained flows or mudflows mainly consist of water and fine material, which is uniformly distributed. Coarse or granular flows mainly consist of water, fine and rougher material. The larger components are mostly accumulated at the front of the flow.

ANALYSIS
Debris flow load parameters are critical input data to dimension debris flow barriers. However there is a limited understanding of the mechanics of debris flows that is compounded by the difficulty in measuring debris flow parameters during real events. The calculations used in the design of these barriers are those suggested by Rickerman (1999) and outlined by Roth (2003); they incorporate mathematical models, observations, experiences, and geomorphic assessment. These mechanical and rheological models were used to determine debris flow peak discharge, debris flow velocity, debris flow depth, energy, and quasi-static force (the final force imparted on the system as the impacting mass decelerates).

![Figure 4: Direction Profile of a Debris Flow is defined by the flow depth (h), cross-sectional area (A), and the front flow velocity (v).](image)

Figure 4: Direction Profile of a Debris Flow is defined by the flow depth (h), cross-sectional area (A), and the front flow velocity (v).
Figure 5: Cross Section of a Stopped Debris Flow. The simplified assumption is that the width of the flow corresponds to the average bed width.

The first step is to estimate the debris flow volume (See Figures 4 and 5) from which, using this procedure, a design debris flow can be calculated providing predictive estimates of impact energy, required barrier height to contain the flow, and quasi-static forces. This information was used in the selection and design of the debris flow barrier. A summary of the analysis is outlined in Tables 2 and 3.

What was unique at these locations is that recent debris flows occurred in each drainage. By using maintenance and construction records a reasonably accurate measurement of debris flow volume for each drainage was determined.

Table 2: Debris Flow Analysis Data and Barrier Design for San Bernardino County, California State Route 18

<table>
<thead>
<tr>
<th>Location</th>
<th>Volume (cubic yds)</th>
<th>Energy (Ft-tons)</th>
<th>Barrier Height (feet)</th>
<th>Quasi-Static force (tons)</th>
<th>Barrier Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>470</td>
<td>445</td>
<td>15</td>
<td>90</td>
<td>UX-150</td>
</tr>
<tr>
<td>2</td>
<td>411</td>
<td>208</td>
<td>15</td>
<td>42</td>
<td>VX-?</td>
</tr>
<tr>
<td>3</td>
<td>196</td>
<td>93</td>
<td>10</td>
<td>19</td>
<td>UXI-050</td>
</tr>
<tr>
<td>4</td>
<td>457</td>
<td>320</td>
<td>12</td>
<td>65</td>
<td>UX-150</td>
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<tr>
<td>5</td>
<td>313</td>
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<td>15</td>
<td>50</td>
<td>UX-075</td>
</tr>
<tr>
<td>6</td>
<td>228</td>
<td>260</td>
<td>20</td>
<td>45</td>
<td>VX-?</td>
</tr>
<tr>
<td>7</td>
<td>130</td>
<td>74</td>
<td>20</td>
<td>13</td>
<td>VX</td>
</tr>
<tr>
<td>8</td>
<td>130</td>
<td>74</td>
<td>10</td>
<td>11</td>
<td>UXI-050</td>
</tr>
<tr>
<td>9</td>
<td>653</td>
<td>445</td>
<td>15</td>
<td>82</td>
<td>VX</td>
</tr>
<tr>
<td>10</td>
<td>196</td>
<td>119</td>
<td>12</td>
<td>19</td>
<td>VX</td>
</tr>
</tbody>
</table>

Table 3: Debris Flow Analysis Data and Barrier Design for Santa Barbara County, California State Route 101

<table>
<thead>
<tr>
<th>Location</th>
<th>Volume (cubic yds)</th>
<th>Energy (Ft-tons)</th>
<th>Barrier Height (feet)</th>
<th>Quasi-Static force (tons)</th>
<th>Barrier Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>South Tunnel</td>
<td>261</td>
<td>185</td>
<td>10</td>
<td>56</td>
<td>UX-050</td>
</tr>
<tr>
<td>North Tunnel</td>
<td>196</td>
<td>148</td>
<td>10</td>
<td>45</td>
<td>UXI-050</td>
</tr>
<tr>
<td>Janet Creek</td>
<td>2600</td>
<td>&gt;370</td>
<td>20</td>
<td>&gt;90</td>
<td>Whale Net</td>
</tr>
</tbody>
</table>
MITIGATION
Once the design parameters had been calculated the system was designed to fit in the drainage. Each design was nested into the existing terrain to minimize impacts, facilitate corridor restrictions, and be maintainable. At each location a field diagram (See Figures 6 and 7) was drawn optimizing topography and geology to ensure suitable foundation materials for the posts and ground anchors.

Figure 6: Schematic of Debris Flow Barrier Design at Location 1 presenting the VX system utilizing posts within the channel.

Figure 7: Schematic of Debris Flow Barrier Design at Location 6 presenting the UX system, the no-post alternative.
RESULTS
Another unique aspect to the case histories is that 11 of the 13 debris flow barriers were impacted following construction. This provided the opportunity to compare the empirical calculations to actual events and assess the performance of the barrier under actual known impact loads (See Table 4).

Following each event the loaded barrier was measured to determine the debris volume contained by the barrier, system component performance (brake activation, anchor performance, etc.) and repair to the system (See Figures 8 and 9).

**Figure 8:** The narrows in San Bernardino County Location 1 Debris Flow Barrier Impact. The field estimate was 161 cubic yards of which 35 percent was soil, 40 percent cobbles and boulders and 35 percent vegetation.

**Figure 9:** North Tunnel Debris Flow Barrier, Design, Construction, Impact.
Table 4: Debris Flow Data for San Bernardino County, California State Route 18 and for Santa Barbara County, California State Route 101

<table>
<thead>
<tr>
<th>Location</th>
<th>Barrier Design</th>
<th>Design Volume M3</th>
<th>Design Energy kJ</th>
<th>Impact Volume Yds³</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>1</td>
<td>UX-150</td>
<td>470</td>
<td>445</td>
<td>161</td>
<td>Soil 35%, Rock 40%, Veg 55%</td>
</tr>
<tr>
<td>2</td>
<td>VX-</td>
<td>411</td>
<td>208</td>
<td>263</td>
<td>Soil 26%, Rock 40%, Veg 25% (10% 4 trees 3’ in diameter 60’ long)</td>
</tr>
<tr>
<td>3</td>
<td>UXI-050</td>
<td>196</td>
<td>93</td>
<td>27</td>
<td>Soil 70%, Rock 3%, Veg 0%</td>
</tr>
<tr>
<td>4</td>
<td>UX-150</td>
<td>457</td>
<td>320</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>UX-075</td>
<td>313</td>
<td>260</td>
<td>62</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>6</td>
<td>VX-</td>
<td>228</td>
<td>260</td>
<td>26</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>7</td>
<td>VX</td>
<td>130</td>
<td>74</td>
<td>33</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>8</td>
<td>UXI-050</td>
<td>130</td>
<td>74</td>
<td>77</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>9</td>
<td>VX</td>
<td>653</td>
<td>445</td>
<td>95</td>
<td>Soil 60%, Rock 35%, Veg 5%</td>
</tr>
<tr>
<td>10</td>
<td>VX</td>
<td>196</td>
<td>119</td>
<td>30</td>
<td>Soil 60%, Rock 35%, Veg 5%</td>
</tr>
<tr>
<td>South Tunnel</td>
<td>UX-050</td>
<td>261</td>
<td>185</td>
<td>300</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>North Tunnel</td>
<td>UXI-050</td>
<td>196</td>
<td>148</td>
<td>200</td>
<td>Soil 70%, Rock 30%, Veg 0%</td>
</tr>
<tr>
<td>Janet Creek</td>
<td>Whale Net</td>
<td>2600</td>
<td>&gt;370</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

CONCLUSIONS

The procedure followed in these case studies proved to be a suitable procedure for assessing, analyzing and designing for debris flows where expected volumes are below 400 cubic yards. It was found at these locations that debris flow volumes were equal to or less than the initial debris flows following the fires and first rains and that using volumes estimated from the initial debris flows are reliable in the design procedure.

The modeling procedure as outlined by Roth (2003) proved to provide conservative but appropriate values of energy, forces and barrier heights for barrier design and anchor loading.

The debris flow barriers designed specifically for debris flow mitigation performed as anticipated in catching and containing debris flows within 185 ft-tons (500 kJ) of design energy.

Maintenance was required to clean out the debris, repair some parts, and reset the friction brakes (energy absorbing devices). No ground anchors were damaged. A support rope was broken in tension upon impact by a three-foot diameter 60-foot long tree. Friction brake resetting and replacement was commonly required.

Cleaning the barriers was done in two ways. At the Narrows the top support ropes were removed from the cable seat at the top of the post and allowed to sag. This resulted in an effective barrier height whereby an excavator could easily reach over the top for material removal (See Figure 10).
At Gaviota Pass access roads were installed at each location to allow a small excavator/backhoe to gain access behind the net from the side (See Figure 11).

![Image of excavator near a net]

**Figure 10: Preparing to disconnect the top support ropes from the top of the post to begin cleaning operations.**

**Figure 11: Cleaning the net from an access road providing access to the back of the barrier.**

**RECOMMENDATIONS**

While it was clear that within this energy and volume range the barriers worked there are a few barrier features that require modification. Initially the designs included a fine-grained mesh behind the barrier. Debris flow locations are typically within drainages and swales. During normal flows it was found that leaves, small branches and fine-grained material would get caught in the mesh (See Figure 12). This turned out to be a maintenance/drainage issue after some buildup. It also restricts barrier flexibility. The design was modified by leaving the bottom 4 feet of barrier free of the fine-grained mesh to allow small debris to flow along its normal course and not build up behind the barrier.
Figure 12: Leaves, small branches and fine-grained material accumulating behind the barrier during normal flows requiring maintenance for cleaning to keep the barrier 100% operational.

The standard design is to attach the infrastructure cables directly to the ground anchors. This turned out to be a maintenance problem. When a barrier fills with debris these points of contact are under tension (See Figure 13). Disconnecting the cables for cleaning operations is virtually impossible without cutting either the infrastructure cable or the ground anchor. Neither option is desired for it would require anchor replacement and cable replacement. The modification being implemented progressively as maintenance operations occur is to install a sacrificial connection between any ground anchor and the infrastructure. This way, when necessary, the connection can be cut with a chop saw and maintenance work can proceed.

Figure 13: Connection between infrastructure and ground anchor under tension and difficult and dangerous to disconnect.

SUMMARY
Debris flow barrier design has been evolving since 1995 (Duffy, 1996) when a debris flow impacted a flexible rockfall barrier. Since then a number of systems have been installed worldwide. Larger and larger volumes are being stopped with new and innovative designs. The
key is in the flexibility and the free draining characteristics of the systems. The case histories discussed herein illustrate that for small debris flows these systems work reliably and are maintainable. This level of service is only the beginning and hopefully this information will aid in advancing systems to apply to a broader range of situations and design load capabilities.

REFERENCES


Implementation and Development of Hybrid Rockfall Barrier Systems in Colorado

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ABSTRACT

The recent application of hybrid rockfall barriers by the Colorado Department of Transportation (CDOT) is the result of observed performance of traditional rockfall barriers to significant rockfall events. The typical hybrid systems constructed in Colorado consist of barrier panels suspended from a system of vertical posts and wire rope that is anchored into bedrock. The primary function of the system is to attenuate the energy of a rockfall impact and to reduce the energy of the rockfall event by allowing the rock pass through the attenuator system in a more controlled manner. The hybrid systems are used in conjunction with more traditional rockfall barriers and catchment areas to more efficiently retain the fallen material. In recent installations, the systems have been located above traditional rockfall barrier fences to reduce both the rockfall energy and the bounce height, which increases the performance characteristics of the lower energy barrier systems.

Field verification of the hybrid attenuator systems has been conducted along a few sections of Interstate I-70 by rolling rock into existing attenuator systems. Based on the initial field verification rolls, CDOT constructed a rockfall attenuator field verification site near Idaho Springs, Colorado. The verification site was constructed to evaluate and compare the various rockfall panels available that include cable nets, ring-nets, and high-strength chain link fence systems. Cube octahedron reinforced concrete rocks ranging from 1,500 kg to 3,600 kg were cast and dropped into the attenuator systems generating combined rotational and translational energies that ranged from 300 to 500 kJ.

The data from the verification testing will be used to verify hybrid attenuator design concepts and determine appropriate applications and design thresholds of future attenuator systems.

INTRODUCTION

A common rockfall mitigation option in Colorado consists of constructing traditional rockfall barrier systems either next to the highway or a short distance upslope of the highway. These traditional barrier systems generally consist of fence panels that are connected on all sides to a braking system and are designed to stop a single rockfall event of a given magnitude. Figure 1 shows a typical barrier installation adjacent to a roadway. These systems are used in areas where narrower ditch widths do not provide enough catchment. These barrier systems are successfully used worldwide and are typically certified by foreign government test facilities.
CDOT has installed multiple types of rockfall barrier systems at various locations throughout the state. Depending on the site conditions, these systems are functioning as they were designed. However, in certain applications near Georgetown Colorado, where rockfall rolling distances can exceed 2,000 feet, several rockfall incidents have required significant repairs or replacement of traditional barrier systems. CDOT has developed and constructed several attenuator/hybrid systems along Interstate 70 in this area. The attenuator systems are designed to mitigate rockfall events with significant rolling distances and to reduce rockfall velocities rather than attempting the stop a single rockfall event at or near the roadway. These systems are used where potential rockfall sources are located in excess of 500 feet above the roadway.

Prior to the installation of the attenuator/hybrid systems, CDOT conducted field rock rolling exercises to verify rockfall modeling using the Colorado Rockfall Simulation Program (CRSP). The rocks were rolled from source areas that ranged from 300 to 1,500 feet above the roadway. The visual observations from the rock rolling exercise indicated the following:

1. Bounce heights resulting from launching features of rolling rock were estimated between 30 to 40 feet.
2. Bounce heights in excess of 15 feet are generated after just 500 feet of rollout.
3. Rotational aspects of rolling rocks significantly contribute to the damage caused by rockfall.

As a result of the observations made during the rock rolling exercise, CDOT began to utilize rockfall hybrid/attenuator systems in conjunction with traditional rockfall barriers. The intent of the hybrid/attenuator system is to mitigate the velocity of rockfall by attenuating the rockfall energy beginning as close to the source area as possible. Rockfall hybrid/attenuator systems are
designed to be used in series where roll out distances from source areas exceed 500 feet (see Figure 2). The systems are similar to traditional rockfall barriers in construction. However, where traditional barriers are attached to a top and bottom support rope, the hybrid systems, currently in use by the CDOT, are only attached to a top support rope. A net panel is suspended from a top support rope and the excess netting is draped along the ground.

![Figure 2: Schematic of mitigation scheme](image)

Several hybrid/attenuator systems have been installed and have withstood rockfall impacts. Damage to the systems appears consistent with damage from high rotational energies associated with the rockfall event (see Figure 3).

![Figure 3: Examples of Torn Hybrid Net Panels.](image)
Until recently, mitigation systems have been constructed within CDOT Right of Way (ROW). As access to property outside of CDOT ROW is acquired, the attenuator/hybrid will be constructed in series beginning as close to the source rock as possible.

As the elevation along the slope begins to increase for the location of the systems, the maintenance and inspection efforts will also increase. Consequently, CDOT believed that testing of the systems to improve the design and constructability was necessary. In fall and winter of 2007 and 2008, testing of various styles of rockfall hybrid/attenuator systems was conducted.

ATTENUATOR TEST SITE

The attenuator test site was located near Idaho Springs, Colorado. The constraints of the test site necessitated the use of a crane to drop rocks onto a ramp to generate sufficient translational and rotation velocities for testing purposes. Figure 4 shows an aerial view of the test facility.

![Aerial view of testing facility.](image)

The testing consisted of releasing concrete rocks, suspended from a crane onto a concrete ramp and into the attenuator/hybrid being tested. The rocks were constructed by casting 14-sided cube octahedron concrete boulders that ranged from 5,000 lbs to 8,000 lbs (1,500 kg to 3,636 kg). The concrete boulders we reinforced with #4 re-bar on a 6 to 12 inch 3-dimensional spacing. Figure 5 depicts the forming setup for the cube octahedron. Figure 6 depicts the completed product.
Figure 5: Concrete Boulder Forms

Figure 6: Cast Concrete Cube Octahedron Boulders
A mechanical release system was utilized to release the concrete boulders once they were hoisted by the crane operator. Figure 7 depicts the mechanical release system.

The concrete boulders were hoisted to 30, 60 or 90 feet above the point of impact just above the ramp on the native bedrock outcrop. Once the concrete boulder impacted the natural rockslope it then was directed down the concrete ramp into an attenuator system. Figure 8 depicts a generalized cross sectional view of the test setup. Four sets of posts were placed at the site. Two posts which consisted of W 8 x 48 sections were placed 20 feet apart and were 20 feet in height. The other two posts which consisted of W 6 x 25 sections were placed 40 feet apart outside of the W 8 x 48 posts. The panels were suspended from one inch wire support ropes. One inch diameter support ropes were also used for the post anchors which were placed upslope.
Multiple net and panel configurations were tested at the facility. The systems testing consisted of the following:

- Chain link
- Cable net
- Ring net
- High strength wire nets large opening diagonal weave
- High strength small opening diagonal weave
- 8 x 10 hexagonal weave mesh

**DATA COLLECTION**

Three high speed cameras were used to collect information to determine translational and rotational velocities. The cameras collected digital video at 250 frames per second. The cameras were set on the sides and base of the rock rolling (see Figure 9).
TESTING

A total of 87 drops were completed between November 21, 2007 and January 25, 2008. Two methods were used to calculate the translational and rotational energies. The first method used the time index from the camera and known distances to determine velocity and angular rotation. The second method utilized a motion analysis software package. Currently, the video is in the process of being analyzed for the effects due to rock impacts of this nature on the various net and panel configurations used. However, initial observations indicate the following criteria to be critical to the design of hybrid systems.

- The aperture of the mesh opening of the net panel
- The weight of the net panel
- The length of netting on the ground.

It should be noted that the current design and testing are specific to rockfall mitigation along I-70 near the town of Georgetown. It is recognized that other hybrid design have been used successfully elsewhere and that the testing described in this discussion was performed for the benefit of the CDOT in their effort to improve the design of existing hybrid systems.
TESTING OF VARIOUS TYPES OF ROCKFALL FLEXIBLE WIRE ROPE MITIGATION BARRIER: AN OVERVIEW OF TESTING TO DATE

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ROTEC INTERNATIONAL, LLC,
Santa Fe, New Mexico
ABSTRACT

Testing of flexible rockfall wire rope net barriers began at Isofer, AG. (ISOSTOP) in Switzerland in the early 1980’s. Ever since, a controlled field testing program was undertaken in order to view the response of such systems under full scale rockfall kinetic energy impacts and loading. This testing was accomplished and verified with the aid of government agency experts and/or technical university personnel.

Each of the tested systems is unique in its design, kinetic energy ratings and foundation and anchoring requirements. Emphasis is placed on the advantages of applying flexible steel wire rope net systems, facilitating design and component improvements as well as overall performance optimization and of course, ease of installation in danger zones and maintenance requirements for same.
INTRODUCTION

Briefly, rockfall and debris flow occurrences are natural events. Mitigation measures which pertain to these types of environmental hazards is not a perfect science, which means scientific methods to calculate the mitigation systems cannot be used and therefore no fences and/or any other rockfall retaining systems available today will guarantee the safety of individuals and the loss of property. However, rockfall and unstable rock slopes can be controlled and the safety and risks can be reduced by the use of properly engineered and tested rockfall and debris flow retaining barriers in identified danger zones.

ISOSTOP, located in Knonau, Switzerland, is a recognized leader in the field of environmental safety. Their first fabricated flexible woven wire rope net panels were used in conjunction with a rockfall retaining barrier and tested in 1985. This test was performed under the supervision of and in collaboration with Dr. Heierli, Swiss Federal Institute of Technology (ETH), who was considered the pioneer in the field of rockfall and debris flow retaining barriers as well as snow avalanche preventive measures, at that time.

When considering the rockfall system test procedures and requirements of today, the previously mentioned test was rather simple in its execution. Multiple 1 Ton rocks were dropped 20 meters from a crane directly into the nets. (Figure: 1) A hand-woven wire rope net was mounted to a structural steel frame, using wire ropes with braking elements which were attached on all four corners to the frame. The objective of this particular test was to prove that the net would not be damaged by the impacting kinetic energy of 200 kilo-Joules (74 Foot-tons) and that the braking elements opened properly. The test passed successfully and with that accomplished, the first ISOSTOP rockfall retaining system was approved by the Swiss Government Inspection Agency, thus allowing the company to erect its first rockfall retaining barrier design to be installed in Switzerland.

Subsequently, after the successful execution of the first test in 1985, an additional total of 103 controlled rockfall barrier full scale field tests were performed by the company in Switzerland and Italy (Index Sheet: 3). Numerous types of rockfall systems having a kinetic impact energy rating as low as 100 kJ (37 Foot-tons) and barriers rated as high as 3000 kJ (1007 Foot-tons) were tested in accordance with the Swiss and European guidelines for the approval of rockfall barriers. All tests were certified with detailed test reports, which were prepared either by a government testing agency and/or by a technical university.

It is of utmost importance that testing of these types of mitigation systems be performed under strict guidance and specified test standards. Collaboration only with a competent institution and/or university which is knowledgeable about the testing procedures and guidelines and the product application, shall perform this type of testing. Backyard testing, executed by a group with little or no experience pertaining to rockfall barriers is extremely risky, insofar as the liability is quite high, especially in cases where the barriers don't live up to their expectations and worst, fail and endanger people and goods.

Over the years, research, development and testing pertaining to flexible wire rope net rockfall barriers within the company, has intensified substantially. From 1985 until about 2002, testing of
the barriers was done in the field, mostly on a 1:1 slope by simply rolling rocks and/or concrete blocks from the top of the slope into the full scale barrier. *(Figure: 2)*

*Figure: 2*  
Rockfall Barrier Installed at a 1:1 Slope Test Site

The company not only tested rockfall barriers with woven flexible wire rope nets, but also fences using ring nets made from wire ropes having the same size diameter as used for the woven nets. Interestingly, when comparing ring nets to woven nets, we saw no advantage to using the former. Rockfall barriers designs, incorporating woven wire rope nets, have proven themselves over their many years in existence and are extremely reliable and if damaged by excessive rockfall and debris flows, can easily be repaired and maintained in the field.

Alternately, testing was done by guiding and releasing weights into the rockfall barriers with the help of a specially designed monorail crane system located at the test facility. *(See Figure: 3)*
Besides the many rockfall barrier tests which were executed by ISOSTOP over the years, testing of the patented braking elements were also performed on a regular basis, using an independent test laboratory for this task. **(Figure: 4 & 5)** Basically, two types of braking element designs are available. One braking element has a single loop and is used in conjunction with the lesser kinetic impact energy rated type fence systems, whereas the double loop design is more suitable for systems retaining higher energies. **(Index Sheet: 2)** The higher the impacting energy, the more elements are needed for dampening such forces.
TYPICAL BRAKING ELEMENTS TEST RESULTS

BRAKING ELEMENT, TYPE BE 24 (WITH DOUBLE LOOP)
TEST: #1, DATE OF TEST: 4/17/2007

BRAKING ELEMENT, TYPE BE 18 (WITH SINGLE LOOP)

Index Sheet: 2
From 2002 on, ISOSTOP tested rockfall fences having kinetic energy ratings of 500 kJ (185 Foot-Ton) up to 3000 kJ (1107 Foot-Ton) at the new test facility which is operated by the Swiss Federal Office for the Environment (FOEN) and the Swiss Federal Research Institute (WSL). *(Figure: 7)*

![Image](image_url)

*Figure: 7* FOEN & WSL Test Facility in Switzerland

New test procedures and guidelines were established, defining a standardized test sequence and providing an objective comparison of the different manufacturer’s rockfall protection systems in the same energy category and in an effort to improve the effectiveness of the barrier as well as provide a useful aid to the rockfall system installer in the field. Rockfall system testing at the above mentioned facility is performed by dropping free falling concrete cubes from a "Derrick" crane into the protection kit, which is installed 15 meters above ground level at an inclination of 30 degrees to the horizontal plane of the rockface.

During the same period, additional testing was done by the Brandenburgische Techniche University Cottbus (BTU), in Switzerland and in Northern Italy.

It should be mentioned that either testing methods of rockfall barriers, such as rolling rocks from a 1:1 slope or releasing the weights from a monorail system into the rockfall barrier and/or dropping concrete blocks in a free fall fashion with a "Derrick" crane into the protection kit, are considered acceptable testing methods, as long they are properly executed and certified by a professional team.
**1000 Kj (369 FOOT-TON) ROCKFAL BARRIER TEST PROCEDURES AND TEST REPORT**

**GENERAL INFORMATION**

As mentioned earlier in this paper, since 1985, in excess of 103 rockfall barrier tests were performed by ISOSTOP, during the last 22 years. Obviously, all these tested systems cannot be described in detail in this paper.

The rockfall barrier test which we will be concentrating on, has a rating of 1000 kJ. (369 Foot-ton). This test was executed in 2007 at a test facility which is located in a quarry in Northern Italy and was supervised by the Brandenburgische Technical University Cottbus, Germany (BTU).

The testing guideline for the approval of rockfall barriers is based on the current requirements as described in the Swiss Federal Guidelines (FOEN) & (WSL) on “Model Testing of Protective Nets Against Falling Objects” and the Guidelines for European Technical Approval (ETAG). These guidelines are similar and in accordance with the “American Association of State Highway and Transportation Officials (AASHTO)”, recommended procedures for testing of rockfall barriers.

**1000 KJ (369 FOOT-TON) ROCKFALL BARRIER TEST OBJECTIVE**

The test’s objective is to measure the resistance of the ISOSTOP 1000 kJ (369 Foot-Ton) rockfall flexible wire rope safety net barrier against falling rocks and other objects, impacting the net at certain speeds and in different areas of the protective safety net system. (Figure: 8)

![Figure: 8](image)

*Figure: 8* Concrete Block, Dropped from the Crane into Test Barrier Wire Rope Net
TEST BARRIER TECHNICAL DATA

Rockfall/Debris Flow Barrier Kinetic Energy Rating: 1000 kJ
Structural Columns Center to Center Line Spacing: 5 meters
Working Height of the Structural Column: 4 meters
Woven Flexible Wire Rope Net Mesh Opening: 200 mm x 200 mm
Top & Bottom Ropes Diameter: 20 mm
Retaining Ropes Diameter: 18 mm

TEST PROCEDURES

As shown in the table below, seven successive tests were executed. Each test having different size concrete blocks weighing 1600 kg and 3200 kg being released by the “Derrick” crane from a dropping height which is either 8 meters and/or 32 meters into the protection kit. (Figure: 9) From these data, one can see that not only the woven wire rope nets themselves, but also the structural steel columns and the retaining ropes of same have been impacted during the test procedures.

<table>
<thead>
<tr>
<th>Description</th>
<th>Test 1</th>
<th>Test 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Concrete Cubes</td>
<td>3,200 kg</td>
<td>3,200 kg</td>
</tr>
<tr>
<td>Dropping Height of Cubes</td>
<td>32 m</td>
<td>8 m</td>
</tr>
<tr>
<td>Speed Recorded</td>
<td>25 m/s</td>
<td>12.5 m/s</td>
</tr>
<tr>
<td>Energy Level</td>
<td>1000 kJ</td>
<td>250 kJ</td>
</tr>
<tr>
<td>System Impact location</td>
<td>Center hit in the mid field</td>
<td>Support column impact at S3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Test 3</th>
<th>Test 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Concrete Cubes</td>
<td>1,600 kg</td>
<td>3,200 kg</td>
</tr>
<tr>
<td>Dropping Height of Cubes</td>
<td>32 m</td>
<td>32 m</td>
</tr>
<tr>
<td>Speed Recorded</td>
<td>25 m/s</td>
<td>25 m/s</td>
</tr>
<tr>
<td>Energy Level</td>
<td>500 kJ</td>
<td>1000 kJ</td>
</tr>
<tr>
<td>System Impact location</td>
<td>Center hit in the mid field, right after Test 2, without repairs to the system.</td>
<td>Center hit in the mid field, using a repaired net.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Test 5</th>
<th>Test 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date tested</td>
<td>5/9/2007</td>
<td>7/12/2007</td>
</tr>
<tr>
<td>Weight of Concrete Cubes</td>
<td>3,200 kg</td>
<td>3,200 kg</td>
</tr>
<tr>
<td>Dropping Height of Cubes</td>
<td>8 m</td>
<td>32 m</td>
</tr>
<tr>
<td>Speed Recorded</td>
<td>12.5 m/s</td>
<td>25 m/s</td>
</tr>
<tr>
<td>Energy Level</td>
<td>250 kJ</td>
<td>1000 kJ</td>
</tr>
<tr>
<td>System Impact location</td>
<td>Column retaining rope impact at S3</td>
<td>Center hit at one of the adjacent end fields.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Test 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date tested</td>
<td>19/12/2007</td>
</tr>
<tr>
<td>Weight of Concrete Cubes</td>
<td>3,200 kg</td>
</tr>
<tr>
<td>Dropping Height of Cubes</td>
<td>32 m</td>
</tr>
<tr>
<td>Speed Recorded</td>
<td>25 m/s</td>
</tr>
<tr>
<td>Energy Level</td>
<td>1000 kJ</td>
</tr>
<tr>
<td>System Impact location</td>
<td>Center hit in the mid field at a working height of 3 meter.</td>
</tr>
</tbody>
</table>

Note: In regards to the actual concrete blocks impact location within the test installation, refer to: (Index Sheet: 2)
DESCRIPTION OF THE ROCKFALL BARRIER TEST FACILITY

The rockfall system’s structural posts and flexible woven wire rope netting was fastened with support ropes incorporating numerous braking elements to the test installation steel frame, which is a permanently anchored structure to the rockface at the test station. (Figure:10)

Figure: 10  Test Facility in Northern Italy

The concrete blocks are weighted and are lifted precisely into position to be released into the nets with the use of a remote controlled “Derrick” crane, equipped with a trigger mechanism. The inclination of the wire rope safety net is approximately 30 degrees in relation to the horizontal plane of the rock face. Load cells which were polled by an electronic amplification and data logging system, were attached to the net retaining wire ropes at various positions. Measurements therefore could be recorded safely and continuously. Similar types of load cells were mounted at the load bearing wire rope anchor location. The impacting speed of the free falling concrete cubes was captured and recorded using two high speed video cameras. One camera was mounted at a slight angle toward the test installation and the second camera was installed in front of the installation, approximately 12 meters below the test set-up. All of the system’s braking elements were marked with color spray in such a fashion that the net support rope movement could be measured after the net was impacted by the concrete block during testing.
FINAL TEST DATA AND MEASUREMENTS

Figure: 11, refers to the dropping heights, speeds and the actual weights of the concrete cubes which were released into the net system during the seven tests performances. From this data, the kinetic impact energy as indicated in this table was then calculated.

<table>
<thead>
<tr>
<th>Test</th>
<th>Release Height of Mass</th>
<th>Weight of Mass</th>
<th>Impact Speed</th>
<th>Kinetic Impact Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unit [m]</td>
<td>[kg]</td>
<td>[kg]</td>
<td>[m/s]</td>
</tr>
<tr>
<td>1</td>
<td>32</td>
<td>3,200</td>
<td>3,240</td>
<td>25.00</td>
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<tr>
<td>2</td>
<td>8</td>
<td>3,200</td>
<td>3,220</td>
<td>12.50</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
<td>1,600</td>
<td>1,640</td>
<td>25.00</td>
</tr>
<tr>
<td>4</td>
<td>32</td>
<td>3,200</td>
<td>3,220</td>
<td>25.00</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>3,200</td>
<td>3,220</td>
<td>12.50</td>
</tr>
<tr>
<td>6</td>
<td>32</td>
<td>3,200</td>
<td>3,220</td>
<td>25.00</td>
</tr>
<tr>
<td>7</td>
<td>32</td>
<td>3,200</td>
<td>3,220</td>
<td>25.00</td>
</tr>
</tbody>
</table>

Figure: 12, refers to the tested and calculated data, which pertain to the structural steel columns foundation forces for different columns heights and location of same within the fence system.

Figure: 13 shows measurements and calculations such as the stopping distances of the impacting concrete blocks after being dropped into the flexible wire rope net. The net’s usable height after the fall of the different weights and the final maximum calculated kinetic energy, retained within the system.
**Figure: 14,** showing the different wire rope forces which were recorded during the test, using electronic load cells with a frequency of 1200 values/second.

<table>
<thead>
<tr>
<th>Test Nr.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
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<tbody>
<tr>
<td>Position</td>
<td>Max. measurements in kN</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.0</td>
<td>1.0</td>
<td>0.8</td>
<td>0.6</td>
<td>0.0</td>
<td>6.0</td>
<td>17.5</td>
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<tr>
<td>II</td>
<td>22.9</td>
<td>2.4</td>
<td>26.7</td>
<td>0.0</td>
<td>2.0</td>
<td>59.5</td>
<td>39.3</td>
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<tr>
<td>III</td>
<td>72.4</td>
<td>18.1</td>
<td>78.0</td>
<td>71.1</td>
<td>57.8</td>
<td>34.0</td>
<td>64.4</td>
</tr>
<tr>
<td>IV</td>
<td>51.4</td>
<td>20.9</td>
<td>53.1</td>
<td>48.7</td>
<td>40.8</td>
<td>57.2</td>
<td>-*</td>
</tr>
<tr>
<td>V</td>
<td>60.3</td>
<td>52.9</td>
<td>13.4</td>
<td>3.2</td>
<td>37.1</td>
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<td>VI</td>
<td>64.2</td>
<td>43.8</td>
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<td>VII</td>
<td>21.6</td>
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<tr>
<td>VIII</td>
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<td>2.1</td>
<td>4.5</td>
<td>10.7</td>
<td>10.2</td>
<td>2.9</td>
<td>14.7</td>
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<tr>
<td>IX</td>
<td>137.8</td>
<td>111.1</td>
<td>141.85</td>
<td>121.9</td>
<td>170.2</td>
<td>159.2</td>
<td>120.6</td>
</tr>
<tr>
<td>X</td>
<td>94.3</td>
<td>27.7</td>
<td>92.81</td>
<td>109.8</td>
<td>91.9</td>
<td>178.8</td>
<td>121.6</td>
</tr>
<tr>
<td>XI</td>
<td>119.0</td>
<td>48.2</td>
<td>82.9</td>
<td>97.4</td>
<td>99.5</td>
<td>101.9</td>
<td>152.6</td>
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<tr>
<td>XII</td>
<td>114.8</td>
<td>93.1</td>
<td>104.4</td>
<td>149.8</td>
<td>91.0</td>
<td>108.9</td>
<td>100.8</td>
</tr>
</tbody>
</table>

*Measurement intake connection damaged during test, no set of measurements exists.*
CONCLUSION

A variety of tested and certified rockfall/debris flow barriers of different designs are available on today’s market. It is therefore of importance that the systems are properly evaluated and compared with each other in accordance with how they are being designed, tested and manufactured.

- For instance, what is the distance from centerline to centerline of the net support column? Larger column distances will require fewer posts and foundations of same. Also, not as many retaining wire ropes and anchors are required for a particular project. In addition, it is a good thing to compare the column foundation forces. The lower the forces, the less foundation work is required.
- Check if some of the fence components such as the braking elements and the fence retaining, bracing and top and bottom net support ropes are factory assembled, which will simplify the fence installation in the field.
- Check as to how many anchors are needed to secure the barrier at the danger zone safely? Data obtained while testing the fence system, such as the wire rope net forces will be the determining factor for calculating the anchor’s pullout strength and the quantity of anchors needed for a particular fence system installation. Naturally, if the anchor requires a lesser pullout strength, the size of the anchor and drilling depth required to secure the anchor in the ground can be reduced, which would amount to a substantial saving.

It should be noted that some of the above mentioned information can be obtained on the internet. FOEN has posted all the test results by different fence manufacturers which were executed at their test facility in Switzerland, in detail on their website.

The fence systems, besides being used for catching falling rocks, etc., are also used to mitigate a variety of other problems. For example, to catch falling tree stumps, which is not uncommon in forested areas and if not handled properly, can create many disturbances. Woven flexible wire rope nets barriers seem to be most suitable for this type of application. (Figure: 15)
In closing, all of ISOSTOP's disaster mitigation systems are manufactured using the best available materials on hand. The system designs incorporate lightweight, tested components in order to simplify the installation in the field and at the same time give the system an esthetically and environmentally pleasing appearance. (Figure: 16)

![Figure: 16](Image)

**Typical Environmentally Pleasing Rockfall Barrier**

**WHAT WILL THE FUTURE BRING?**

Extensive testing of new designs of natural disaster mitigation types of systems and repeat testing of systems which are currently available on the market in order to improve and update the design and to optimize the performance of all these systems, is absolutely mandatory.

It is the company's intention to field test a 5000 kJ (1845 Foot-ton) rockfall/debris flow system in the very near future. Also, new and specially designed mitigation systems of lesser energy ratings, using different types of fabrics than the woven wire rope nets to retain impact energies of falling rocks and other masses will be designed and tested during the coming years. Other than that, systems component testing, such as the braking elements, special wire ropes and all other types of hardware, will be done on a regular basis.
ROCKFALL BARRIER TESTS PERFORMED BY ISOFER AG, SWITZERLAND

<table>
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<tr>
<th>Test</th>
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NOTES:

**ETH** : Swiss Federal Institute of Technology

**WSL** : Swiss Federal Research Institute

**FOEN** : Swiss Federal Office for the Environment

**BTU** : Brandenburgische Technical University, Cottbus

Index Sheet: 3
**References:**

- Dr. Heierli, Swiss Federal Institute of Technology (ETH)  
  1985: Executed and certified first rockfall woven wire rope net and braking elements test in Switzerland.

- A. Boll, Swiss Federal Research Institute (WSL)  
  1988-1990: Executed and certified numerous full scale rockfall barrier tests in Switzerland.

- W. Gerber, Swiss Federal Office of the Environment (FOEN) & Swiss Federal Research Institute (WSL)  
  1991-2006: Assisted and certified numerous full scale rockfall barrier tests in Switzerland.

- N. Wienberg, Brandenburgische Technical University, Cottbus (BTU)  
  2007: Assisted and certified full scale rockfall fence tests in Northern Italy.

- M. Toniole, Isofer, AG, (ISOSTOP) Knoau, Switzerland, Manager for Natural Disaster Mitigation System.  
  Supervised numerous rockfall and debris flow tests in Switzerland and Italy.
REVISITING AN OLD PROJECT WITH NEW TECHNOLOGY—DIGITAL TERRAIN MODELING AND MULTI-LAYERED VIRTUAL GEOLOGIC HAZARD MAPPING ALONG A PROPOSED HIGHWAY REALIGNMENT, RIO GRANDE GORGE, NEW MEXICO

William C. Haneberg
Haneberg Geoscience, 10208 39th Avenue SW. Seattle WA 98146
bill@haneberg.com
ABSTRACT

The utility of digital terrain modeling and virtual engineering geologic mapping for highway alignment studies is illustrated using modern multi-layered digital terrain modeling and virtual mapping techniques to revisit a 1992 along the Rio Grande gorge between Rinconada and Pilar, New Mexico. The original study used data and tools of the day— an enlarged 1:24,000 topographic map base, black and white aerial photographs, and analog tools such as mirror stereoscopes and a zoom transfer scope— to build upon existing geologic quadrangle maps with project-specific fieldwork and produce two map products: an engineering geologic map and an interpretive geologic hazard map. The work described in this paper included development of a suite of shaded relief images, a suite of topographic derivative maps that accentuate aspects of the landscape of interest in engineering geologic studies, and synthesis of the digital terrain data with satellite imagery and existing geologic maps to produce a landslide hazard map showing features that could not have been elucidated using the data and tools available in 1992. In particular, digital terrain modeling and virtual mapping allowed delineation of geomorphologically distinct sub-units within previously mapped large-scale landslide deposits that may provide information about relative ages and reactivation. The techniques described in this paper are broadly applicable over scales ranging from detailed site-specific studies using airborne lidar to create very high-resolution digital elevation models to regional studies using satellite-derived topographic data.
INTRODUCTION

In 1992, the New Mexico State Highway and Transportation Department contracted with the New Mexico Bureau of Mines & Mineral Resources, the state’s geological survey, to evaluate geologic hazards along a proposed realignment of State Highway 68 through the Rio Grande gorge between the settlements of Rinconada and Pilar. This stretch of two lane highway, which is sandwiched between the Rio Grande to the northwest and the Pilar Cliffs to the southeast, had historically been subjected to debris flows, rockfalls, rockslides, and localized slumps. The alternative alignment evaluated in 1992 traded the existing geologic hazards for a set of different hazards that included large rotational landslides known locally as Toreva block slides, earthflow complexes, and hydrocompactive soils developed in weak Tertiary sediments and sedimentary rocks on the northwest side of the Rio Grande.

Engineering geologic and geologic hazard assessment maps for the realignment study were prepared using standard technology at the time: a 1:12,000 base map photographically enlarged from 1:24,000 USGS topographic sheets, non-rectified aerial photographs, colored pencils, and analog tools such as a zoom transfer scope. Mapping, which was led by the author, was field based with office refinement.

This paper revisits the original project to examine how modern digital tools and techniques such as digital elevation models (DEMs), quantitative terrain modeling, and office-based virtual mapping using multiple georeferenced data layers can be used to gain greater insight into geologic conditions affecting highway alignment selection and leverage the value of fieldwork by using virtual mapping techniques. Techniques to be discussed will include the use of multiple shaded relief images to accentuate geomorphic features, applications of topographic derivative maps (e.g., slope angle, aspect, curvature, roughness), stacking of map layers to produce composite base maps for geologic interpretation, virtual geologic mapping, and the use of GIS and scientific visualization software to convey information about geologic hazard distributions.

GEOLOGIC SETTING

The geologic setting of the area surrounding the proposed highway corridor described in this paper is depicted in a number of regional scale reports and maps, most notably Kelley (1978), and 1:24,000 geologic maps of the Carson and Trampas 7.5’ quadrangles (Bauer and Helper, 1994; Kelson and Bauer, 1998; Bauer et al, 2005). Regional geologic hazards— which include various forms of slope instability, hydrocompactive soils, and low to moderate levels of seismicity— are summarized in Haneberg (1992 a,b) and Haneberg et al (2002).

The Rio Grande follows the Embudo fault, a high-angle Neogene structure with left-lateral movement that juxtaposes Tertiary rocks of the Taos Plateau to the northwest with Proterozoic rocks of the Picuris Mountains to the southeast, adjacent to the proposed highway corridor. Estimated structural relief of about 3 km across the Embudo fault (Muehlberger, 1979) has led to the development of the Pilar Cliffs, a 5-km-long and 300-m-high escarpment developed in highly fractured and sheared Proterozoic metavolcaniclastic and metasedimentary rocks. The sheared
and fractured rock of the Pilar Cliffs commonly produces rockfalls, rockslides, and debris flows that affect State Highway 68 during and immediately after heavy rainstorms. One notable storm in 1991 spawned a debris flow that temporarily dammed the Rio Grande and restricted flow for several years afterward and destabilized a 200 ton block of schist that slid or rolled down the cliffs, left a crater in State Highway 68, and came to rest along the opposite side of the Rio Grande. Haneberg and Bauer (1993) estimated that the block was traveling about 21 m/s (48 miles/hour) when it struck the road.

To the northwest of the fault and river, basement rocks are covered by Tertiary sedimentary rocks associated with the opening of the Rio Grande rift. The Miocene to Pliocene Santa Fe Group, here represented at the surface by the Chamita Formation, consists of poorly sorted and weakly indurated gravel and sand beds ranging and ranges in thickness from 150 m to 1100 m. Unlike lower lying members of the Santa Fe Group, the Chamita Formation is not known to contain localized silt or clay strata that might contribute to slope instability.

The Santa Fe Group is capped by tholeiitic basalt flows of the Pliocene Servilleta Formation. Limited exposures of highly brecciated Proterozoic schist and quartzite along the northwest side of the Rio Grande help to constrain the location of the Embudo fault along the right bank of the river.

Down-cutting by the Rio Grande, perhaps abetted by a wetter climate and occasional earthquakes, has led to the development of large rotational slide masses known in the southwestern United States as Toreva blocks, named after their type locality near Toreva, Arizona (Reiche, 1937). The slides are extensive enough to be shown on regional as well as 7.5 minute geologic maps (Kelley, 1978; Bauer and Helper, 1994; Kelson and Bauer, 1998; Bauer et al, 2005). Investigations of lacustrine deposits along the Rio Grande gorge near Los Alamos, about 50 km southwest of the study area, have shown that landslide dammed lakes persisted for hundreds of years during the wetter and cooler late Pleistocene Epoch (Reneault and Dethier, 1996).

Southwest of the study area, oversteepening of slopes underlain by Santa Fe Group deposits and capped by the Servilleta Formation has produced extensive deposits of landslide debris along the valley floor and along the route of State Highway 68. Weathering of the landslide debris and loosening of large basalt boulders derived from the Servilleta Formation caprock has given rise to rockfalls along the highway, including a 1988 event resulting in five fatalities, and wire rope rockfall protection nets have been installed in several places along the road. Hydrocompaction is also common in places where the valley widens and the highway crosses broad alluvial fans of reworked Santa Fe Group sediments south of Velarde.

**PREVIOUS WORK**

The original 1992 project was a reconnaissance study based on aerial photograph interpretation, limited field mapping with office refinement, and synthesis of information from geologic quadrangle maps then in preparation. Access to the proposed corridor was limited and only by
foot. Drilling was limited to two hand auger holes in potentially hydrocompactive silts and sands. A 1:12,000 enlargement of standard U.S. Geological Survey 1:24,000 topographic quadrangles was used for the map base and supplemented by black and white stereo aerial photographs supplied by NMSHTD. Engineering geologic map units were refined in the office by using a mirror stereoscope to identify landforms and transferring contacts to the topographic base using a zoom transfer scope. Although some digital elevation models were available at the time, they were comparatively crude. Perhaps more importantly, the GIS software and technical expertise necessary to make use of digital elevation models were not available at NMBMMR during the early 1990s.

Results of the 1992 study are described in Haneberg et al (1992, 2002) and briefly summarized here to provide background for the analysis undertaken for this paper. At the time of the original study, a 1:24,000 geologic map of the Trampas quadrangle was in press and a 1:24,000 geologic map of the adjoining Carson quadrangle was in preparation (Bauer and Helper, 1994; Kelson and Bauer, 1998). Mapping for the original project, which was limited to an 0.8 km (½ mile) wide corridor centered around a proposed highway alignment supplied by NMSHTD, was undertaken using a multi-level approach that added an engineering geologic map and an interpretive geologic hazards map to information contained in the standard geologic quadrangle maps.

The 1992 engineering geologic map used Unified Engineering Geologic Mapping System (UEGMS), then known as the Genesis-Lithology-Qualifier or GLQ system, described by Keaton (1984) and Keaton and DeGraff (1996). Figure 1 shows a slightly simplified and redrafted color version of the 1992 engineering geologic map draped over a shaded relief image produced from a 10 m digital elevation model that was not available at the time of the original study. The original map is available online as part of Haneberg et al (1992). UEGMS map units, which can be stacked to represent the local stratigraphy (including the thickness of each unit if known), shown in Figure 1 are:

- **Ss-b(ro)** — Landslides (S) of sand through boulders (s-b) and with evidence of rotational movement (ro).

- **Ss-b** — Landslides (S) of sand through boulders with unspecified kinematics.

- **Soc-b** — Landslides (S) of organic soil and clay through boulders (oc-b) with unspecified kinematics. In this project, these units were active earthflow complexes.

- **Roc-m** — Residual (R) deposits of organic soil and clay through silt (oc-m). In this project, these deposits comprised wet meadows with characteristics of the Roc-b deposits except for scars and other features indicative of landsliding.

- **Cb(ta)** — Colluvium (C) composed of boulders (b) with a talus slope morphology (ta). In this project, this unit described boulder fields derived from the Servilleta basalt caprock.
hCms— Colluvial (C) slope wash of silt and sand (ms) with hydrocompactive (h) potential. Kelson and Bauer (1998) and Bauer et al (2005) mapped these as alluvial deposits.

As-b(fp)— Alluvium (A) of sand through boulders (s-b) with floodplain (fp) morphology.

As-b(te)— Alluvium (A) of sand through boulders (s-b) with terrace (te) morphology.

Because the UEGMS units do not convey information about potentially hazardous conditions (with the exception of the h modifier for potentially hydrocompactive deposits), an interpretive geologic hazard map was prepared to convey qualitative information about such issues as relative slope stability (including the potential for reactivation), hydrocompactive potential, rockfall source potential, and liquefaction potential. The geologic hazards map is available in Haneberg et al (1992) and a simplified version was published in Haneberg et al (2002).
Figure 1. Redrafted and slightly simplified version of the engineering geologic map prepared as part of the original 1992 study, draped over a shaded relief image produced from a US Geological Survey 10 m digital elevation model that was not available in 1992. The proposed highway realignment is shown in red. The original map is available in Haneberg et al (1992).
DIGITAL TERRAIN MODELING AND VIRTUAL GEOLOGIC MAPPING

Concept and History

The virtual mapping technique described in this paper was originally developed to leverage the value of airborne lidar topographic data obtained to support landslide hazard mapping on a remote island in Papua New Guinea (Haneberg et al, 2005; Haneberg, 2007a). In that project, only two weeks were available between electronic delivery of the lidar data and fieldwork. Digital terrain modeling and virtual mapping allowed the project team to create a digital elevation model optimized for mapping in the jungle covered volcanic terrain and produce a provisional landslide hazard map during that very limited timeframe. Subsequent fieldwork was limited to field verification of key areas adjacent to a major gold mine. The underlying idea is that stacking spatial data layers is synergistic and using stacked data provides more insight than examination of each layer separately. Variations on the technique have been used to produce:

- Engineering geologic maps and process-based seismic and static landslide hazard models of steep and heavily forested Mt. Sutro in San Francisco, using very high-resolution airborne lidar data commissioned specifically for that project (Haneberg, 2007a).

- Tectonic maps to provide the geologic context for a hydrogeologic and geotechnical characterization project in a quarry located along the San Andreas fault using off-the-shelf commercial airborne radar topographic data (unpublished data).

- Fault and landslide maps along a natural gas pipeline corridor across thrust-faulted karstic limestone and volcanic rocks of the tectonically active Papua New Guinea highlands using airborne lidar data, including reprocessing of the original lidar point cloud data to optimize it for geologic hazard mapping (unpublished data).

- A high-resolution surficial geologic map of glacial terrain in the Seattle area using publicly available lidar data from a regional consortium (Troost et al, 2006).


Because the method relies heavily on digital elevation models, emphasis is on identification of landforms that are either significant by virtue of their origin (e.g., landslides) or their role as guide structures (Johnson et al, 2004) that indirectly reflect the deformation and perhaps structures beneath Earth’s surface (e.g., zones of en echelon depressions or ridges). Outcrop observations, borehole information, and color aerial photos or multispectral/hyperspectral imagery can add additional information about other potentially important aspects such as
lithology, mineralization, soil development, degree of weathering, the local stratigraphic sequence, and structural relationships.

**Virtual Mapping Layers**

The virtual mapping approach described here uses a hierarchy of layers, each consisting of one or more maps that can be alternated, to create an almost infinite number of combinations useful for engineering geologic or geomorphologic mapping. This is, in essence, a modern and more flexible digital implementation of the traditional practice of layering maps on a light table. Figure 2 illustrates the hierarchy using a series of representative, but not exhaustive, thumbnail images. The fundamental piece of information required for virtual mapping is a sufficiently detailed digital elevation model. A 10 m digital elevation model obtained from the US Geological Survey National Map Seamless Server (http://seamless.usgs.gov) was used to create the examples shown in this paper. The 10 m designation refers to the grid size, which is to say that a 10 m digital elevation model supplies elevations at points separated by 10 m on a regular grid. Although a freely available 10 m digital elevation model is useful for the purpose of this paper, in practice a standard- to high-resolution lidar-based digital elevation model, ideally scoped and specified with geologic applications in mind, with 1 m or 2 m resolution would be expected for a project being undertaken today.
Figure 2. Schematic illustration of layers used for the virtual mapping technique described in this paper.
Figure 3. Shaded relief images illustrating the effects of varying the simulated illumination. A) Illumination from 315°/30°. B) 045°/30°. C) Composite image created by adding five images with illumination ranging from 270°/30° through 090°/30° in 45° increments. D) Vertical illumination.
Figure 4. Selected topographic derivative maps. A) Slope aspect (in degrees measured clockwise from north). B) Slope angle (in degrees). C) Topographic roughness with units of \( \pm \log \) meters as defined by Haneberg et al. (2005) and Haneberg (2007 b). D) Topographic index as defined in equation (1). In this example, all of the derivative maps are enhanced by draping them over the composite shaded relief image shown in Figure 3C.
The base layer (Layer 1) typically consists of a suite of shaded relief images with different simulated illumination azimuths, inclinations and, if appropriate, simulated vertical exaggeration. These are typically generated with simulated illumination from 270°, 315°, 000°, 045°, and 090° with the inclination of the simulated light source chosen to maximize textural elements within the landscape while minimizing shadows. Illumination from the south is generally avoided because it often produces the optical illusion of inverted topography, which can be difficult to interpret (although in some cases it can be useful). Shaded relief maps are additive, so composite shaded relief images can be developed by adding together two or more directional images using the map algebra capabilities of GIS programs. Figure 3 compares two directional shaded relief images (illumination from 315° and 045°) with a composite image (sum of all images from 315° through 000° to 045°) and an image with overhead lighting. Note that each image accentuates different aspects of the landscape.

Layer 2 consists of a suite of topographic derivative maps. These maps do not contain any information not already in the digital elevation model, but rather recast or present the digital elevation data in ways that accentuate certain aspects of the topography. Typical topographic derivatives include aspect (the direction in which a slope faces), slope angle, and various measures of curvature (e.g., Burrough and McDonnell, 1998). Topographic roughness, which represents the local variability of the land surface, can also be a useful derivative, although there is no standard definition or measure (Barnett et al, 2004; McKeen and Roering, 2004; Haneberg, 2007; Grohman et al, 2007). The example shown in this paper was calculated using the method of Haneberg et al (2005) and Haneberg (2007 b), which is simple, robust, and easy to implement using functions available in most GIS programs. In situations where bedrock predominates and surficial deposits are minimal or non-existent, the combination of slope aspect and slope angle may represent the strike and dip of large-scale discontinuities such as bedding planes or pervasive joint sets (Jaboyedoff et al, 2007).

For projects in which airborne lidar topographic data are available, ground strike density and return intensity maps are useful additions to Layer 2. Ground strike density maps can help to assess the reliability and level of detail of lidar-derived topographic products. In particular, it is important to realize that lidar data will not depict features smaller than the ground strike spacing in an area regardless of the nominal resolution of the digital elevation model. Lidar return intensity, although uncalibrated, can provide information about the properties of the ground surface. Rock outcrops, for example, may be more reflective than surrounding materials.

More sophisticated derivatives appropriate for Layer 2 include properties such as the topographic index, defined as (e.g., Quinn et al, 1995)

\[ TI = \ln \left( \frac{\bar{a}}{\sqrt{\tan \beta}} \right) \]  

(1)

in which \(\bar{a}\) is the area contributing shallow subsurface groundwater flow to an individual raster and \(\beta\) is the slope angle at that raster. The topographic index reflects the predicted wetness of a raster as the ratio of the area contributing water to the ability of water to be drained by
gravitationally driven shallow groundwater flow and may be useful for analyzing the delivery of surface water and shallow groundwater to a slope. Various measures of slope stability can also be calculated as derivative layers although, strictly speaking, they can require information not contained in the digital elevation model. For example, a digital elevation model can be combined with a soils map, forest cover map, and knowledge of geotechnical properties to perform a process-based probabilistic infinite slope analysis for static and seismic conditions across an entire project area (Haneberg, 2007a; Haneberg, 2004). Threshold criteria can also be used to create variations on derivative maps, for example to isolate all areas with slopes above (or below) a specified level. Figure 4 shows four topographic derivative maps—slope aspect, slope angle, topographic roughness, and topographic index—calculated from the 10 m US Geological Survey digital elevation model encompassing the project area.

Layer 3, which is optional, can include different types of supporting raster information such as aerial orthophotos, multispectral or hyperspectral satellite imagery, or geophysical potential maps (gravity, electromagnetic, etc).

Layer 4 comprises vector overlays that can include topographic contours (with different contour intervals and/or degrees of smoothing), faults or lineaments, project boundaries, cultural features such as roads buildings, and point data such as outcrop or borehole locations. The combination of vector contours with shaded relief images and another raster layer such as slope angle, for example, can create a composite map that accentuates landforms of interest in engineering geologic studies (Figure 5). The contour interval chosen can affect the geologic utility of a contour map, and in some cases it may be useful to produce several maps with different contour intervals. As shown in Figure 5, the 5 m contours that are useful for accentuating landslide topography on the northwest side of the Rio Grande are too dense to reveal detail in the much steeper Pilar Cliffs on the southeastern side of the river.

The uppermost Layer 5 includes one or more maps developed by interpreting combinations of the underlying layers. Depending on the project, this might include structural geology or tectonic elements, landslides and related features, geomorphic surfaces such as stream terrace levels, or a general engineering geologic map. Maps in Layer 5 are typically drawn by alternating or shuffling the underlying layers in a way that allows a trained geologist to delineate landforms of interest based on their geometric and textural signatures, and then drawing geologic features on the uppermost or active layer. For example, landslides might be identified on the basis of scarps (steeper than average slopes), diagnostic contour patterns, morphology on shaded relief images, and contrasting roughness relative to surrounding terrain. The relative ages of different landslides might be further estimated by comparing the steepness of scarps or surface roughness. As such, it is a qualitative virtual extension of the logical process used to map landforms in the field rather than an attempt to replace geologic experience with statistical measures or computationally driven classification of landforms. It is best to perform virtual mapping using drawing software that supports import of georeferenced raster and vector files (including support for different projections and map datums). Canvas (GIS version) and Map Publisher (which works with Illustrator and Freehand) are two commercial options. Otherwise, each layer will
have to be projected into a common datum and coordinate system using GIS software and imported into a drawing or CAD program.

Figure 5. Composite shaded relief image from Figure 3C without (left) and with (right) vector contour lines added. Contour interval is 5 m.

Figure 6 shows an intermediate map product combining linear and curvilinear features identified using several shaded relief images and derivative maps, the Embudo fault as mapped by Bauer et al (2005) and Kelson and Bauer (1998), a topographic roughness map, and a shaded relief image. The linear and curvilinear features were identified by alternating the underlying shaded relief images and topographic derivative maps, and drawing the linear features as lines on the active drawing layer. If desired, separate layers can be created for linear/curvilinear features identified on different shaded relief images and/or topographic derivative maps. In this example, the results from all combinations of underlying layers were combined on one map. As indicated on the map, differences in roughness magnitude and distribution may provide information about the relative ages of landslide features. For example, the younger Toreva block slide shown in Figure 6 has well defined scarps and a relatively small proportion of smooth topography compared to the older Toreva block slides, which would have been difficult to recognize on the ground or using topographic maps alone.

Figure 7 shows another Layer 5 map, in this case an interpretive landslide map in which a combination of linear features, roughness, slope angle, and shaded relief images were used to identify several landslide domains based solely on surficial expression. A public domain Landsat false color image was used to identify wet areas associated with active landslides, which stand out in semi-arid country by virtue of their lush vegetation. The landslide domains on Figure 7 are: intact (albeit erosionally degraded) rotational Toreva block slides, disrupted Toreva block...
slides that may indicate more recent (but not current) episodes of movement, active landslides inferred corresponding to wet areas with fresh scarps, and incipient Toreva block zones that may represent areas undergoing renewed movement.

Figure 6. Interpretive map showing linear features identified by alternating the shaded relief maps and topographic derivative maps, and drawing linear or curvilinear features as vector features. The Embudo fault was traced from another underlying layer containing existing geologic maps by Kelson and Bauer (1998) and Bauer et al (2005). The linear features are draped over the topographic roughness map (Figure 4C) and composite shaded relief image (Figure 3C). Older, younger, and disrupted Toreva block landforms can be discriminated on the basis of curvilinear feature distribution and topographic roughness patterns.
SUMMARY AND DISCUSSION

Comparison of the landslide units from an engineering geologic map prepared using methods commonly available in 1992 (Figure 1) with those in Figure 7 show that the use of high-resolution digital elevation models can allow engineering geologists to map potential hazards more efficiently, in more detail, and with greater insight than was possible even 16 years ago. In
particular, digital terrain modeling and virtual mapping allowed delineation of differences in geomorphology that may be related to the relative ages of landslides and their reactivation potential (or current activity) to a degree that would not have been possible using the topographic contour maps and aerial photos available during the original 1992 study. Had time and space allowed, similar virtual maps could have been prepared to identify small sedimentary basins containing potentially hydrocompactive soils or potentially liquefiable alluvium on the basis of elevation, slope, roughness, contour closure, and/or spectral reflection. The advantages of terrain modeling and virtual mapping are further multiplied in areas where climate, rough terrain, or remoteness may significantly limit opportunities for fieldwork. Unlike purely statistical or mechanistic approaches that attempt to eliminate geologic expertise and inference under the guise of removing bias in favor of supposedly more objective criteria, the virtual mapping approach advocated in this paper is built upon the same logical processes used for decades by field geologists. The difference is that important aspects of the geomorphology of a project area can be amplified, accentuated, and combined to help identify potential geologic hazards (or, conversely, routes or corridors that minimize exposure to hazards).

It is important to emphasize that virtual mapping is not intended to replace traditional fieldwork, but rather the supplement it and leverage the value of both the data and field time, both of which can be expensive. As with digital rock slope modeling and virtual mapping (e.g., Haneberg, in press; Haneberg et al, 2006), there will always be important information that cannot be gleaned from the geometry of surface models alone regardless of the degree of processing or manipulation. Although tools such as multispectral or hyperspectral imaging may be able to provide some information about the chemical composition or mineralogy of surficial materials, fieldwork is still necessary to ascertain details of rock texture and type, outcrop-scale structures, in situ rock quality, and stratigraphy.

A higher resolution lidar or photogrammetric digital elevation model, had one been available, would have added to the value of the hazard assessment by providing the ability to identify smaller features and produce more detailed maps. In practical terms, the smallest landforms that can be identified and mapped as such have linear dimensions about an order of magnitude greater than the digital elevation model resolution or grid spacing (Haneberg, 2008, unpublished data). Thus, the smallest feature that one might expect to map using a 10 m digital elevation model would be about 100 m x 100 m = 10,000 m$^2$. A 2 m lidar digital elevation model, in contrast, would allow mapping of landforms on the order of 20 m x 20 m = 400 m$^2$. Features smaller than that may appear as anomalously high or low patches, but will likely be indistinguishable as specific landforms. Lidar ground strike density maps provide additional information about the reliability of digital elevation models by clearly showing how ground strike density (or its reciprocal, average ground strike spacing) varies across a project area. Lidar return intensity maps can also be useful tools for identifying features such as bare rock surrounded by less reflective vegetation.

Although virtual mapping makes use of quantitative data and intermediate products such as topographic derivative maps, its products are fundamentally qualitative. For example, the hazard map in Figure 7 does not include quantitative estimates of slope hazard (e.g., factors of safety or
reliability indices) or the absolute ages of landslides. It is possible, however, to combine qualitative virtual maps with quantitative geologic hazard models to provide an integrated hazard assessment that considers rare or unprecedented conditions such as unusually wet conditions or large earthquakes. Haneberg (2007a; also see Haneberg, 2004), for example, discussed how a process-based probabilistic shallow slope stability model PISA-m was used in conjunction with qualitative engineering geologic mapping in a landslide hazard assessment in San Francisco. Other possibilities include empirical or process-based predictions of rockfall hazards (e.g., Guzzetti et al, 2004; Lan et al, 2007), debris flows (e.g., Mergili and Fellin, 2007), karst (e.g., Lyew-Ayee et al, 2006), and soil erosion (e.g., Mitasova et al, 1996). The best approach will almost always be one that combines fieldwork, virtual mapping, and quantitative modeling to perform an integrated hazard assessment.

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In-situ Shear Testing of Soft Rocks for Remediation of an Unstable Slope

By

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Abstract

A 50-year old, 2500-foot long segment of SR-7 south of Marietta, Ohio, on a hillside above the Ohio River, was plagued by periodic rock falls and landslides. Periodic lane closures required maintenance by the Ohio Department of Transportation (ODOT). Heavy rains in early 2004 caused extensive rock falls and slides closing the southbound lanes for regrading and concrete barrier installation. After reopening, additional rock falls damaged the barrier and a passing vehicle, again closing the lanes. HDR performed a geotechnical investigation to provide a more permanent solution. Construction began four months later, after the investigation and remedial design were completed. Route 7 was completely reopened in December 2005.

The problem was caused by poor quality “Mudstone” formations subject to strength deterioration over time when exposed to air and moisture. Good test samples were hard to obtain, typically resulting in conservative analyses assumptions. Borehole Shear Testing was used to evaluate the strength of these formations in various states of decomposition for use in remediation schemes. The selected scheme used on-site rock to construct a buttress, significantly reducing the amount of off-site waste. Regular interaction with the Department during design and construction was required to meet the project schedule. The estimate of cost for options ranged from $12 million to $25 million. The selected $12 million option required about 1.2 million cubic yards of excavation, including sandstone used for the buttress, and the final cost was within the budget.

Introduction

The subject section of State Route 7 (WAS-7-18.10) is just south of Marietta, Ohio, on a hillside overlooking the Ohio River (Figure 1). In this area, SR-7 was widened from 2 lanes to 4 lanes in the 1960s by excavating into the hillside. The section of roadway in this study is 2,500-feet long, beginning at approximate Station 1033+50 and ending at approximate Station 1058+50.

While the project section has been subject to rockfalls and landslides since its construction (see Photos 1 and 2), the problems accelerated after January 2004 due to several periods of heavy rainfall caused by remnants of 2 hurricanes that passed through the area. Reconstruction and repair work took place in 2004, but rockfalls continued. This includes damage to a concrete barrier constructed at the edge of pavement in November 2004 (see Photos 3 through 5). This led to closure of the southbound lanes and subsequently to this investigation.

Site Geology

Bedrock units are from the Washington Formation, generally between the Creston Reds (Mudstone), located at the base of the slope, and a series of interbedded mudstone and sandstone units in the upper section of the slope. The massive Upper Marietta Sandstone is located about midway through these intervals. Structurally, the rock formations dip gently to the south-southeast, out of the slope; however, local variations in the project area are possible (Figure 1). A geologic profile based on borings and field reconnaissance information is shown in Figure 2. The figure shows the grade of SR-7 through the area and the rock units, which are described in the following paragraphs.
For this investigation, the principal rock units were identified by their lithology. The designations, summarized in Table 1, are intended to differentiate key units. Although the lateral and vertical continuity of some units above the Upper Marietta Sandstone appear to be relatively consistent, some areas are obscured, and this could not be fully confirmed. The maximum elevation at the crest of the slope in this area is approximately 890 feet.

Rock units having a primary influence on the stability of this slope are the mudstones, which weather to develop landslide-prone residual soil and colluvium, or undercut more resistant overlying formations. While the SH1 Unit has not been classified as a mudstone in the table, the unit is a mudstone interbedded with siltstone and shale. While this unit appeared to be more stable than the other mudstones (due to the interbedded subunits), it is believed the presence of the mudstone could dominate its long-term performance.

**Table 1**  
**Principal Rock Units Observed in the Project Limits**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Description</th>
<th>Approx Thickness (feet)</th>
<th>Approx Elevations</th>
</tr>
</thead>
<tbody>
<tr>
<td>?</td>
<td>Undefined</td>
<td>15</td>
<td>875 890</td>
</tr>
<tr>
<td>SS3</td>
<td>Interbedded Sandstone/Siltstone</td>
<td>25</td>
<td>850 875</td>
</tr>
<tr>
<td>M S3</td>
<td>Red Mudstone</td>
<td>10</td>
<td>840 850</td>
</tr>
<tr>
<td>SS2</td>
<td>Shaley Sandstone</td>
<td>15</td>
<td>825 840</td>
</tr>
<tr>
<td>M S2</td>
<td>Red Mudstone</td>
<td>15</td>
<td>810 825</td>
</tr>
<tr>
<td>SH3</td>
<td>Shale</td>
<td>10</td>
<td>800 810</td>
</tr>
<tr>
<td>SS1</td>
<td>Sandstone (Upper Marietta)</td>
<td>50</td>
<td>750 800</td>
</tr>
<tr>
<td>SH2</td>
<td>Shale/Siltstone</td>
<td>15</td>
<td>735 750</td>
</tr>
<tr>
<td>M S1</td>
<td>Red Mudstone</td>
<td>35</td>
<td>700 735</td>
</tr>
<tr>
<td>SH1</td>
<td>Interbedded Mudstone/Shale/Siltstone</td>
<td>45^1</td>
<td>655 700</td>
</tr>
</tbody>
</table>

^1 Base elevation is the lowest elevation of borings obtained during investigation.

**The Investigation Phase**

**Office Reviews and Field Reconnaissance**

A review was made of available geologic information, plans for construction of the 4-lane facility (dated 1958) and borings completed for remediation activities in 2004.

HDR provided a geologist-led team to make field observations of slide activities, locate and describe exposed rock formations and measure discontinuities (orientation and condition) of the massive Marietta Sandstone (SS1 Unit) and the overlying sandstone units. Control for this field reconnaissance was provided by the ODOT survey crew, which established reference station markers on the barriers and guiderail at road level and on the bench and slopes above the Marietta Sandstone. Elevations were provided at the stakes.
HDR obtained elevations of the top and base of the SS1 Unit at 100-foot intervals along the project length using a tape hung over the SS1 exposure with an observer at the base of the slope taking measurements of the SS1 base, as well as the base of the underlying SH2 Unit from that level. This information was extrapolated to estimate contact points between rock units over the entire project length. Due to its prominence, measurement of the orientation and condition of discontinuities focused in the SS1 Unit based on criteria established by Bieniawski, 1989.

**Test Borings**

This phase of the investigation included borings at selected locations to:

- better define the rock units above and below the SS1 Unit;
- obtain samples for strength and index testing of the mudstone units and, to a lesser extent, of the other units (excluding the SS1 Unit); and
- provide locations where in-situ shear strength testing could be conducted.

One goal of the program was to select locations that represented stages of deterioration in the mudstone units, since it was recognized that the strength of these units is time-dependent and that the strength loss is significant.

The scope of the program was dictated in part by time constraints on the investigation. The completed program included 7 borings and a total of 417.5 lineal feet of drilling. An attempt was made to push an undisturbed sample in weathered mudstone without success. Dennison samples were included in the program to obtain samples of rock for laboratory strength testing if core recovery was poor. However, good core samples for testing were recovered from coring operations.

Borings were drilled using Standard Penetration Testing (SPT) in soil and very weathered rock, and rock coring where SPT refusal was obtained. NX–sized core was obtained in rock using a double tube barrel with a split inner barrel. Runs were typically 5 feet long, although shorter runs were made when conditions suggested better recovery might be obtained. All cores in mudstone were wrapped in plastic in the core boxes to preserve the in-situ moisture content as much as possible. Representative core samples were wrapped in additional plastic, further sealed with tape and carefully packed for transportation to the office and/or laboratory.

Ground water levels were measured at the end of drilling (0 hours) and where possible, after at least 24 hours or longer (e.g. 6 days in 1050.0-1).
Field Shear Testing

Field shear testing was conducted by Mr. Roger Failmezger, P.E., of In-Situ Soil Testing, LC (IST) and Dr. David White, P.E., from Iowa State University (ISU), both of whom have experience performing in-situ borehole shear testing. There are no existing ASTM or AASHTO methods for this test; however, testing is based on procedures that have evolved dating since the 1960s. Borehole shear testing in stiff clay soils, as well as in softer soil and rock, are discussed by Lutenegger et al, 1978. Different devices and procedures are used for testing of soil and rock and selection of the Borehole Shear Test (Soil) or the Rock Shear Test (Rock) device was based on observation of boring results or field conditions. Some tests were performed in borings and others were performed in colluvium or near-surface weathered rock by using a small diameter hand auger to prepare a hole for the shear testing device. In either case, it was important that the prepared core or auger hole was of a diameter only slightly larger than the test device since the amount of lateral movement required to set the shear plates in the soil or rock is very small.

A total of 25 field shear tests were performed during the field investigation, including 10 using the Borehole Shear Test device and 15 using the Rock Shear Test device. A general breakdown of testing by rock unit and condition is shown in Table 2.

<table>
<thead>
<tr>
<th>Location/ Boring</th>
<th>Preparation Method</th>
<th>Rock Unit</th>
<th>Test Device</th>
<th>No. of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>1042.0-1</td>
<td>Cored hole</td>
<td>SH1 &amp; MS1</td>
<td>Rock Shear</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Borehole Shear</td>
<td></td>
</tr>
<tr>
<td>1042.3-1</td>
<td>Augered hole</td>
<td>MS1 Colluvium</td>
<td>Borehole Shear</td>
<td>1</td>
</tr>
<tr>
<td>1042.5-1</td>
<td>Augered hole</td>
<td>MS1 Colluvium</td>
<td>Borehole Shear</td>
<td>1</td>
</tr>
<tr>
<td>1042.75-1</td>
<td>Augered hole</td>
<td>MS1</td>
<td>Borehole Shear</td>
<td>1</td>
</tr>
<tr>
<td>1044.0-1</td>
<td>Cored hole</td>
<td>MS1</td>
<td>Borehole Shear</td>
<td>3</td>
</tr>
<tr>
<td>1046.0-1</td>
<td>Cored hole</td>
<td>SH2 &amp; MS1</td>
<td>Rock Shear</td>
<td>3</td>
</tr>
<tr>
<td>1050.0-1</td>
<td>Cored hole</td>
<td>MS2</td>
<td>Rock Shear</td>
<td>2</td>
</tr>
<tr>
<td>1054.0-1</td>
<td>Cored hole</td>
<td>SH1, SH2 &amp; MS1</td>
<td>Rock Shear</td>
<td>4</td>
</tr>
<tr>
<td>1054.0-2</td>
<td>Cored hole</td>
<td>Weathered MS2 SH3</td>
<td>Borehole Shear</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rock Shear</td>
<td>2</td>
</tr>
<tr>
<td>1056.5-1</td>
<td>Augered hole</td>
<td>Weathered MS2</td>
<td>Borehole Shear</td>
<td>1</td>
</tr>
</tbody>
</table>

Testing equipment and testing operations at selected locations are shown in Photos 6 through 11.

Laboratory Testing

Laboratory tests were performed on samples obtained from the investigation with the primary focus on the mudstone units. AASHTO methods were used to perform the following tests.
Index Testing (Atterberg Limits, Gradation and Classification) are summarized in Table 3. All tests were performed on intact or weathered mudstone samples from the MS1 and MS3 Units. Note that the samples contained more than 90% fines (with clay defined as the minus 2 micron fraction of the sample) and that the majority of fines are silt-sized. Also, there is little difference between the results on SPT samples and cored samples believed to represent more intact rock.

### Table 3
**Summary of Laboratory Classification Tests**

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth</th>
<th>Sample</th>
<th>AASHTO Class</th>
<th>ODOT Class</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay (&lt;2µ)</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1042.0-1</td>
<td>1.5-3.0</td>
<td>Core</td>
<td>A-4(4)</td>
<td>A-4b</td>
<td>0</td>
<td>1</td>
<td>58</td>
<td>41</td>
<td>23</td>
<td>17</td>
<td>6</td>
</tr>
<tr>
<td>1044.0-1</td>
<td>1.5-3.0</td>
<td>SPT</td>
<td>A-4(8)</td>
<td>A-4b</td>
<td>2</td>
<td>7</td>
<td>58</td>
<td>33</td>
<td>28</td>
<td>18</td>
<td>10</td>
</tr>
<tr>
<td>1044.0-1</td>
<td>18.3-18.7</td>
<td>Core</td>
<td>A-4(8)</td>
<td>A-4b</td>
<td>0</td>
<td>1</td>
<td>57</td>
<td>42</td>
<td>27</td>
<td>18</td>
<td>9</td>
</tr>
<tr>
<td>1054.0-2</td>
<td>2.5-4.0</td>
<td>SPT</td>
<td>A-6(15)</td>
<td>A-6a</td>
<td>0</td>
<td>1</td>
<td>55</td>
<td>44</td>
<td>35</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td>1054.0-2</td>
<td>7.5-9.0</td>
<td>SPT</td>
<td>A-6(13)</td>
<td>A-6a</td>
<td>0</td>
<td>1</td>
<td>61</td>
<td>38</td>
<td>35</td>
<td>23</td>
<td>12</td>
</tr>
<tr>
<td>1054.0-2</td>
<td>12.5-14.0</td>
<td>SPT</td>
<td>A-6(12)</td>
<td>A-6a</td>
<td>0</td>
<td>0</td>
<td>61</td>
<td>39</td>
<td>32</td>
<td>20</td>
<td>12</td>
</tr>
<tr>
<td>1050.0-1</td>
<td>34.9-35.4</td>
<td>Core</td>
<td>A-6(10)</td>
<td>A-6a</td>
<td>6</td>
<td>3</td>
<td>56</td>
<td>35</td>
<td>32</td>
<td>20</td>
<td>12</td>
</tr>
</tbody>
</table>

In-situ Moisture Contents – Results of tests on 23 samples ranged between 3.0 and 13.6 percent with 18 more intact, cored samples having lower moisture contents between 3 and 9 percent, while 4 SPT samples ranged between 5.6 and 13.6 percent.

Unconfined Compressive Strength Tests – Sixteen tests were performed, and results are summarized in Table 4.

### Table 4
**Summary of Unconfined Compression Test Results**

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>No. of Tests</th>
<th>Results Range (in psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS1</td>
<td>7</td>
<td>60-960</td>
</tr>
<tr>
<td>MS3</td>
<td>1</td>
<td>90</td>
</tr>
<tr>
<td>SH1</td>
<td>4</td>
<td>70-690</td>
</tr>
<tr>
<td>SH2</td>
<td>2</td>
<td>820-2260</td>
</tr>
<tr>
<td>SH3</td>
<td>2</td>
<td>1070-1520</td>
</tr>
</tbody>
</table>

Slake Durability Index (SDI) Tests – This includes one test in the MS2 Unit and 2 from the MS1 Unit. It was planned that 5 cycles would be performed; however, 2 cycles were sufficient to result in complete or near-complete deterioration. Two-cycle SDI values of 0.0% and 0.7% were obtained in the MS1 Unit with 6.1% in MS2. Based on these results and office wet-dry tests described later in this paper, testing was stopped at 2 cycles.

Triaxial Shear Testing – Two CU tests were performed on core samples. A 3-point test was conducted on samples each soaked for one day in the triaxial cell with back-pressure, prior to testing. These were intended to represent the strength of a relatively intact sample. The
second 3-point test was performed on samples soaked for 5 days at a small back-pressure prior to testing. These were intended to represent a fully softened condition. Results from the first test indicated that one of the 3 samples was not comparable to the other 2. A plot using the 2 similar samples showed an effective friction angle, Ø', of 14.5º with an effective cohesion of 13.5 psi (1944 psf). Results from the second test were so erratic that a useful strength envelope could not be developed. Additional triaxial shear testing was anticipated, but time constraints did not allow for completion of those tests.

- Ring Shear Test – One test was performed by Dr. White at ISU on a remolded bulk sample of colluvial soil derived from the MS1 Unit and obtained from the toe of the slope at the base of the hillside. The measured effective friction angle, Ø', was 9.3º with an effective cohesion of 1.5 psi (216 psf). It is noted that the corresponding Borehole Shear Test performed in the colluvium at that same location gave a Ø' of 11.8º with an effective cohesion of 0.9 psi (130 psf).

- X-Ray Diffraction (XRD), Scanning Electron Microscope (SEM) and Spectroscopy (EDS) Test – One test was performed by Dr. White at ISU on the sample of material obtained for the Ring Shear test. Results indicated that Quartz, Kaolinite, Illite, Calcite, Goethite and Hematite were present in the sample of colluvium. Data on mineralogy of the local formations available from the literature (Martín, 1998) was used in conjunction with the testing by Dr. White. The mineralogy of clay-sized fractions in the Dunkard mudstones reported by Martín (1998) indicated that Kaolinite and Illite compose nearly 53% of samples tested.

**Office Testing**

To further observe the deterioration characteristics of the mudstone, wet-dry tests were performed in the office on selected mudstone core samples. This permitted comparison with laboratory SDI tests conducted under a more aggressive procedure. While it is understood that these wet-dry tests are not certified in any way, it is believed that the results are comparable with other laboratory tests such as SDI and with field observations of weathering characteristics.

Six core samples of material from the MS1 and SH1 Units were subdivided by cutting them approximately in half. Unit weights were estimated and one portion of each sample was subjected to 2 wet-dry cycles while the other portions of the samples were subjected to 2 air-dry cycles without wetting.

Five of the six wet-dry samples disintegrated completely within 2 cycles and the 6th was partially disintegrated (see Photos 12 and 13). It was concluded that mudstone layers in the SH1 and MS Units are equally susceptible to deterioration, and that agitation, conducted as part of the SDI testing, is not required to cause deterioration of these units. Air-dried samples deteriorated to a much lesser extent.
Analysis and Considerations for Remediation Options

Shear Strength Tests

Selection of design shear strengths for the mudstone units was based on correlation of field and laboratory test results. Figure 3 is a plot of field shear test results for what is believed to be relatively intact rock from the MS1 Unit, or more likely material in various stages of weathering. The borings in the area of these tests were cored during drilling operations. Based on test results, the Adopted Design [Effective Shear] Strength used to represent partially weathered mudstone were $\phi = 15^\circ$ and $c' = 100$ psi. Results in the MS2 Unit for what was believed to be intact (partially weathered) or fully weathered rock were consistent with the adopted strength.

Figure 4 includes field shear test results for weathered rock and soils derived from the MS1 Unit where SPT or hand-auger methods were used to advance the borings. Also shown are results from the ring shear test and the triaxial shear test on samples soaked for one day. Based on test results, the Adopted Design [Effective Shear] Strength selected to represent completely weathered mudstone (residual soil) are $\phi = 12^\circ$ and $c' = 0$ psi. Test results for weathered material from the MS1 and MS2 Units were consistent with the adopted strength. A summary of Adopted Design Shear Strengths used in analyses to evaluate remedial options is provided in Table 5. Also provided are wet unit weights that are based on measured densities from testing. Wet densities for partially weathered mudstone represent a general average for samples with dry densities in the 135 to 150 pcf range, while the value of 120 pcf used for completely weathered mudstone is taken from the ring shear test.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\phi$ (psi)</th>
<th>$c'$ (psi)</th>
<th>$\gamma_{wet}$ (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partially weathered mudstone</td>
<td>15</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>Completely weathered mudstone</td>
<td>12</td>
<td>0</td>
<td>120</td>
</tr>
</tbody>
</table>

Based on the investigation, key formations in the evaluation of the stability in this slope are the MS1, MS2 and MS3 Units. The SH1 and SH2 Units were also believed to be important to slope performance since the SH1 Unit contains interbedded mudstone, and the slickensided, closely-jointed SH2 Unit was located between the MS1 and SS1 Units.

Back-Calculated Strengths

An equilibrium analysis of stability (i.e., FS=1.0) was performed at 2 locations to back-calculate an estimate of strength for the MS1 Unit and colluvium derived from that unit, as follows:

- Near Station 1051, a block of jointed SH2 material caused a failure in the underlying MS1 Unit. Results indicated a cohesion of 4.2 psi (600 psf) would have been necessary with a friction angle of 12° for a stable condition when there is no water table influence. For a
friction angle of $15^\circ$, the cohesion of 4 psi (585 psf) would have been required. With full water pressure, the required shear strength would increase.

- The existing slopes below the SS1 Unit were approximately 1.5H:1V. With a friction angle of $12^\circ$, a cohesion of 1.7 psi (240 psf) was required to maintain equilibrium with no water table influence, but a cohesion of 380 psf (2.6 psi) was required with full water pressure. For a friction angle of $15^\circ$ and no water table, a cohesion of 200 psf (1.4 psi) was required. The required cohesion would increase to 350 psf (2.4 psi) with full water pressure.

The results of the equilibrium analyses are summarized in Table 6. The range of shear strengths from this analysis is believed to be consistent with measured field and laboratory strength tests illustrating the time-dependent reduction of shear strength through a reduction in cohesion with weathering.

<table>
<thead>
<tr>
<th>Location</th>
<th>$\phi^\circ$</th>
<th>$c^\prime$ psf (psi)</th>
<th>Ground Water Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approx Station 1051 (1:1) Failure</td>
<td>12$^\circ$ 15$^\circ$</td>
<td>600 (4.2) 585 (4.0)</td>
<td>None None</td>
</tr>
<tr>
<td>Existing 1.5:1 slope below SS1</td>
<td>12$^\circ$ 15$^\circ$</td>
<td>240 (1.7) 200 (1.4)</td>
<td>None None</td>
</tr>
<tr>
<td>Existing 1.5:1 slope below SS1</td>
<td>12$^\circ$ 15$^\circ$</td>
<td>380 (2.6) 350 (2.4)</td>
<td>Full Full</td>
</tr>
</tbody>
</table>

Published Studies of Mudrock Degradation

An extensive discussion of weathering effects on the strength of mudrocks is presented by Taylor et al, (1987). In that discussion, a figure is presented to illustrate the typical pattern of strength degeneration from an intact to a fissured and then fully weathered condition. That figure is reproduced in Figure 5. Figure 6, also reproduced from Taylor (1987), presents a plot of the strength degeneration for a specific mudrock of Carboniferous age. This pattern is consistent with results from the current investigation, which indicate loss of cohesion with time as the mudstone units are exposed to weather conditions, as well as stress release.

While a much more extensive evaluation of the mudstones at this site could be made, there are several features from the presentation by Taylor et al, (1987), which are believed to be relevant and consistent with findings at this site.

- The principal clay minerals in Carboniferous Mudrocks reported by Taylor et al, (1987), were Kaolinite and Illite/Mica with lesser amounts of Chlorite and Smectite. The significant presence of Kaolinite and Illite found in tests by Dr. White is consistent with Taylor’s findings.
The moisture content increases as weathering proceeds and may provide an indication of the degree of weathering, although the data in the article for fresh and unweathered material does overlap. Using data from the testing program, Figure 9 shows a possible correlation of dry density with moisture content for tested samples where dry density decreased as moisture content increased. To include moisture contents from three SPT samples in the plot, their dry densities were assumed such that they would be consistent with the other data. The samples were from the weathered MS2 Unit in Boring 1054.0-2 with SPT N-values ranging between 22 and 41. While it is recognized that these 3 points are not entirely valid, the assumed densities were included to observe the densities that would be necessary to fit the pattern of true data. The estimated dry densities ranging from about 115 to 125 pcf seem reasonably consistent with material having SPT values in the stated range.

Given the above, plots were developed to investigate the possible relationship between friction angle and cohesion (from field and laboratory tests) with in-situ moisture content and/or dry density. Figures 7a, 7b, 8a and 8b present the resulting plots. While the data is limited, it does appear there is no obvious relationship between friction angle and either moisture content or dry density. On the other hand, it does appear that cohesion is related to moisture content and dry density. Further, it appears that the cohesion decreases toward zero as moisture content increases and dry density decreases. This data is consistent with the belief that strength loss during deterioration is principally due to loss of cohesion, which in turn is consistent with the design assumptions.

Finally, the time-dependent reduction in cohesion and the related depth of weathering were considered for use in analyses. Little or no information is available to substantiate the rate of weathering in these mudrocks. A literature review provided no significant information on this issue, but suggest full reduction of strength (i.e., decrease of cohesion to zero) could take decades (Taylor, 1987). This would be reasonably consistent with observations of slopes in mudrock along Interstate 77 in and around Marietta, which have continued to slide and creep with each passing year, and in particular after the flooding in the past 2 years.

Given the lack of existing data on this issue, it was assumed for long-term performance analysis that the depth of complete weathering was 10 to 15 feet, and the cohesive strength decreased to zero within that depth. Therefore, analyses were performed for a range in the depth of weathering.

**Evaluations (Options for Remediation)**

Four remedial schemes or options were considered, consistent with schemes requested by the Department. The options included:

- Option 1 - A full cut
- Option 2 - A rock buttress
- Option 3 - Soil nailing
- Option 4 - A combination of a rock buttress and soil nailing
Stability analyses for evaluation of slope remediation options were conducted using STABL6H, while design of a soil nailing system was made using the program SnailWin V3.1. The targeted minimum safety factor using adopted strengths was 1.3. It was assumed that the groundwater for that analysis would be at the ground surface (full groundwater). This was believed to be a conservative assumption for slopes fully exposed to weather conditions during wet periods.

Based on the results of the investigation, Option 2 was selected as the most practical and economical remedial scheme. This option required less excavation than Option 1 and was not subject to risks associated with soil nailing in this application.

Option 2 utilized a rock buttress placed against rock units below the SS1 Unit, including the SH1, MS1 and SH2 Units. Excavation from the SS1 Unit was used for the buttress. Application of the buttress option to the mudstones above the SS1 Unit was based on a review of possible grading schemes, including the availability of adequate buttress rock quantities on site, the cost of moving excavation, risks associated with a cut slope above the SS1 Unit and ROW impacts. The criteria for the buttress design in the upper units were the same as for the area below the SS1 Unit. Ultimately, the selected option included a 3H:1V cut slope in the units above the SS1 Unit (Marietta Sandstone) in conjunction with the rock buttress below the SS1 Unit.

A ¼:1 slope was believed to be suitable for excavation in the Marietta Sandstone, if some level of risk associated with planar and wedge failures was acceptable, and given that they are most likely to occur during construction. In addition, the potential that any rock falls will reach to the roadway was believed to be very small, given the 40-foot wide bench that would be located at the base of the SS1 Unit, the 1.7H:1V buttress slope below the bench, the small catchment area and the concrete barrier at the roadway shoulder.

Table 7 summarizes cost estimates for options, including variations and combinations evaluated.

<table>
<thead>
<tr>
<th>Option</th>
<th>Description</th>
<th>Estimated Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Full cut with 3H:1V slopes above and below SS1 Unit</td>
<td>$15.2 M</td>
</tr>
<tr>
<td>1A</td>
<td>Full cut with 4H:1V below SS1 and 3H:1V above SS1 Unit</td>
<td>$19.4 M</td>
</tr>
<tr>
<td>2</td>
<td>Buttress below SS1 and 3H:1V cut above SS1Unit</td>
<td>$11.9 M</td>
</tr>
<tr>
<td>2.1</td>
<td>Buttress below SS1 and above SS1 to top of MS2 Unit</td>
<td>$13.0 M</td>
</tr>
<tr>
<td>2A</td>
<td>Buttress below SS1 and above SS1 to top of MS3 Unit</td>
<td>$13.6 M</td>
</tr>
<tr>
<td>3</td>
<td>Soil Nailing below SS1 and above SS1 to top of MS3 Unit</td>
<td>$25.2 M</td>
</tr>
<tr>
<td>3A</td>
<td>Soil Nailing below SS1 and 3H:1V cut slope above SS1</td>
<td>$17.1 M</td>
</tr>
<tr>
<td>4</td>
<td>Buttress below SS1 and Soil Nailing above SS1</td>
<td>$21.1 M</td>
</tr>
</tbody>
</table>
Construction

Construction went very well and generally as planned. The final cost of the work was $13.5 million, which included additional costs for excavation and repairs of a separate rock slope just north of the site. Approximately 1.4 million cubic yards of excavation were made, including 227,000 cubic yards of rock used in the buttress. Photos 14 and 15 show the finished slopes. The roadway was opened to traffic and the slope has performed well since it was repaired.

Bibliography

Photo 1. SR-7 Slopes in 1980s

Photo 2. SR-7 Slopes in 1980s
Photo 3. Post 2004 Repair

Photo 4. Post 2004 Repair

Photo 5. Post 2004 Repair
Photo 6. Borehole Shear Test

Photo 7. Borehole Shear Device

Photo 8. Borehole Shear Test
Photo 9. Rock Shear Test

Photo 10. Rock Shear Test Device

Photo 11. Rock Shear Test
Photo 12. Before Wet–Dry Cycles

Photo 13. After 2 Wet–Dry Cycles
Photo 14. Completed Slope Remediation Showing Rock Buttress
Photo 15. Completed Slope Remediation

Figure 1. Project Location Map and Structural Geology

Reference:
Geology of Washington County, Ohio
Bulletin 96, 1977
By H.R. Collens and B. E. Smith
Department of Natural Resource
Division of Geologic Survey
Contains on the Meigs Creek Coal
Figure 2. Project Geologic Profile

Figure 3. Field Strength Test Results [MS1 - Intact]
Field Shear Strength Tests (MS1 Wthd)

![Field Shear Strength Test Graph](image)

Figure 4. Field Strength Test Results [MS1 - Weath'd]

![Shear Strength Parameters Diagram](image)

Figure 5. Shear Strength Parameters for Intact, Fissured and Fully Weathered Mudrocks (after Taylor, 1987)
Figure 6. Shear Strength Parameters for Progressively Weathered Mudrocks (after Taylor, 1987)
Figure 7a. $\phi'$ vs In-situ Moisture

Figure 7b. $\phi'$ vs Dry Density

Figure 8a. Cohesion vs In-situ Moisture

Figure 8b. Cohesion vs Dry Density

Figure 9. In-situ Moisture vs Dry Density
Petrographic Evaluation of Coarse Aggregates Used in a Flexible Pavement Study on Noise and Frictional Characteristics
by Karol Kowalski and Terry R. West
Purdue University, West Lafayette, IN
(trwest@purdue.edu)
Abstract

A Ph.D. study was conducted by the first author to determine the relationship between texture and friction relative to the International Friction Index (Kowalski, 2007). Noise and friction properties of flexible pavements involve: surface texture, friction, polish resistance and tire/pavement interaction noise. The study involved aggregate gradations (fine, coarse and s-shaped), aggregate sizes (9.5 mm and 19 mm, nominal maximum aggregate sieve size - NMAS) and mix types, hot mix asphalt (HMA), stone matrix asphalt (SMA) and porous friction course (PFC). Two high-friction aggregates (quartzite and steel slag) and three lower-friction aggregates were included: dolomite, hard limestone and soft limestone. The dolomite is a reef rock with steeply dipping beds of the Wabash Formation (Silurian), the hard limestone is from the Louisville Formation (Silurian) and the soft limestone is from the Salem Limestone (Mississippian). Variable percentages of the high friction aggregate types (0 to 90%) were assembled to include the different, less resistance aggregates. The current paper is a further evaluation of the aggregate petrography. Megascopic description is as follows: Quartzite is a pink, fine grained massive quartz-rich rock with interlocking grains. Steel slag is a black, massive material with visible air voids. Dolomite is a gray, fine grained, massive dolomite-rich rock without obvious sedimentary features. Hard limestone is a white, fine grained rock. Soft limestone is a white to tan, clastic rock of sand-sized grains. Greater petrographic detail and associated engineering laboratory data are provided in the paper. Conclusion: The harder the aggregate, the better the frictional resistance, noise abatement is related to aggregate gradation with larger NMAS yielding more noise.


INTRODUCTION

A Ph.D. dissertation by the first author was completed in the School of Civil Engineering at Purdue University after three years of course work and research. Titled “Influence of Mixture Composition on the Noise and Frictional Characteristics of Flexible Pavements”, the dissertation is a comprehensive study consisting of 326 pages of text, figures and tables (Kowalski, 2007). The second author, Dr. Terry R. West, Professor of Earth & Atmospheric Sciences, Purdue University, served as a member of the research advisory committee for the study. The dissertation involved an extensive study of the noise and frictional characteristics of flexible pavements including both a laboratory and field investigation. In the course of the work, five different aggregate types were used in varying amounts, along with several other variables, to design and construct flexible pavement samples for testing. The purpose of the current paper is to provide a more extensive, petrographic evaluation of aggregates, including lab test data, used in the original study. In addition, emphasis here is limited to the laboratory study on frictional characteristics of the flexible pavements, with only minimal attention given to the noise abatement portion of the research.

Pavement friction is primarily a function of the surface texture, which includes both micro- and macrotexture. Pavement microtexture is defined as “A deviation of a pavement surface from a true planar surface with characteristic dimensions along the surface of less than 0.5 mm” while the pavement macrotexture is defined as “a deviation of 0.5-50 mm” (Henry, 1996; Wambold et al., 1995). Microtexture (a function of the surface texture of the aggregate particles) provides a gritty surface that disrupts the continuity of the water film and produces frictional resistance between the tire and pavement. Macrotexture (determined by the overall properties of the pavement surface) provides surface drainage channels for water expulsion from the contact area between the tire and pavement. This expulsion prevents hydroplaning and improves wet frictional resistance by enhancing the tire/pavement contact (Fulop et al., 2000; Hanson and Prowell, 2004).

While efforts to increase the mechanical durability of pavements are at the core of the Superpave technology, none of the existing mix design methods specifically focuses on addressing their frictional characteristics. This property is typically ensured by using quality coarse aggregate with a history of good frictional performance (West et al., 2001). Generally, igneous and metamorphic rock constituents polish to a lesser extent than do sedimentary rocks. Frictional resistance of carbonate rocks were thoroughly investigated by researchers in Indiana (West and
Cho, 2001). The mineralogy of the aggregates also affects the frictional resistance (West and O’Brien, 2005).

The material properties (which include aggregate types and mixture composition) were studied in an attempt to develop a relationship between them and the frictional characteristics of the pavement. The research was limited to flexible pavements.

Owing to limited availability of high friction aggregates in some areas, there is a need to combine them with locally available materials that may have lower polishing resistance. There is also a need to assess and optimize the combined effects of pavement micro- and macrotexture on the level of pavement friction. In order to achieve this optimization in a timely fashion, it was first necessary to identify an accelerated method to polish test samples and test their frictional properties.

The main purpose of the thesis research was to develop the laboratory device (and testing procedure) to accelerate polishing of hot mix asphalt (HMA) surfaces in order to evaluate changes in their frictional characteristic as a function of the polishing level. A second objective involved evaluation of various blends of aggregates to optimize the combination of micro- and macrotexture to achieve a satisfactory level of friction. Development of the relationship between mixture composition and frictional characteristics was a major consideration. Another goal involved the development of the International Friction Index (IFI)-based, flag value that can be used as a baseline for the laboratory friction measurements.

Based on a literature survey, mixture composition seems to affect both noise and frictional properties of flexible pavements. This suggests that it may be possible to predict and modify both noise and frictional properties of the pavement by changing the aggregate type and HMA composition.

The scope of this study included the investigation of the relationship between mixture composition and the following pavement characteristics: surface texture, friction, polishing resistance and tire/pavement interaction noise. Based on the relationship between texture and friction, the International Friction Index (IFI)-based flag friction value was developed for laboratory testing.

The Ph.D. study included both laboratory and field measurements. The overall research plan included literature study, material (aggregate and binder) characterization, test sites selection, mix design, sample preparation, testing and data analysis. As stated previously, only the laboratory analysis regarding frictional properties is considered in the current paper.
The study involved laboratory testing of various aggregate gradations (fine, s-shaped and coarse) and aggregate sizes (9.5 mm and 19 mm Nominal Maximum Aggregate Size, NMAS) of Superpave mixtures. Aggregates commonly used in HMA in the north central region of the US (natural sand, dolomite and two types of limestones) were combined with different percentages (from 0 to 70%) of two, high-friction aggregates (quartzite and steel slag) to produce the mixes used in the study. In addition, stone matrix asphalt (SMA) and porous friction coarse (PFC) mixes were also tested.

Friction and texture measurements were conducted on 50 laboratory-prepared and polished HMA slabs. In order to obtain frictional resistance curves, measurements were performed after compaction and periodically during the slab polishing cycle. Laboratory texture and friction tests were conducted using, respectively, the Circular Track Meter (CTM) and the Dynamic Friction Tester (DFT) devices.

**TESTING PROCEDURES**

The most widely used device to accelerate polishing of aggregates is the British Polishing Wheel, standardized in ASTM D 3319. However, this method evaluates only the loss of microtexture of the coarse aggregate fraction, neglecting any contributions of the fine aggregate or the pavement macrotexture (McDaniel and Coree, 2003). The changes in microtexture are typically measured by the British (Pendulum) Skid Resistance Testing (BSRT) device used to obtain the BPN or British Pendulum Number. Currently there is no widely accepted method that can measure both the changes in micro- and macrotexture of HMA specimens during polishing.

Frictional characteristics of pavements usually are reduced over time under traffic conditions. Deterioration of tire/pavement friction below a minimum, acceptable (safe) level prevents the pavement from serving its desired function (Roberts, 1996). The need for a minimum friction number has been recognized by several interest groups, including law enforcement agencies. However, due the various legal issues, minimum acceptable friction level requirements have not been published. Instead, most agencies currently use a so called “friction flag value” which is defined as the friction number at or below that a site investigation needs to be conducted (Li et al., 2003).

For example in Indiana, the INDOT pavement inventory friction test program includes all interstate, state, and U.S. routes. Inventory tests are conducted annually on interstate highways and every three years on the other roads (Li and Noureldin, 2005). Highways with a friction number at
or below the flag value are field inspected by the appropriate INDOT district to evaluate pavement conditions and determine if resurfacing is necessary (Li et al., 2003).

An extensive review to identify currently followed friction requirements was recently conducted (Henry, 2000). National and state requirements were not established, but the existence of friction flag values for most states was noted. Although this value varies from state to state, it appears that most are based on findings from an NCHRP study conducted in the 1960s to determine recommended minimum friction values (Kummer and Meyer, 1967). In this study, researchers analyzed many factors, including driver behavior, friction level versus wet accidents, and friction level versus highway maintenance costs. They recommended minimum requirements for BPN and SN (Skid Number) values when the friction trailer test (ASTM E 274) is conducted at 64 km/h using a rib tire. As a final conclusion, the authors provided recommended values related to mean traffic speeds on the investigated highways. This is provided in Table 1.

<table>
<thead>
<tr>
<th>Mean Traffic Speed, km/h</th>
<th>Skid Number, SN</th>
<th>British Pendulum Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>48</td>
<td>31</td>
<td>35</td>
</tr>
<tr>
<td>64</td>
<td>33</td>
<td>40</td>
</tr>
<tr>
<td>81</td>
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<td>45</td>
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<td>97</td>
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<tr>
<td>113</td>
<td>46</td>
<td>--</td>
</tr>
<tr>
<td>129</td>
<td>51</td>
<td>--</td>
</tr>
</tbody>
</table>

* ASTM E-274 friction trailer test conducted at 64 km/h using rib tire.

In Indiana, INDOT utilized Kummer and Meyer’s findings to determine the friction flag value for the trailer using a smooth tire (at 64 km/h). During this analysis, the recommended skid number for roads with a mean traffic speed of 81 km/h was selected (SN value=37). Then the recommended value was converted to an average result for a smooth tire (instead of the rib). INDOT tests concluded that the average friction difference for slick concrete, asphalt surfaces and network pavements equals 18, when results from smooth and rib tires are compared. Subtracting 18 from SN=37 yields a flag value of 19. Based on this, INDOT set their recommended SN value, smooth tire at 20.
It is recognized that the methods and systems vary significantly for measuring texture and friction that are applied in different parts of the world. Frictional resistance can be reported in terms of friction coefficient (µ), British Pendulum Number (BPN), International Friction Index (IFI), skid number (SN) and friction number (FN) (Henry, 2000). The International Friction Index was developed in an attempt to include texture and friction values obtained from different test methods. The IFI (currently standardized in the ASTM E 1960 specifications) consists of two parameters: the calibrated wet friction at 60 km/h (F60) and the speed constant of wet pavement friction (Sp). Both F60 and Sp are estimated from standard curves available in the literature. F60 represents the average wet coefficient of friction experienced by a passenger car during locked-wheel slide at speed of 60 km/h. Sp is a measure of how strongly the pavement wet friction is dependant on the sliding speed of a passenger car (high Sp value indicates a low sensitivity to slip speed) (Cenek et al., 1997). Besides the unifying aspect of IFI, the model can be applied to predict friction values at speeds other than that at which the friction was measured. Using those two IFI parameters (F60 and Sp), the wet friction at any slip speed can be estimated.

F60 and Sp are based on equations involving DF20 and MPD, where

\[ DF_{20} = \text{wet friction number measured at the speed of 20 km/h}, \]
\[ MPD = \text{mean profile depth (mm)}. \]

These same equations are provided in the ASTM E 1960 specification. It should be noted that in the typical range of friction and texture values for asphalt highways, changes in the wet friction (DF20) effect the calibrated wet friction (F60) much greater than do changes in macrotexture (MPD). Moreover, in the low range of DF20 or MPD values, changes in the second parameter (MPD or DF20, respectively) are less significant than for the high range of DF20 or MPD values.

The IFI, International Friction Index, is defined using the F60 and Sp parameters. For an F60 of 0.22 to 0.30, Sp ranges from 76-95 km/h. Also, F60 is 0.15 to 0.20 for Sp of 60-75 km/h. In keeping with this, the selected F60, friction flag value is 0.17 to 0.20.

**COARSE AGGREGATE SELECTION**

Currently, a common practice used in Indiana for improving polishing resistance of pavements is to substitute a portion of the carbonate aggregate with a “high friction” aggregate. During this study, mixes with different types of aggregates combined in various proportions were examined. These mixes were evaluated during the laboratory portion of the study, as described below.
Four types of coarse aggregate and two types of fine aggregate were used to prepare the laboratory mixes. Coarse aggregates were selected based on their frictional characteristics and an attempt was made to include the wide wide range of aggregates commonly used in the north-central US. As shown in Table 2, the coarse aggregates included two friction aggregate types (FAT): quartzite and steel slag. Quartzite (Q), was imported from South Dakota, and steel slag, (SS), was supplied by a source located in northern Indiana. Coarse aggregates also included two carbonate aggregate types (CAT): dolomite and limestone. In this study, dolomite from the Wabash formation (a steeply inclined reef deposit, designated D), and two sources of limestone were used. To differentiate the limestone sources, the higher quality (i.e., higher friction and polishing resistant) limestone is the “hard” limestone (HL) and a lower frictional quality limestone became the “soft” limestone (SL). HL is a typical limestone aggregate from the Louisville formation and SL limestone is from the Salem formation. SL contains oolites, has a high LA abrasion loss and polishes substantially when exposed to traffic. This aggregate is not commonly used alone for concrete or asphalt pavements.

| Table 2. Physical properties of aggregates used in the laboratory portion of the study. |
|---------------------------------|----------------|-----------------|----------------|----------------|----------------|
| Type               | Aggregate Symbol |
| FAT Quartzite  | Q  | 2.63 | 0.2  | 22.0 |
| FAT Steel Slag | SS | 3.60 | 1.1  | 14.3 |
| CAT Dolomite  | D  | 2.68 | 1.1  | 24.3 |
| CAT Hard Limestone | HL | 2.63 | 1.5  | 23.7 |
| CAT Soft Limestone | SL | 2.47 | 3.3  | 48.3 |
| Fine Aggregate | Manufactured Sand | MS | 2.74 | 1.2 |

All aggregate blends used in this study contained two types of fine aggregates: natural (siliceous) sand (NS) and manufactured (crushed dolomite) fine sand (MS). These two sands were used in all the mixes, albeit in varying amounts.

In the lab study, conventional dense graded HMA (Superpave) mixes were fabricated and the asphalt slabs produced from these mixes were tested. Factors investigated were: two high
friction resistant aggregate types (FAT): quartzite and steel slag; three carbonate aggregate (CAT) types, hard dolomite, hard limestone and soft limestone; five different aggregate contents of high resistant aggregates (FAC) at 0%, 10%, 20%, 40%, and 70%; three mixture gradations (G), fine, coarse and S-shaped, and two aggregate sizes (NMAS = 9.5 mm and 19 mm).

A full matrix of these combinations would contain 180 cells (2 FAT x 3 CAT x 5 FAC x 3 G x 2 NMAS). A partial factorial design was implemented instead for which 46 laboratory samples were prepared. 36 different mixes were eventually tested in the primary matrix, 2 FAT x 3 CAT x 3 G x 1 FAC x 2 NMAS. All of these mixes had the same content of high friction aggregates (FAC) equal to 20%.

Provided in Table 2 are the physical properties for the aggregates used in the laboratory study. Note the very high value for Los Angeles Abrasion loss for the soft limestone. It also has a lower specific quality and higher absorption.

The quartzite is a pink, fine grained massive, quartz-rich rock with interlocking grains. Steel slag is a black, massive material with visible air voids. Hard limestone is a white, fine grained rock whereas the soft limestone is a white to tan clastic rock of sand size grains containing some oolites.

Aggregate composition is an important factor in the performance of asphalt pavements in Indiana. For high traffic roads, harder aggregates are required to provide long term frictional resistance for HMA surface mixes. Presented in Table 3 is the relationship between traffic volume in ESALs (Equivalent Single Axial Load) and the required coarse aggregate for the surface course (INDOT, 1999). Note that for <1,000,000 ESALs all six aggregate types are allowed; air-cooled blast furnace slag, steel furnace slag, sandstone, crushed dolomite, crushed stone (essentially limestone) and gravel. For 1,000,000 to 3,000,000 ESALs limestone and gravel are not acceptable and for greater than 3,000,000 ESALs limestone and gravel are not acceptable but only 50% dolomite can be used with slag or sandstone. For the dolomite, INDOT specifications require that such rock contain a minimum of 10.3% elemental magnesium which translates into the presence of 78.1% dolomite mineral or more. The dolomite used in the laboratory study reported here had a dolomite mineral content well above 78.1%.
Table 3. Coarse Aggregate Types for HMA Surface Mixtures.

<table>
<thead>
<tr>
<th>Coarse Aggregate Type</th>
<th>Traffic ESAL (&lt;1,000,000)</th>
<th>Traffic ESAL (&lt;3,000,000)</th>
<th>Traffic ESAL (&gt;3,000,000)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air-Cooled Blast Furnace Slag</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Steel Furnace Slag</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Crushed Dolomite</td>
<td>Yes</td>
<td>Yes</td>
<td>Note 1</td>
</tr>
<tr>
<td>Crushed Stone</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Gravel</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

Note: 1. Dolomite may only be used when blended equally with slag or sandstone.

One of the objectives involving the use of different aggregate types (2 FATs and 3 CATs) was to extend the range of aggregates currently used for surface courses on high traffic roads. Obviously the two limestone samples if used alone, or the dolomite in greater than 50% present could not be used on roads with >3,000,000 ESALs according to the specifications. The quartzite too was a substitute for steel slag.

Table 4 is the classification of Coarse Aggregate Requirements for INDOT specifications (INDOT, 1999). The bituminous surface course would fall under Class A requirements. Referring to Table 2, note that the Los Angeles Abrasion loss for SL exceeds the maximum allowable Los Angeles Abrasion loss of 40% shown for Class A aggregates.

CONCLUSIONS

In the current study an investigation was conducted on the influence of the aggregate mixture composition on the frictional properties of flexible (asphalt) pavements. Results show that it is possible to predict and modify frictional properties of the pavement by changing aggregate type and HMA composition. A new laboratory testing methodology was developed to determine two crucial properties for characterizing pavement friction: polishing rate and terminal friction value. From these the IFI or International Friction Index can be determined.

This study also found that increasing the friction aggregate content (quartzite or steel slag) substantially improved polishing resistance of HMA mixes. The overall frictional resistance of the 9.5 mm NMAS mixtures was lower than for mixes with 19 mm NMAS. In addition, mixes with steel slag generally exhibited slightly higher polishing resistance than mixes with quartzite. In general, aggregates with a lower Los Angeles Abrasion loss provided a higher polishing resistance.
The influence of the carbonate aggregate type (dolomite, limestone or soft limestone) on the frictional properties of mixes was also studied. In general, the mixes with soft limestone exhibited lower friction values than those with dolomite and hard limestone. Note the Los Angeles Abrasion loss for SL (soft limestone) was greater than the maximum allowable loss of 40% for Class A aggregate.

During this study, the baseline values (for use in laboratory testing) were determined for the macrotexture (expressed by mean profile depth [MPD]), dynamic friction ($DF_{20}$) and calibrated wet friction ($F60$) values for typical asphalt pavements. Based on the literature review and field measurements involving the towed friction trailer, (ASTM E 274) equipped with both rib and smooth tires, and Circular Track Meter and Dynamic Friction Tester, the approximate IFI (International Friction Index) flag value ($F60$) was determined. This value (based on CTM and DFT devices) was found to equal 0.17-0.20.
### Table 4. INDOT Coarse Aggregate Specifications.

<table>
<thead>
<tr>
<th>Characteristic Class</th>
<th>AP</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quality Requirements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freezer-and-Thaw Beam Expansion, % Max. (Note 9)</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Los Angeles Abrasion, % Max. (Note 1)</td>
<td>40.0</td>
<td>40.0</td>
<td>40.0</td>
<td>40.0</td>
<td>40.0</td>
<td>40.0</td>
<td>40.0</td>
</tr>
<tr>
<td>Sodium Sulfate Soundness, % Max. (Note 2)</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Brine Freeze-and-Thaw Soundness, % Max. (Note 8)</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
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<tr>
<td>Absorption, % Max. (Note 3)</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
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<tr>
<td>Additional Requirements</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Deleterious, % Max.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay Lumps and Frangible Particles</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Coke</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Iron</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass Per Cubic Meter for Slag, kg, Min.</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
</tr>
<tr>
<td>Weight Per Cubic Foot for Slag, (lbs), Min.</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
</tr>
<tr>
<td>Crushed Particles, % Min. (Note 6)</td>
<td>70.0</td>
<td>70.0</td>
<td>70.0</td>
<td>70.0</td>
<td>70.0</td>
<td>70.0</td>
<td>70.0</td>
</tr>
<tr>
<td>Asphalt Seal Coats</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted Aggregates</td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td></td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Los Angeles abrasion requirements shall not apply to blast furnace slag.
2. Aggregates may, at the option of the Engineer, be subjected to 50 cycles of freezing and thawing in accordance with AASHTO T 103, Procedure A, and may be accepted, provided they do not have a loss greater than specified for Sodium Sulfate Soundness.
3. Absorption requirements apply only to aggregates used in portland cement concrete and HMA mixtures except they shall not apply to blast furnace slag. When crushed stone coarse aggregates from Category I sources consist of production from lodes whose absorptions differ by more than two percentage points, the absorption test will be performed every three months on each size of material proposed for use in portland cement concrete or HMA mixtures. Materials having absorption values between 5.0 and 6.0 that pass AP testing may be used in portland cement concrete. If variations in absorption produce satisfactory production of portland cement concrete or HMA mixtures, independent stockpiles of materials will be sampled, tested, and approved prior to use.
4. Non-durable particles include soft particles as determined by TDM 206 and other particles which are structurally weak, such as soft sandstone, shale, limonite concretions, coal, weathered schist, cemented gravel, cinders, shells, wood, or other objectionable material. Determination of non-durable particles shall be made from the total mass (weight) of material retained on the 9.5 mm (3/8 in) sieve. Scratch Hardness Test shall not apply to crushed stone coarse aggregate.
5. The bulk specific gravity of chalk shall be based on the saturated surface dry condition. The amount of chalk less than 2.45 bulk specific gravity, shall be determined on the total mass (weight) of material retained on the 9.5 mm (3/8 in) sieve for sizes 1 through 8, 55, and 91 and on the total mass (weight) of material retained on the 4.75 mm (No. 4) sieve for sizes 9 and 11.
6. Crushed particle requirements will apply to gravel coarse aggregates used in HMA mixtures, compacted aggregates, and asphalt seal coats except seal coats used on shoulders. Crushed particle requirements for HMA mixtures are set out in 904.02(c). Determination of crushed particles shall be made in accordance with ASTM D 4221.
7. Air-cooled blast furnace slag and steel slag coarse aggregate shall be free of objectionable amounts of coke and iron.
8. Brine freeze-and-thaw soundness requirements are subject to the conditions stated in note 2.
9. Freeze-and-thaw beam expansion shall be tested and retained in accordance with TDM 210.

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ACKNOWLEDGEMENTS

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STATUS OF NCHRP ROCK SCOUR PROJECT

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ABSTRACT

The essence of National Cooperative Highway Research Program (NCHRP) Project 24-29 is geotechnical site characterization in scour-relevant terms for use by hydraulic engineers. The project goal is to develop guidelines for evaluating scour at bridge foundations on rock that can be integrated with the procedures of FHWA HEC-18. Rock scour in natural open channels appears to be related to five processes: 1) dissolution of soluble rocks, 2) abrasion of degradable rocks, 3) quarrying and plucking of jointed rocks, 4) cavitation, and 5) physical and chemical weathering that prepares rock masses and surfaces for subsequent scour. The definition of ‘rock’ for scour purposes is just as problematic as the definition of rock for other engineering applications. The physical properties of rock material can range from strong soil to much better than the best concrete. Two benchmark materials are being considered for rock in the context of scour: concrete and mortar. Rock exposed in channels that has characteristics of moderately good concrete probably is sufficiently resistant to hydraulic forces that it might be unscourable during the life of a conventional bridge. Rock with characteristics less that of mortar might be highly susceptible to scour when exposed to the normal range of stream flow during the life of a bridge. Quantifying the rate of rock scour is a challenge because it probably is governed by a threshold loading condition (velocity, hydraulic shear stress, or stream power) below which no scour occurs, but above which scour losses accumulate. NCHRP Project 24-29 is beginning field, laboratory, and modeling studies to refine the approach to quantifying rock scour at bridge sites.

INTRODUCTION

A number of bridges throughout the United States may be founded on erodible rock. Rock erosion processes include gradual dissolution by chemical weathering; disintegration and wearing away by impact and abrasion of bedload and suspended load particles; jacking and plucking of blocks of hard, jointed rock; and cavitation. Soft rock formations may scour rapidly during a single flood event, whereas hard rock formations may show no observable evidence of erosion after decades of floods. Geotechnical properties of most rock materials are not sufficiently well understood for the rock formations to be considered “scour-resistant”, let alone to define the time-rate of scour in susceptible formations. State departments of transportation (DOTs) are required by the Federal Highway Administration (FHWA) to evaluate scour at bridge sites and protect bridge structures from failure. Hydraulic engineers thus may be forced to consider all rock formations as if they were cohesionless sediments for the purpose of estimating scour depths. In many cases, this approach may be overly conservative, with large predicted scour depths that result in excessive foundation costs for new bridges and/or expensive retrofitting of existing bridges.

Scour at bridge foundations traditionally is evaluated by hydraulic engineers with input from geologists and geotechnical engineers. NCHRP Project 24-29 focuses on recognition of rock and rock-like materials that may be susceptible to scour processes and characterization of bridge foundation conditions in terms that accurately reflect the scour susceptibility and can be used by hydraulic engineers to calculate design scour depths. In essence, the research calls for
geotechnical site characterization expressed in scour-relevant terms for use by hydraulic
engineers. The objectives of NCHRP Project 24-29 are to develop (a) a methodology for
estimating the time rate of scour and the design scour depth of a bridge foundation on rock and
(b) design and construction guidelines for application of the methodology. The project was
initiated in the fall of 2006 and is scheduled to be completed in the fall of 2009 for a total
research budget of $750,000.

The challenge of NCHRP Project 24-29 is to synthesize relevant components of the complex
gеology of the United States so that a methodology can be developed to improve the
identification and characterization of erodible rock formations, time-rates of scour, and
maximum scour depths using conventional field and laboratory techniques in a way that will
produce consistent and verifiable results and can be implemented by state DOT personnel. The
work plan seeks to address three significant problems:

(1) discriminating rock formations that erode by general deterioration (weathering and
abrasion) from those that erode block-by-block along pre-existing discontinuities or
discontinuities that are created by crack propagation caused by turbulence intensity
fluctuations,

(2) estimating time-rates of erosion, and

(3) developing a methodology and guidelines for applying the methodology that rely on
conventional field, laboratory, and analytical procedures familiar to state DOT personnel
so that the results of this research will be practical enough to actually be implemented.

NCHRP Project 24-29 is subdivided into two phases with four tasks in Phase I and five tasks in
Phase II. Phase I was completed in the fall of 2007 with an interim report followed by an interim
meeting.

Phase I
Task 1 – Review the Technical Literature
Task 2 – Conduct Survey of State and Federal Agencies
Task 3 – Analyze Information and Propose Preliminary Methodology
Task 4 – Interim Report and Updated Phase II Work Plan

Phase II
Task 5 – Investigate Bridge Sites
Task 6 – Conduct Laboratory, Field, and/or Modeling Studies
Task 7 – Develop Methodology for Determining Time-Rate of Scour and Scour Depth
Task 8 – Develop Design and Construction Guidelines
Task 9 - Submit Final Report
The remaining sections of this status report contain some background information, a description of the five scour processes acting in open channels, preliminary conclusions, and an overview of plans for the Phase II studies of NCHRP Project 24-29.

BACKGROUND INFORMATION

The failure of the Schoharie Creek Bridge on Interstate Highway 90 in New York on April 5, 1987, drew attention to potentially dangerous erosion of materials thought to be resistant and stable. This failure led to a mandate from the Federal Highway Administration that all bridges be evaluated for susceptibility to collapse under similar circumstances. Hydraulic Engineering Circular 18 (HEC-18; Richardson and Davis, 2001) provides guidance and procedures for evaluating scour at bridges. The issue of scour in rock formations is noted in HEC-18, but the guidance is for an engineering geologist familiar with the area to be consulted for evaluation of weathered or other potentially erodible rock formations. Scour competence of rock is discussed in Appendix M of HEC-18. Four recommendations are given for determining if rock foundations are scour resistant, but noting that additional research is needed in this area. The four recommendations are:

- Geologic, geomorphologic, and geotechnical analyses
- Flume tests to determine the resistance of rock to scour
- Erodibility Index procedure

FHWA Memo HNG-31 notes that geologic studies have shown that even the hardest of rock can scour when exposed to moving water for geologic-scale periods of time. Investigation procedures listed in the FHWA Memo are:

1) subsurface investigation,
2) evaluation of geologic formations and discontinuities,
3) calculation of rock quality designation (RQD) from rock core samples,
4) determination of unconfined compressive strength,
5) determination of slake durability index,
6) determination of soundness when exposed to sodium sulfate or magnesium sulfate solutions, and
7) determination of Los Angeles Abrasion Test loss.

Erosion of rock and rock-like materials was studied extensively for stability of unlined spillway channels of dams. The early studies by Moore (1991) and Moore et al. (1994) built on an understanding of the power required for excavation of earth materials described by Kirsten (1982, 1988) which was called the rippability index. Kirsten’s empirical approach correlated the generalized engineering properties of rock with the horsepower rating of equipment that could or could not excavate the material. Kirsten’s (1982, 1988) rippability index classifies earth materials on a continuous range from loose granular or soft cohesive soils through hard, massive rock. A particular type of spillway channel erosion was upstream or upslope advance of knickpoint called headcuts. These procedures, particularly Moore et al. (1994), provided the basis for the field procedures guide for the Headcut Erodibility Index in the National Engineering Handbook.
Turbulent energy dissipation of water flowing over a headcut was expressed in terms of hydraulic power for the headcut erodibility index method.

The headcut erodibility index, $K_h$, represents a measure of the resistance of the earth material to erosion. The index is the product of index numbers for four components of earth materials.

$$K_h = M_s \times K_b \times K_d \times J_s$$  \[1\]

where $M_s$ = material strength number, $K_b$ = block or particle size number, $K_d$ = discontinuity shear strength number, and $J_s$ = relative ground structure number.

Uncorrelated compressive strength is used for $M_s$ without consideration of variability throughout the rock or earth mass. The mean block size of intact rock material is used for $K_b$, which is determined from the spacing of discontinuities within the rock mass or mean grain size for granular material (Barton et al. 1974). The shear strength of the discontinuities in the rock mass is used for $K_d$. It also represents shear strength in granular soils. The number $J_s$ reflects the orientation and shape of individual blocks as determined by the orientations of discontinuities with respect to direction of stream flow.

Annandale (1995) elaborated on the headcut erodibility index and called it simply Erodibility Index which is the name used in HEC-18. Annandale (2000) characterized the glacial till deposits that formed the foundation soils of the Schoharie Creek Bridge on Interstate Highway 90 using the erodibility index procedures and calculated the decrease in instantaneous available hydraulic power of the 1987 flood from its maximum value at the streambed down into the scour hole around the bridge pier (Figure 1). He interpreted the maximum scour depth to be determined by the intersection of the available stream power curve and the earth material resistance curve. Annandale (2000) used two ranges of unconfined compressive strength for the $M_s$ value in calculating the erodibility index, resulting in two curves for the power required to erode the earth materials. The actual scour depth produced by the 1987 flood is indicated on Figure 1 and matches closely with the erodibility index calculation.

**ROCK EROSION PROCESSES**

Four erosion processes in natural rock-bed channels have been identified by Hancock et al. (1998) and Wohl (1999). These processes are dissolution of soluble rocks, abrasion of degradable rocks, hydraulic quarrying of fractured rocks, and cavitation. Hancock et al. (1998) identified an additional important process – the time between scouring flood events during which rock weathering and fracture enlargement occurs – a precondition time that increases the susceptibility of the rock-bed formation to scour by abrasion or hydraulic quarrying.
Dissolution of Soluble Rocks

Some rock types are soluble in water, particularly limestone, dolomite, gypsum, and salt. Limestone and dolomite dissolve relatively slowly and are strong enough to form caves and steep-sided sinkholes, whereas gypsum and salt (halite) are much more soluble than limestone and dolomite, but typically are not sufficiently strong to support cave openings. Limestone and dolomite present different types of issues than gypsum and halite. The weakness of gypsum and salt formations is easily recognized during foundation investigations for bridges, and no bridges should be founded in these rock types. Therefore, scour of these formations should not adversely affect bridge foundations. Gypsum veins and zones within more durable rock types would be expected to dissolve locally which could alter the boundary conditions contributing to other scour processes, such as enhanced opportunity for quarrying because of lower shear strength along the joint surface.

Limestone and dolomite in settings where caves and sinkholes have formed in prehistoric time, possibly even geologic time, present potential hazards to bridges from a foundation support standpoint, and geotechnical investigations in these regions typically are designed with an objective of detecting subsurface voids. Limestone and dolomite formations that have collapsed solution cavities and other features of soluble rock types present a potential scour concern related to collapse debris or breccia that might be present in the vicinity of a bridge foundation. The variable scour resistance of limestone rubble and soil that fills collapsed solution cavities could be a concern, even though the solubility of the limestone formation, per se, might not be a problem during the engineering life of a bridge structure.
**Abrasion of Degradable Rocks**

Rock erosion by abrasion can be accomplished by the shear stress of clear water flowing over the rock surface or by material flaking or breaking off of a rock surface by the impact force of sediment entrained in the flow as bedload or suspended load that intermittently impacts the rock-bed channel. Hydraulic shear stress of clear water flaking small rock fragments and mineral grains is a micro-scale form of quarrying in a strict sense, but it is included in the abrasion because of its scale and the general lack of bedding planes or joint surfaces that control the process. Impacts of bedload fragments or occasional impacts of suspended load fragments produce fractures within minerals, dislodge individual grains, or break off flakes from the rock-bed channel surface. Experimental studies of windblown sediment transport reveal that the mass of material removed by abrasion is proportional to the kinetic energy of the impact. Abrasion rates are sensitive to local flow conditions and the details of the flow hydrograph; high stream velocities and the largest, rare flow events produce the largest instantaneous erosion rates.

Abrasion rates are sensitive to grain-scale microphysics. Particle velocity relative to the channel bed is more important than water flow velocity. For a moving particle to impact the channel bed, the particle must decouple from the flow, because the flow velocity vanishes in the boundary layer at the bed. The entrained sediment must possess enough momentum to decouple from the flow, punching through the near-bed flow boundary layer, and forcibly impacting the bed. The sediment concentration is not sufficient for predicting erosion. Analogous to eolian abrasion by suspended sediment, particle trajectories are influenced by the response of water flowlines to the microtopography of the bed. A particle may be steered by the water around obstacles, or be forced to impact the obstacle obliquely if its inertia is sufficiently low. Increased sediment in the flow may actually decrease the rate of erosion as sediment supply begins to choke off access to the bed, as described by Sklar and Dietrich (1998). A threshold may have to be exceeded for erosion by abrasion to be initiated. A threshold discharge and/or flow duration may have to be exceeded to expose a rock-bed channel that has been buried by sediment during low flow conditions.

The most actively abrading portions of rock-bed channels are where sculpted rock bedforms and potholes occur (Hancock et al., 1998). These bedforms tend to originate where abrupt flow expansions on the downstream edges of bed protrusions promote flow recirculation zones that are associated with flow separation. Hancock et al. (1998) describe flow separation occurring where the boundary layer of a stream of viscous fluid detaches itself from the boundary in response to abrupt expansions or adverse pressure gradients, generating a free-shear layer with a region of separated flow. These flow separation regions, which are characterized by high water flowline curvature, allow entrained sediment to decouple from the flow and impact the bed. This abrasion must be accomplished by suspended grains because the erosional bedforms require that the grains be capable of delivering significant kinetic energy to the back sides of flow obstacles and expansions.

Dickinson and Baillie (1999) used an empirical approach to evaluate rock erosion at 11 bridge sites in the Coast Ranges of Oregon because they concluded that the rock scour phenomenon in natural channels was too complex to accurately reproduce in flume studies or numerical simulations. The rock material at these bridge sites varied from very soft siltstone to hard tuff and basalt. Evidence of erosion was interpreted from cross sections made by Oregon Department
of Transportation; continuous historical stream gauge information was used to calculate hydraulic shear stress and stream power; the nature of the bedrock exposure across the channel indicated that cycles of wetting and complete drying did not occur in the geologic materials at foundation depth. Their study of general scour excluded local scour and contraction scour and related average rate of scour across natural stream channels to geomechanical properties of the rock and the hydraulic power of the stream flow.

Dickinson and Baillie (1999) ran conventional slake durability tests (ASTM D4644) and found that the test was unrepresentative of the local conditions for materials that do not have an opportunity to desiccate completely between wetting cycles. Therefore, they modified the slake durability test to exclude heating and complete drying of the rock samples. They call their modification a ‘continuous abrasion test’ and run the test for about 8 hours, measuring the loss every 30 minutes for 2 hours and every 1 to 2 hours for the duration of the test. The abrasion rate is greatest at the beginning of the test during which time angular fragments become subrounded. After about 2 hours, the weight loss rate is relatively uniform. Dickinson and Baillie (1999) used the slope of the curve of weight-loss versus natural log of time in minutes as an index which they called the ‘abrasion number’. They plotted the average erosion determined from repeated surveyed cross sections to integrated stream power over the period represented by the cross sections and compared the abrasion number of samples from each bridge and found a promising trend.

**Hydraulic Quarrying of Fractured Rocks**

Two simple physical models of rock-bed quarrying exist: one for lifting and one for sliding blocks from their intact positions (Hancock et al., 1998). Block lifting would be generated by pressure differences in the flow, whereas block sliding or rotating would be generated by shear stress on the upper surface of the block.

![Figure 2. Models of block removal by hydraulic quarrying (lifting and sliding). Based on Hancock et al. (1998).](image)

The minimum threshold velocity necessary to lift an ideal rock block that was completely detached along horizontal and vertical joints can be estimated using the Bernoulli equation. The water in the joints will exert a certain pressure on the joint faces, whereas the free-flowing water above the block will have lower pressure dictated by the flow velocity. To initiate lift of the block, the force difference associated with pressure differences must be sufficient to overcome the buoyant weight of the block. The minimum threshold velocity required to initiate lift of the rock block ignoring frictional resistance along the joint surfaces can be estimated as
(1/2)

\( v_t = \left( \frac{(\gamma_r - \gamma_w)gz}{\gamma_w} \right)^{1/2} \)  \[2\]

where \( v_t \) is threshold velocity, \( \gamma_r \) is rock unit weight, \( \gamma_w \) is water unit weight, \( g \) is acceleration of gravity, and \( z \) is block thickness.

Tinkler and Parish (1998) evaluated quarrying by applying flume test results from Reinius (1986) to develop a relation for the threshold velocity required for quarrying slabs of varying thickness. Clast shapes in their study area are normally very slabby, with a \( c \) axis much smaller than a and b axes (Corey-shape factors of 0.1 to 0.2). They observed slabby blocks in close proximity to the downstream sides of drop structures which acted as knickpoints. Tinkler (1993) determined that as river stage rises, the incoming velocity, \( v_i \), accelerates at a drop structure with head \( h \), to an outgoing velocity given by

\[ v_o = \sqrt{v_i^2 + 2gh} \]  \[3\]

which is plotted in Figure 3. Equations for block lifting and block sliding are plotted in Figure 4 with \( \beta \) substituted for \( z \) and a lift coefficient, \( \phi \), for vertical and favorably inclined joints.

![Figure 3](image-url)  

**Figure 3.** Increase in flow velocity caused by acceleration over a drop structure.
Figure 4. Velocity threshold to lift or slide rock slabs up to 2 m thick. Curves for favorably inclined and vertical joints are calculated from Tinkler and Parish (1998) after Reinius (1986); curves for minimum lifting and sliding resistance are calculated from Hancock et al. (1998).

Cavitation
Cavitation occurs when velocity fluctuations in a flow induce pressure fluctuations that cause formation and implosion of vapor bubbles. The shock waves generated by implosions can weaken bedrock and pit the rock surface, a phenomenon relatively common along concrete spillway tunnels of dams. Cavitation may occur at flow separations induced by joints, bedding planes, or other surface irregularities in bedrock (Wohl, 1999). Cavitation-induced erosion of sandstone bedrock produced 10-m deep pools in 1983 in the Glen Canyon Dam spillway tunnels which were discharging as much as 900 m$^3$/s. Cavitation-induced erosion in natural channels has not been documented and probably is not a significant rock erosion process in natural channels.

Baker and Costa (1987) plotted powerful flood flows onto a diagram relating mean velocity to mean depth. They set the Froude number equal to 1.0 to create a line separating subcritical and supercritical flow regimes and solved for mean velocity. They also evaluated the threshold velocity and depth for cavitation using a relation developed by Barnes (1956) and Baker (1974) relating the mean velocity at the threshold of cavitation to the mean flow depth. These threshold relations are plotted in Figure 5. Also plotted on Figure 5 are data points of calculated and estimated flow parameters from tables in Baker and Costa (1987).

Baker and Costa (1987) conclude that their analysis suggests that channel adjustments produced by cavitation tend to inhibit or reduce the forces that would cause the cavitation threshold to be crossed in nature. They state that few, if any, powerful natural flows barely exceed the threshold conditions for cavitation. Their analysis led them to conclude that channel adjustments produced by cavitation tend to be self-moderating so that cavitation does not fully develop.
Hancock et al. (1998) considered cavitation in their study of a steep rock-bed channel of the Indus River in Pakistan. The role of cavitation in erosion of natural river channels was unclear, so Hancock et al. (1998) evaluated threshold mean velocities and channel slopes using simplifying assumptions based on the Bernoulli equation and the Darcy-Weisbach equation for open-channel flow with Manning’s n value. The relations they developed are plotted on Figure 6 with slopes above the heavy dotted line steep enough to generate velocity and depth conditions considered likely to produce cavitation, slopes below the heavy dashed line too gentle to generate velocity and depth conditions to produce cavitation, and cavitation considered to be possible for velocity and depth conditions between the two heavy dashed lines.

Figure 5. Mean velocity and depth for cavitation in natural channels.

Figure 6. Mean flow velocity, depth, and slope conditions where cavitation may be possible. Based on Hancock et al. (1998).
Hancock et al. (1998) conclude that velocity and slope thresholds for cavitation are likely to be exceeded only in locally steep or narrow reaches, but they note that such locally steep reaches typically are locations with rapids that may aerate the flow. Entrained air impedes cavitation by increasing the compressibility of the water. They suspect that erosion of rock by cavitation in natural rock-bed channels is not significant. They report looking for, but not finding, pitting or cracking of rock surfaces which they expect to result from cavitation damage. Therefore, either cavitation damage to the rock surface did not occur or the evidence of damage was erased by post-cavitation abrasion wear during the falling limb of the flood hydrograph. If cavitation conditions are achieved in natural channels, they may not last for a substantial amount of time but the effects may generate micro-cracks that contribute to general degradation of the rock surface which promotes weathering between flood events.

**Preconditioning Time**

Hancock et al. (1998) anticipated that a period of time for joint weathering or ‘preconditioning’ probably was needed for block to become completely detached from a rock-bed channel along its bounding discontinuities. The joint surfaces could be weathered, wedged apart, and/or weakened by bedload impacts during the preconditioning period. They observed gravel fragments wedged tightly into joints in granitic rock and considered that periods of high-velocity turbulent flow caused blocks to vibrate or oscillate which allowed the joints to open sufficiently for the gravel to become wedged in the joints.

Tinkler and Parish (1998) documented changes in a thinly bedded rock-bed channel and concluded that periods of low flow allowed drying and slaking of parts of the channel. These low-flow processes tended to enhance the susceptibility of the rock to scour during subsequent higher flow events. They noted that some places gave a hollow sound when tapped with a rock hammer suggesting that the rock was splitting along bedding planes during the time between stream flows. They also found that ice was a factor, not so much for wedging rock fragments apart, but for freezing onto the upper rock surface and enhancing its ability to become buoyant when water subsequently rose above the level of that part of the rock-bed channel.

It also stands to reason that the susceptibility of rock blocks to hydraulic quarrying is directly related to the mass of the block. Therefore, abrasion acting on the upper surface of a block that is large enough to remain in place will reduce the mass of the block over time possibly to the extent that it becomes susceptible to hydraulic quarrying.

**PRELIMINARY CONCLUSIONS**

Clear evidence of serious, widespread rock scour at bridge sites seems to be limited. Therefore, part of the potential benefit of NCHRP Project 24-29 will be to develop a reliable methodology that will allow rock-founded bridges to be removed from the scour-critical list based on the traditional HEC-18 procedures that evaluates foundation materials as if they were sandy soil. The methodology being considered is a simple screening check-list approach, to allow some of the processes to be dismissed if possible, followed by a more rigorous evaluation of process than cannot be dismissed. A flow diagram for the general approach is shown on Figure 7.
In most settings, soluble rock material will be sufficiently important for bridge bearing capacity that some information will be available at the beginning of the bridge scour evaluation process. Cavitation probably can be dismissed at most, if not all, bridge sites by a simple comparison of the flow velocity, flow depth, and channel slope. It is likely that cavitation processes will not be of concern, but if they are of concern, then it is likely that the rock-bed channel will be so steep and narrow that a bridge will span the channel and bridge foundations will not be exposed to the cavitation processes.

The other two processes, namely abrasion and hydraulic quarrying, require some additional consideration. The NCHRP Project 24-29 approach is seeking to define ‘rock’ in terms that will be sufficiently clear that scour can be quantified without elaborate methods. Concrete is being considered as a benchmark material for rock scour. A weaker and softer benchmark material also
is needed for comparison because some rock materials may be resistant to scour even though they are not as good as concrete. Mortar is being considered as a potential benchmark material.

The National Engineering Handbook (USDA, 1978, Section 8 ‘Engineering Geology’) defines Rock as a compact, semi-hard to hard, semi-indurated to indurated, consolidated mass of natural materials composed of a single mineral or combination of minerals. In contrast, Soils are defined as unconsolidated, unindurated, or slightly indurated, loosely compacted products of disintegration and decomposition of Rock or other Soils.

Rock for scour evaluations can be considered to be earth material that is durable, resistant, and strong, or at least more durable, more resistant, and stronger than soil. Natural earth materials occupy a complete range from very loose disaggregated mineral grains to very dense solid rock. The transition zone from soil to rock is broad with weakly cemented materials that have never been stronger than they are at the present to severely weathered materials that in the past were much stronger than they are at the present. For scour evaluations, rock with concrete-like properties can be considered to be resistant to scour processes. An initial screening-level definition of Rock potentially useful in scour evaluations has three components which may be evaluated qualitatively to quantitatively with simple tests:

1. Coherence/strength
2. Durability
3. Resistance

Coherence is the property that keeps rock fragments intact under constant or static conditions. It may be caused by cementation or crystallization of minerals that allow pieces of rock to remain solid while being handled, dropped, and even struck with a hammer. It is an impression that an observer gets simply by looking at the material in an exposure or a sample from a boring or test pit. Coherence can be described by strength, the property that controls the amount of force needed to break intact rock fragments. Unconfined compressive strength can be determined in the laboratory or estimated by the reaction of the rock to being struck with a hammer. A hammer blow will leave a dent in concrete and in rock materials with comparable strength. A hammer blow will produce a crater in weaker materials, whereas stronger materials will pit or a hammer will rebound.

Durability is the property that keeps rock fragments intact under some adverse environmental loading cycle. The three common environmental loading cycles are wetting and drying, heating and cooling, and freezing and thawing. A simple durability test is the jar slake test in which a hand-size fragment is immersed in water and observed over a period of a few hours. Durable fragments show no appreciable deterioration after several hours in a bucket of water. Nondurable samples slake to a pile of smaller fragments or even a mound of fine grains with soil-like consistency.

Resistance is the property that allows a rock mass to remain substantially unaffected by hydraulic forces that could cause abrasion and quarrying or plucking processes to occur in scour-susceptible materials. Abrasion occurs as flowing water passes over a rock surface; bedload and suspended load sediment can wear away the rock at the surface by forceful impact (saltation) and by rolling, sliding, or grinding. Quarrying or plucking occurs as turbulent water flowing over jointed or fractured rock creates pressure differentials and turbulence-induced vibrations that
jack blocks of intact rock upward, lifting them into the flowing water where they can be moved downstream. Resistance to wear is a function of compactness which is reflected in the unit weight of the rock. Resistance to quarrying is a function of the unit weight of the rock and the spacing of joints, bedding planes, or fractures to define discrete blocks.

Water velocity at the threshold of lifting blocks of rock (quarrying) is shown on Figure 8 for three basic joint conditions: 1) essentially frictionless joints (minimum threshold velocity), 2) rough joints that are favorably inclined for quarrying processes, and 3) rough joints that are vertically oriented.

![Figure 8. Simplified diagram for evaluating threshold velocity for block lifting converted to feet.](attachment:image.png)

Rock that is likely to withstand flowing water without substantial scour will have properties that are generally consistent with those of concrete and possibly mortar; concrete looks like rock, is strong enough to support loads, does not slake, and is heavy enough for large fragments to remain in place when subjected to relatively swiftly flowing water. Concrete has been used in engineering applications as scour protection. Scour-resistant rock will

1) have an estimated unconfined compressive strength comparable to concrete ($\geq 2,500$ psi),
2) remain intact when immersed in water,
3) have a unit weight comparable to concrete ($\geq 150$ lb/ft$^3$) and
4) have joint or bedding planes that define relatively large blocks ($\geq 4$ feet across).

Additional evaluation will be needed to quantify the factors and evaluate other features, particularly if large flow velocities are expected. However, in general, rock conditions that meet the four criteria describe above probably resist the scouring action of flowing water. Abrasion by transported bedload and suspended load can wear away durable rock material over time, which will reduce slab thickness and enhance the susceptibility of quarrying or plucking rock blocks.
Therefore, more detail is needed regarding abrasion in a meaningful definition of Rock for evaluating scour susceptibility.

**PLANS FOR PHASE II STUDIES**

The basic premise of the Phase II investigation is summarized in Table 1 in terms of the four rock scour processes plus a period of physical and chemical weathering which prepares rock material for scour during subsequent stream flows.

Table 1. Scour processes, general observations, and recommended approach for Phase II studies.

<table>
<thead>
<tr>
<th>Potential Rock Scour Process</th>
<th>General Observations</th>
<th>Recommended Phase II Approach</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preparation of Rock for Subsequent Scour</td>
<td>Physical and chemical weathering processes, such as wetting and drying, freezing and thawing, and salt crystallization, weaken rock material over periods of time when stream discharge is low.</td>
<td>Field observation of rock surface weathering and gravel fragments wedged into fractures in blocky rock. Identify reference points for future observations to characterize rate of rock condition deterioration.</td>
</tr>
<tr>
<td>Dissolution of Soluble Rocks</td>
<td>Probably dealt with at initial bridge planning or foundation design stage. General dissolution is a rate problem; ancient solution cavities filled with rubble and soil are local scour problems.</td>
<td>Identify susceptible rock types and filled or unfilled cavities reported in the literature. Field observations of active and paleo-karst-like features. Use solubility tables for susceptible rock types.</td>
</tr>
<tr>
<td>Abrasion</td>
<td>Rock material hardness and toughness in relation to amount and hardness of sediment load particles sliding, rolling, or saltating in the flow.</td>
<td>Field observation of bedload deposits; evaluation of watershed for sources of hard bedload materials. Laboratory measurements of abrasion rates.</td>
</tr>
<tr>
<td>Quarrying or Plucking</td>
<td>Rock mass discontinuities; orientation and roughness of joints and fractures; block sizes and shapes; general blocky or smooth shape of channel in jointed rock formations.</td>
<td>Field observations and examination of core from borings; measurement of joint spacing and orientation; examination of fracture conditions and filling materials for pre-conditioning for block removal.</td>
</tr>
<tr>
<td>Cavitation</td>
<td>Rare in natural channels; requires very steep, narrow rock channels in which high velocities can occur with deep flows. Probably no bridges will be exposed to cavitation processes.</td>
<td>Check hydraulic parameters against threshold conditions in plot of mean velocity versus mean flow depth and slope.</td>
</tr>
</tbody>
</table>

Plans are being made to visit six bridge sites with varying geologic conditions, as indicated on Figure 9. Laboratory testing is being planned, also, particularly to try to address the time-rate of scour. Numerical modeling of open-channel flow over rock will be conducted by Dr. Erik Bollaert, a rock-scour team member from AquaVision in Lausanne, Switzerland. In addition to
guidelines for determining time-rate of scour and scour depth, guidelines will be developed for design and construction of bridge foundations on rock. An additional task is being undertaken to develop semi-probabilistic hydrologic data for bridges on ungaged streams.

Reasonable variety in geologic condition and climate is clear from the locations of the Phase II field sites. The glacial till site is the I-90 Bridge across Schoharie Creek in New York where the Erodibility Index approach seems to have been successful and substantial post-failure information was developed. Soft limestone in Florida and soft sandstone in Oregon are sites which have been evaluated by FDOT and ODOT, so hydraulic and geologic data are readily available. The interbedded sandstone and siltstone site in southeast Utah is at a bridge that currently is being evaluated by UDOT because of problematic rock erosion. The hard limestone site in Kentucky was one of the many sites reported by Hopkins and Beckham (1999) to have negligible scour. The volcanic rock site in California has been examined by Caltrans and is being monitored for scour problems.

Many other geologic conditions exist but budget limitations prevent additional sites from being evaluated as part of this research. Geologic conditions that would be interesting to evaluate include granitic rock, claystone, and soft rock channel material (e.g., claystone) with hard rock bedload fragments (e.g., basalt) such as exists in parts of western Colorado and eastern Utah.
Dr. Jean-Louis Briaud, a rock-scour team member from Texas A&M University, has been conducting an independent rock-scour project for TxDOT; he has learned that no rock-founded bridges in Texas have scour problems. Two potential benefits of the NCHRP Project No. 24-29 research are (1) guidelines for evaluating time-rate of scour and scour depth and (2) a method for removing rock-founded bridges from the scour-critical list based on current HEC-18 procedures that treat all foundation materials as sand. Dr. Briaud’s findings in Texas underscore the benefit of having a method of removing bridges founded on stable bedrock from the scour-critical list.

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EVAPORITE KARST IN THE SEMIARID WESTERN UNITED STATES; EXAMPLES FROM SOUTH DAKOTA, WYOMING AND ARIZONA

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ABSTRACT

Evaporite rocks composed of gypsum, anhydrite and halite underlie approximately one-third of the United States. Dissolution of these rocks in many areas of the semiarid southwestern United States generally has occurred at depth, in places more than several hundred feet deep, and has resulted in ground subsidence in many areas of the country. Evaporite karst has resulted in hazards similar to those in carbonate rocks, affecting houses, roads, and other man-made structures, and has also affected formational hydrologic characteristics. The U.S. Geological Survey, in cooperation with State geological surveys, other federal agencies, and academia, is preparing a digitized map showing the distribution of karst in the conterminous United States, including carbonate-rock karst as well as karst in evaporite rocks. The present Engineering Aspects of Karst (Davies and others, 1984) will be revised to better display extent of potential karst based upon surficial exposure of mapped bedrock. Evaporite rocks are poorly represented on that map and in some areas they are not shown at all. The varieties of karstic features that are produced by dissolution of the host evaporites, as well as by the collapse in overlying strata, include intrastratal collapse breccia, breccia pipes, and sinkholes. The differences between karst in carbonate and evaporite rocks in the humid eastern United States, and those in the semi-arid to arid western United States are delimited approximately by a zone of mean annual precipitation of 32 inches. The Holbrook Basin of Arizona is an example of the surface expression of collapse due to removal of salt at depth. Thick halite, anhydrite, and sylvite, interbedded with Permian red beds, have been removed resulting in the development of presently active collapse structures in overlying non-soluble clastic rocks. Sinkholes in sandstone, thus produced, may be placed within the definition of karst, even though they themselves were not directly removed by solution. In the Black Hills of South Dakota and Wyoming, gypsum and anhydrite deposits are found in four stratigraphic units ranging from Pennsylvanian to Jurassic in age. Evidence of recent collapse includes fresh scarps surrounding shallow depressions, sinkholes more than 60 feet deep, and sediment disruption and contamination in water wells and springs. Subsidence has caused damage to houses and water and sewage retention sites. Substratal anhydrite dissolution in the Minnelusa Formation (Pennsylvanian and Permian) has produced breccia pipes and pinnacles, a regional collapse breccia, sinkholes, and extensive disruption of bedding. Collapse features may extend as much as 1,000 feet into overlying rocks. Anhydrite removal in the Minnelusa probably dates back to the early Tertiary when the Black Hills was uplifted and anhydrite removal continues today. As the anhydrite dissolution front in the subsurface Minnelusa moves down dip and radially away from the center of the Black Hills uplift, these resurgent springs will dry up and new ones will form as the geomorphology of the Black Hills evolves. Abandoned sinkholes and breccia pipes, preserved in cross section on canyon walls, attest to the former position of the dissolution front. Processes involved in the formation of gypsum karst should be considered in land use planning in this increasingly developed part of the northern Black Hills.
INTRODUCTION

During the energy crisis in the 1970s, the USGS (U.S. Geological Survey) prepared maps at a scale of 1:7,500,000 (the National Atlas) showing the distribution of geologic hazards in the United States, such as swelling soils, volcanic eruptions, earthquakes, and landslides. Another map, Engineering Aspects of Karst was published by Davies and others (1984) depicted areas of karstic rocks (limestone, dolomite, and evaporites), and “pseudokarst” areas. These were classified as to their engineering and geologic characteristics (size and depth of voids, depth of overburden, rock/soil interface conditions, and geologic structure). This generalized map was the only synopsis of karstic conditions in the country (fig. 1). However, Federal, State, and local government agencies, the speleological community, and academia have repeatedly expressed the need for a more accurate and detailed national map to better understand the distribution of soluble rocks in the United States. Maps at a variety of scales are needed for the following: to educate the public and legislators about karst issues; to provide a basis for cave and karst research; and to aid Federal, State, and local land-use managers in managing karst-related issues, including subsidence and protecting cave resources. With increased development in the western United States, the hazard of evaporite-karst collapse is becoming more prevalent.

![Figure 1. Map showing the distribution of karst and “pseudokarst” in the United States. (Generalized from Davies and others, 1984)](image-url)

The present USGS “National Karst Map Project” will produce an updated map in digital form, derived primarily from maps prepared by and for the individual States, and to link that national map on a web-based network to state- and local-scale maps and related data (Weary and others, 2008). The USGS will facilitate compilation of the national map by cooperating with State geological surveys to update or produce state karst maps and to establish standards and consistent digital products. The newly formed National Cave and Karst Research Institute (http://www2.nature.nps.gov/nckri/index.htm) will establish a web-based network of karst information that will be used to augment the national map.
EVAPORITE KARST IN THE UNITED STATES

The occurrence of karst in carbonate rocks (limestone and dolomite) in the United States is fairly well known, as shown by the Davies map. However, the widespread distribution of karst in evaporites, including gypsum, anhydrite and salt, in the United States is not as well appreciated. Evaporites underlie about one-third of the country, but are not everywhere exposed at the surface. Gypsum is many times more soluble than limestone in water. Therefore, karstic features such as sinkholes, caves, and fracturing and collapse of overlying non-soluble rocks are more readily developed in gypsum than in limestone. Halite and other salts are even more soluble than gypsum.

In the humid eastern United States, where average annual precipitation commonly is greater than approximately 30 inches, gypsum deposits generally are eroded or dissolved to depths of many meters below the land surface. In the semi-arid western part of the United States, gypsum tends to resist erosion and typically caps ridges, mesas, and buttes. In spite of its resistance to erosion in the west, gypsum commonly contains visible karst features, such as cavities, caves, and sinkholes, attesting the importance of ground-water movement, even in low-rainfall areas. Salt is so soluble that it survives at the land surface only in arid areas, such as in Sevier Valley, Utah, and Virgin Valley, Nevada and Arizona (Epstein and Johnson, 2003). Elsewhere, salt has been dissolved to depths ranging from tens to hundreds of feet below the present land surface.

While the distribution of carbonate karst on Davies’ map is generally adequate, his map only depicts gypsum karst in a few areas (see fig. 3). In an extensive text on the back of the map, Davies mentions caves and fissures in gypsum in western Oklahoma and the eastern part of the Texas Panhandle. His map does not show the distribution of salt or salt karst, however, even though his text mentions natural subsidence and man-induced subsidence due to solution mining in salt beds in south-central and southwestern Kansas.

Several national maps of evaporite deposits were prepared earlier than the map of Davies. The first was a map by Adams and others (1904) that showed several mines in gypsum throughout the United States, supplemented with a few detailed regional maps of gypsum-bearing formations. A similar map, with additional localities, was produced by Stone and others (1920). Krumbein (1951) depicted the subsurface and outcrop distribution of evaporites in a variety of lithologic associations and by individual geologic periods. He showed that these rocks are much more widespread than generally realized. Figure 2 is a compilation of his ten systemic maps. Withington (1962) prepared an annotated bibliography of gypsum and anhydrite deposits, categorized by age. The distribution of rock salt (halite) deposits was mapped by Pierce and Rich (1962), and this was upgraded by Johnson and Gonzales (1978) and by Ege (1985). The precursor of the 1984 Davies’ map was one prepared earlier (Davies and Legrand, 1972) and was based on Davies’ extensive knowledge of karst systems in the country. The present-day understanding of the widespread distribution of marine evaporites in the United States was presented by Smith and others (1973). They showed four categories of deposits, in combinations of gypsum, anhydrite, halite, polyhalite, sylvite, and carnallite. Modification of earlier national
anhydrite maps were prepared by Dean and Johnson (1989), and Johnson (1996), and finally the map of Johnson (1997) showed generalized areas of evaporite karst throughout the United States.

Figure. 2. Distribution of evaporite deposits in the United States. Modified from Krumbein, 1951.

Figure 3 combines the information of Ege (1985) and Johnson and others (1989) showing the present perception of evaporite distribution and evaporite karst in the United States, including the limited areas of evaporite karst depicted by Davies and others (1984) and the larger areas shown by Johnson (1997). Collapse due to human activities, such as solution mining, are also shown, as well as a line of mean annual precipitation (32 in.) that may approximate the boundary between distinctively different karst characteristics, both carbonate and evaporite karst, between the humid eastern United States and the semi-arid west. Also shown are surface and subsurface evaporite-karst features in the Holbrook Basin in Arizona and the Black Hills of South Dakota and Wyoming. Evaporite karst subsidence, and the hazard it presents to the nation’s infrastructure, has not received the publicity attained by subsidence in carbonate rocks, mainly because collapse in evaporite rocks occurs in less populated areas. With continued development in the semi-arid western United States, the hazard is becoming more prevalent. Johnson (2005, and references therein) describes many areas where subsidence due to evaporite dissolution in the United States is an ongoing hazard. Other examples of areas where sinkholes have developed in evaporites include west-central Colorado (Kirkham and others, 2003), Texas (Gustavson and others, 1982), New Mexico (Land, 2003), Arizona (Neal and others, 1998), Oklahoma (Johnson, 2003), South Dakota (Stetler and Davis, 2005), and Michigan (Black, 2003).
Figure 3. Map showing distribution of outcropping and subsurface evaporite rocks in the United States and areas of reported evaporite karst. The 32.5" mean-annual-precipitation line approximates a diffuse boundary between eastern and western karst. From Epstein and Johnson, 2003.

HOLBROOK BASIN, ARIZONA

The map by Davies (1984) shows areas of carbonate rocks in Arizona, but not evaporites. Several workers have reported a variety of evaporite- and carbonate-karst features that are not found on his map (fig. 4A). The anhydrite basins shown are similar to the ones depicted in Figure 3. Subsurface halite deposits were mapped by Eaton (1972), Johnson and Gonzales (1978), Ege (1985), and Neal and others (1998); more detailed mapping of salt deposits in the Holbrook Basin was done by Pierce and Gerrard (1966) and Rauzi (2000). An area of breccia pipes was delimited in northwest Arizona by Harris (2002); they were probably the result of collapse over carbonate rocks, but evaporite collapse may not be ruled out. Scattered gypsum and anhydrite localities were shown by Withington (1962).
The Holbrook Basin in east-central Arizona is interesting because it demonstrates that dissolution of deeply buried evaporites can cause subsidence of overlying non-soluble rocks. The basin is more than 100 miles wide and contains an aggregate of about 1,000 feet of salt, anhydrite, and sylvite interbedded with clastic red beds in the Permian Sedona Group (formerly the Supai Group) (Pierce and Gerrard, 1966; Neal and others, 1998; Rauzi, 2000). The top of the salt is between 600 and 2,500 ft below the surface (Mytton, 1973). These workers describe the removal of evaporites at depth along a northwest-migrating dissolution front, causing the development of presently active collapse structures in the overlying Coconino Sandstone and Moenkopi Formation. For example, in the area about 10 mi northwest of Snowflake, AZ, the Coconino and other rocks dip monoclinally southward along the Holbrook anticline towards a
large depression enclosing a dry lake. The depression is the result of subsidence due to evaporite removal. Collapse extends upwards from the salt, forming a network of spectacular sinkholes, some of which are nearly one mile long, in the overlying Coconino Sandstone (Neal and others, 1998; Harris, 2002) (fig. 5A). Draping of the Coconino has caused opening of extensive tension fissures, some of which are many tens of feet deep (fig. 5B). The term “karst”, it seems, must include non-soluble rocks whose collapse structures are the result of dissolution of evaporite rocks below. A somewhat similar situation prevails in the Black Hills of South Dakota and Wyoming.

Figure 5. Collapse structures in clastic rocks overlying the salt-bearing Sedona Group in the Holbrook Basin, 8-10 mi northwest of Snowflake, AZ. A) Steep-sided sinkhole in a hole-pocked area called "The Sinks," located in the Coconino Sandstone. Note the variable amount of subsidence along major joints. B) Open tension fractures in the Moenkopi Formation caused by flexure of the Holbrook "anticline" (actually a monocline) due to dissolution of salt at depth. Also see figures in Harris (2002).

BLACK HILLS, WYOMING-SOUTH DAKOTA

The Black Hills is an elongate domal uplift within the semi-arid Great Plains of Wyoming and South Dakota, about 130 miles long and 60 miles wide, with an outcrop pattern of sedimentary units encircling a central core of Precambrian rocks (fig. 6). In western South Dakota it is experiencing increased urban development requiring an assessment of both the subsidence-hazard potential and ground-water contamination. There are four zones of rock with contrasting lithologies and differing karstic features. These are, from the center (and oldest) outwards: (1) The limestone plateau, made up of Cambrian to Pennsylvanian limestone, dolomite, and siliciclastic rocks, and containing world-class caves, such as Wind and Jewel Caves in the Madison (Pahasapa) Limestone. Overlying these limestones, within the plateau, is the Minnelusa Formation, which contains as much as 235 ft of anhydrite in its upper half in the subsurface. This anhydrite has been dissolved at depth, producing a variety of dissolution structures. (2) The Red Valley, predominantly underlain by red beds of the Spearfish Formation of Triassic age and containing several gypsum beds totaling more than 75 ft thick in places. The Gypsum Spring Formation, which overlies the Spearfish, contains a respectable gypsum unit that has developed
abundant sinkholes. (3) The "Dakota" hogback, held up by resistant sandstone of the Inyan Kara Group of Cretaceous age, and underlain by shales and sandstones of the Sundance and Morrison Formations. (4) Limestone and shale extending outward beyond the hogback, which are shown as karstic by Davies and others (1984) map, but which lack known karst features.

Figure. 6. Generalized diagram showing the geology and geomorphology of the Black Hills. Most of the urban development and karst features are in the Red Valley, underlain by Triassic red beds (where gypsum karst is becoming a growing concern) and in the limestone plateau, underlain by a variety of Pennsylvanian and Permian rocks. Modified from Strahler and Strahler, 1987, with permission.

Gypsum and anhydrite are conspicuous evaporite deposits in four sedimentary rock units in the Black Hills (fig 7). Gypsum and, to a lesser extent, anhydrite are exposed at the surface in many places (fig. 8). Evaporites comprise about 30 percent of the Minnelusa Formation (generally present only in the subsurface), less than 5 percent of the Opeche and Spearfish Formations, and about half of the Gypsum Spring Formation. Whereas karstic features in limestone and dolomite, such as caves and underground drainage, are abundant in the Black Hills, similar solution features are also abundant in gypsum (\(\text{CaSO}_4\cdot2\text{H}_2\text{O}\)) and its anhydrous counterpart anhydrite (\(\text{CaSO}_4\)). Calcium sulphate rocks are much more soluble than carbonate rocks, especially where they are associated with dolomite undergoing dedolomitization, a process
which results in groundwater that is continuously undersaturated with respect to gypsum (Raines and Dewers, 1997).

Figure 7: Stratigraphic column showing distribution of gypsum and anhydrite in the northern Black Hills.

Figure 8. Typical exposure of gypsum interbedded with red beds in the Spearfish Formation, about 10 mi southeast of Newcastle, WY.

**Evaporite rocks in the Black hills**

The Minnelusa Formation in the northern Black Hills consists of approximately 500 feet of dolomite, sandstone, and shale with anhydrite prevalent in the middle. The anhydrite is mostly absent in surface outcrops, having been removed by solution in the subsurface. The solution of
anhydrite and consequent formation of voids in the Minnelusa at depth resulted in foundering and fragmentation of overlying rocks, producing extensive disruption of bedding, a regional collapse breccia, many sinkholes, and breccia pipes and pinnacles (e.g., Epstein, 1958a,b; Brobst and Epstein, 1963; Bowles and Braddock, 1963)(fig. 9). Some sinkholes and resistant calcite-cemented pinnacles extend upward more than 1,000 ft into overlying strata (Bowles and Braddock, 1963). The collapse breccia consists of angular clasts of limestone, dolomite, and sandstone in a sandy matrix that is generally cemented with calcium carbonate. It has a vuggy secondary porosity, which, along with the porous sandstone, makes Minnelusa an important aquifer in the Black Hills.

Figure 9. Dissolution features in the Minnelusa Formation. A) Disrupted bedding and breccia pipes (arrow) in the upper part of the Minnelusa Formation, Hot Brook Canyon, 3 mi west of Hot Springs, SD. The Opechee shale forms the pine-covered slope above the Minnelusa, with the Minnekahta Limestone forming the top. Gypsum is not abundant in the 110 ft of poorly exposed red shale, siltstone, and fine-grained sandstone of the Opechee Formation, a confining unit between the Minnelusa Formation and Minnekahta Limestone. Regional collapse due to anhydrite dissolution in the Minnelusa has caused undulations in the Minnekahta. The lower part of the Minnelusa is not brecciated, showing that the anhydrite that was removed lay in the covered middle slope. B) Close-up of breccia pipe seen in A. C) Sinkhole (outlined) and caves in the Minnelusa Formation exposed on 400-foot-high cliff in Redbird Canyon, about 10 miles east of Newcastle, WY, in Custer County, SD. The collapse resulted from removal of anhydrite
by ground water prior to fluvial erosion, which exposed the sinkhole on the canyon wall. D) Breccia pipe with calcite cement still connected to canyon wall stands out in relief. Progressive erosion will isolate it forming a pinnacle.

The Spearfish Formation consists of about 820 ft of fine red beds with several layers of gypsum in the lower 200 ft. Anhydrite, which probably was the original form of calcium sulphate to be deposited in the Spearfish, underwent about a 40 percent expansion when hydrated to form gypsum. As a result, beds of gypsum in the Spearfish Formation are commonly highly folded (fig. 10). Additionally, when the gypsum dissolved, it become mobile and was injected downward as thin veinlets into fractures in the confining red beds (fig. 11). These veinlets are generally less than ½ inch wide, they occur along a multitude of variably oriented fractures beneath the parent gypsum bed, and they contain gypsum fibers lying perpendicular to the fracture walls. Thus, the lower 200 ft or so of the Spearfish has developed secondary fracture porosity, allowing it to supply water to wells, many sinkholes have developed in it, and resurgent springs are numerous. Ground water flows through the fractures and solution cavities in the gypsum. Thus, the lower 200 ft of the Spearfish is an aquifer at least in the northern Black Hills. This is not surprising since high ground-water flow has been reported in gypsum in many areas of the United States (Thordarson, 1989). The upper part of the Spearfish, about 600 ft thick, consists of red siltstone, shale, and very fine-grained sandstone. Gypsum beds are lacking. Bedding is regular and the unit lacks the fractures seen in the lower part of the formation. This part of the Spearfish is a confining layer.
Figure 10. Contorted gypsum in the Spearfish Formation in the Red Valley of the southwestern Black Hills, southeast of Newcastle, Wyoming.

Figure 11. Thin gypsum veinlets extending down from parent gypsum bed (not shown) and filling a multitude of fractures in the lower part of the Spearfish Formation near Cascade Springs, along State Highway 71, 13 miles southwest of Hot Springs, SD.

The Spearfish is undergoing collapse today. A series of springs that apparently occupy sinkholes, as well as many dry sinkholes, occur in the lower half of the formation, generally within 200 ft of its base, and at or near where several beds of gypsum are exposed (figs. 12, 13). Several lines of reasoning suggest that many of the sinkholes in the Spearfish Formation are too large to be accounted for by solution of the relatively thin gypsum beds within that formation; they were more likely produced by the removal of much thicker gypsum in the Minnelusa Formation, approximately 500 ft below: (1) The gypsum beds exposed in the lower Spearfish aggregate no more than about 25 ft in thickness, whereas the sinkholes are more than 50 ft deep in places, (2) Several of the sinkholes lie below many of the gypsum beds, and (3) the waters of some of the lakes occupying the sinkholes are derived from underlying formations (Klemp, 1995).

The Plains Indians that inhabited the area 300 years ago trapped and slaughtered thousands of buffalo for their primary food by stampeding the animals over the steep rim of one of the large sinkholes near Beulah, WY; the Vore Buffalo Jump, currently undergoing archeological...
excavation. This sinkhole is more than 200 feet across and about 50 feet deep (fig. 13). The hole is rimmed by several convoluted, disjoined, and disrupted gypsum beds 8 to 10 feet thick. Contortions in the gypsum here and in the surrounding area indicate hydration and expansion of original anhydrite. No gypsum is seen in the base of the sinkhole which is probably less than 50 feet above the Minnekahta Limestone. The underlying Minnekahta Limestone crops out about one mile to the west along the service road where a four-foot bed of gypsum lies at the base of the Spearfish. Layers of bones of at least 15,000 bison are found in an excavation 20 feet below the lower level of the sinkhole, indicating rapid sedimentation during the last 300 years. The sinkhole is located immediately north of I-90, and because there are many sinkholes in this immediate area (Epstein and others, 2005b), there is potential for future collapse along several miles of the highway. The Hot Springs Mammoth Site in Hot Springs, SD., is another large sinkhole in the Spearfish Formation that was the site of a breccia pipe extending down into the Minnelusa Formation. That sinkhole was a trap for large mammals at least 26,000 years ago (Laury, 1980, Agenbroad and Mead, 1994, Epstein and Others, 2005b).

Figure 12. Sixty-foot-deep sinkhole, A, within a larger 1,000-foot wide, flat-floored sinkhole, near Beulah, WY, north of I-90. This hole formed in 1985, examined by the man in the foreground. He heard running water at depth below the range of his flashlight. This suggests that passageways developed by the dissolution of gypsum at shallow depth. Accompanying this dissolution was the precipitation of thin tabular gypsum injected into the surrounding sediments (seen on the highwall to left), producing a disrupted zone and fracturing allowing for rapid movement of ground water and contributing to continued removal of gypsum.
The Gypsum Spring Formation consists of about 35 ft (11 m) equally distributed between ledge-forming white gypsum at the base and shaly siltstone with thin gypsum at the top. Many sinkholes have developed in the Gypsum Spring near Spearfish, SD. In 1972, the City of Spearfish constructed a sewage lagoon on the Gypsum Spring Formation. The lagoon leaked into sinkholes and the lagoon was abandoned in favor of an expensive water-treatment plant (Epstein and others, 2005b).

Dissolution front in the Minnelusa formation

The upper half of the Minnelusa Formation contains abundant anhydrite in the subsurface, and except for a few areas near Beulah and Sundance, Wyoming (Brady, 1931), and in Hell Canyon in the southwestern Black Hills (Braddock, 1963), no anhydrite or gypsum crops out. A log of the upper part of the Minnelusa from Hell Canyon contains 235 ft (72 m) of anhydrite and gypsum (Brobst and Epstein, 1963). Where anhydrite is present in the Minnelusa, its rocks are not brecciated. Where the rocks are brecciated in outcrop, anhydrite is absent. Clearly, the brecciation is the result of collapse following subsurface dissolution of anhydrite. The Madison and Minnelusa are the major aquifers in the Black Hills. They are recharged by rainfall on and by streams flowing across their up-dip outcrop area. In the Minnelusa, removal of anhydrite progresses downdip with continued dissolution of the anhydrite (fig. 14), collapse breccia is formed, breccia pipes extend upwards, and resurgent springs develop at the sites of sinkholes. Many lakes and resurgent springs, such as Cox, Mud and Mirror Lakes, and McNenny springs, are near the position of the dissolution front (fig. 15, 16). Cox Lake, a resurgent (artesian) spring with a flow of nearly 5 cubic feet (0.5 cu m) per second, occupies a sinkhole that is more than 60 ft (18 m) deep (fig. 15). The chemical signature of the water indicates that the
Minnelusa Formation and underlying Madison Limestone are the contributing aquifers (Klemp, 1995). The lake is near the anhydrite dissolution front shown in fig. 14. As the Black Hills is slowly lowered by erosion, the anhydrite dissolution front in the subsurface Minnelusa moves downdip and radially away from the center of the uplift. The resurgent springs will dry up and new ones will form down dip as the geomorphology of the Black Hills evolves. Abandoned sinkholes on canyon walls (fig. 9C) attest to the former position of the dissolution front.

Figure 14. Dissolution of anhydrite in the Minnelusa Formation and down-dip migration of the dissolution front.
Figure 15. Cox Lake, on left, is a resurgent (artesian) spring in the northern Black Hills. It occupies a sinkhole that is outlined by the darker water just beyond the edge of the dock. Mud Lake is on the right.

Because ground water has dissolved the anhydrite in the Minnelusa in most areas of exposure, and because anhydrite is present in the subsurface, a transition zone should be present where dissolution of anhydrite is currently taking place. A model of this zone has been presented by Brobst and Epstein (1963, p. 335) and Gott and others (1974, p. 45) and is shown here in figure 14. Consequences of this model include (1) the updip part of the Minnelusa is thinner than the downdip part because of removal of significant thickness of anhydrite, (2) the upper part of the Minnelusa should be continually collapsing, even today, and (3) the properties of the water in this transition zone may be different than elsewhere because of sulphate solution. This process suggests that present resurgent springs should be eventually abandoned and new springs should develop down the regional hydraulic gradient of the Black Hills. One such example is along Crow Creek, just east of the Wyoming-South Dakota border in the northern Black Hills, where a cloud of sediment (marl) from an upwelling spring lies 1,000 ft (300 m) north of McNenny Springs (fig. 16, X). This circular area, about 200 ft (60 m) across, might eventually replace McNenny Springs.
Solution of anhydrite in the Minnelusa probably began soon after the Black Hills was uplifted in the early Tertiary and continues today. Recent subsidence is evidenced by sinkholes more than 60 ft deep opening up within the last 20 years (fig.12), collapse in water wells and natural springs resulting in sediment disruption and contamination (Hayes, 1996), and fresh circular scarps surrounding shallow depressions. The migration of the dissolution front is similar to that reported above for the Holbrook Basin in Arizona, and for Michigan by Black (2003).

Karstic collapse due to dissolution of gypsum and anhydrite is an active process in the Black Hills. Sinkholes have disrupted foundations of houses, driveways, highways, and sewage lagoons. With increased development, especially in the northern Black Hills between Spearfish and Rapid City in South Dakota, collapse due to dissolution of soluble rocks can be exacerbated by removal of ground water by pumping. As development increase in the Red Valley, an increase in frequency of sinkhole collapse in the Spearfish Formation may be expected. Appreciation of the processes involved in the formation of gypsum karst should be considered in land use planning in this increasingly developed part of the northern Black Hills.

Subsidence caused by calcium-sulphate dissolution has resulted in collapse of houses, unstable and unsuitable sewage-lagoon sites, and draining of retention ponds (Davis and Rahn, 1997; Davis and others, 2003). Outcropping gypsum beds form level terraces in some places, and such sites are considered by some to be suitable for building homes in the Black Hills area. However, because of the potential for karst and collapse, the siting of houses upon these gypsum deposits poses obvious engineering problems (fig. 17).
Figure 17. Lookout Peak overlooks the City of Spearfish, SD, offering scenic views of the surrounding Hills. Lower down a terrace has formed on a 20-foot-thick gypsum bed in the Gypsum Spring Formation, offering a potential building site (arrow). This same bed has developed numerous sinkholes elsewhere in the area. Many of the “bumps” on the hillside are landslide debris. Note the excavation of the toe of one such landslide deposit in the middle ground in preparation for construction of a house.

**HUMID AND SEMI-ARID KARST, A COMPARISON**

Comparing the known locations of surface evaporite karst with a map showing annual average rainfall shows a striking relationship between precipitation and the occurrence of evaporite karst (fig.3). Most surficial karst features in gypsum shown in Figure 5 lies west of a zone with annual precipitation of about 32 inches (represented by the 32.5-inch isobar in the figure). Many of the
karst areas shown in Figure 5 are due to dissolution at depth. In Michigan, earlier studies suggest that the karstic collapse features there were formed soon after deposition of the Devonian evaporites (Landes and others, 1945), but Black (1997) showed that sinkhole development occurred after the most recent glaciation.

The degree to which soluble rocks are dissolved depends, in part, on the amount of rainfall and the solubility of the rock. Sulphate-bearing rocks, gypsum and anhydrite, are about 10-30 times more soluble in water than carbonate rocks (Klimchouck, 1996). Both carbonates and sulphates behave differently in the humid eastern United States and the semi-arid to arid west. Low ground-water tables and decreased ground-water circulation in the west does not favor very rapid carbonate dissolution and development of karst. In contrast, sulphate rocks are dissolved much more readily and actively than are carbonate rocks, even under semi-arid to arid conditions. The presence of extensive karst in carbonates in the west probably dates to a more humid history. Additionally, the generally thicker soils in humid climates provide the carbonic acid that enhances carbonate dissolution. Gypsum and anhydrite, in contrast, are more readily soluble in water that lacks organic acids. This relationship suggests an interesting topic for future study.

**Human-Induced Karst**

It is well known that subsidence in karstic rocks can be exacerbated by human activities. Lowering of the water table by well-pumping or by draining of quarries can reduce support of soils overlying sinkholes, thus causing their collapse. Subsurface mining of salt and other evaporites may eventually cause collapse of overlying rocks, such as at the Retsof mine in Livingston County, NY (Nieto and Young, 1998; Gowan and Trader, 2003). Localities of subsidence due to solution mining were mapped by Dunrud and Nevins (1981) and are shown in Figure 5. Ege’s (1979) bibliographic list of ground subsidence due to evaporite dissolution contains many instances where such subsidence was due to human activities. Knowing the location of shallow and deep mines is important to local officials, in order to understand the potential for such subsidence. For example, abandoned gypsum mines in western New York are abundant, and recent settlement of many houses near Buffalo, New York, partly may be the result of subsidence over these mines.

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US-70 Ruidoso Downs to Riverside, New Mexico
A Geotechnical Overview of New Mexico’s First Highway Design-Build Project

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ABSTRACT

The US 70 project, located in the Hondo Valley between Ruidoso Downs and Riverside, is the first design build highway project for the State of New Mexico. US 70 is a major east-west route, which; through the Hondo Valley is one of the most traveled and dangerous highways in the State. Twenty-seven fatalities occurred in this section between 1998 and 2002 motivating the New Mexico State Highway Transportation Department (NMDOT) to widen 38-miles of the highway.

Widening of the highway was completed in 2005. Kleinfelder was the geotechnical engineer tasked with; among other duties, the geotechnical design of miles of cut slopes in varying geologic materials.

We faced numerous challenges during the design and construction of the project. Those included: 1) completing the geotechnical design just ahead of construction; 2) completing geologic mapping and drilling on existing cut slopes up to 150-feet high; 3) dealing with geology that varied from strong limestone to weakly cemented soils; 4) overbreak created from blasting problems in the interbedded sedimentary rock; and 5) providing sufficient design information to satisfy Kleinfelder’s and the State’s design requirements while maintaining the team’s expedited schedule.

These challenges were overcome using empirical methods and pressuremeter testing, completing kinematic evaluations in the field for the “hard” rock slopes, and using back analysis of failed slopes in the “weak” rock and employing controlled blasting techniques to mitigate overbreak of the backwalls. Without finalizing the design prior to some of the widening, construction proceeded at the risk of the contractor. Our initial design was updated (and changed appropriately) once the all geotechnical information was available and during a stringent review process by the owner.
INTRODUCTION

Project Description
The US 70 project, located in the Hondo Valley between Ruidoso Downs and Riverside, New Mexico, is the first design build highway project in the State of New Mexico (Figure 1). US 70 is an east-west route that begins at I-10 just west of Lordsburg, Arizona and runs to the northeast to I-40 in Amarillo, Texas. As a result, US 70 through the Hondo Valley is one of the most traveled and dangerous highways in New Mexico (NMDOT, 2001). Twenty-seven fatal car accidents occurred within the Hondo Valley portion of US 70 between 1998 and October 2002. Five fatal accidents occurred over the busy Labor Day weekend in 2002, one month after notice to proceed and days before the beginning of construction.

One of the goals of New Mexico’s 1995 “Long Range Comprehensive Transportation Plan” was to provide four-lane or wider highways between all developed or developing areas of the State. The New Mexico State Highway and Transportation Department (NMDOT) proposed widening the two-lane US70 route to four lanes through the Hondo Valley. Notice to proceed was awarded to Sierra Blanca Constructors (SBC) on August 1, 2002. Construction of the US 70 Hondo Valley project was completed in the summer of 2005.

US 70 is a scenic route, with an alignment that winds along the foothills of mountains to the north and the Rio Ruidoso and Rio Hondo rivers to the south (Figure 2). A key requirement of the Record of Decision (FHWA, 2002) was to preserve the rural character of the Hondo Valley and minimize environmental impacts to the rivers and local irrigation features that lie south of the roadway. The highway design, as a result, required extensive cut slope excavation on the north side of US 70 and retaining wall and reinforced soil slope (RSS) construction on the south side.
Design Build
The 30 percent design-build for the project was completed in February 2002. Kleinfelder completed the design-build field investigations from August through December 2002. Design-build projects are fast paced and construction began on the cut slopes in September 2002; approximately three months before the completion of the geotechnical investigation. The alignment was divided into six segments and design information was presented to the team and NMDOT for review on a per segment basis. This allowed for work to occur along various sections of the alignment while the design in other sections was being completed.

Local Geology
The geology along US 70 in the Hondo Valley consists of two Permian-age sedimentary formations that, because of folding and faulting, are encountered repeatedly along the length of the project. A belt of intense faulting that trends north-south, accompanied by many types of intrusive as well as extrusive rocks, is found west of the alignment. This area includes Sierra Blanca, the most dominant topographic feature in Lincoln County. The Capitan Mountains are located north of the alignment, and this mountain range consists mainly of Tertiary intrusive masses and dikes of various compositions. The exposed sedimentary rock sequence within Lincoln County ranges in age from Ordovician to Tertiary.

The project is located in the Sacramento Section of the Basin and Range Physiographic Province; characterized by gently tilted strata of Permian-age sedimentary rock with occasional Tertiary intrusions. These strata are part of the Mescalero Arch and were deposited when the Pedernal Mountains were eroded and subsided. Intrusives were introduced to the strata during mountain building and uplift. Subsequent to the intrusions, Yeso Formation sediments were folded and further uplifted.

Exposed rocks along the alignment consist of sedimentary rock of Permian age and intrusive sills of Tertiary age. The Permian-aged rocks include the Yeso and San Andres Formations. The Yeso Formation consists of interbedded siltstone, mudstone, limestone, sandstone, shale, and gypsum. The limestones are generally argillaceous (Figure 3). The sandstone beds are generally pinkish yellow and the shales are variegated. The gypsum beds are white to gray and contain thin partings of shale and silt. The measured thickness of the Yeso formation has been reported to range up to 2000-feet near Picacho (Griswold, 1995).
The San Andres overlies the Yeso Formation and is composed mainly of gray limestone with lenses of white to gray sandstone (Glorieta Sandstone Member) and gypsum (Figure 4). This unit has been reported as being up to 700 feet thick (Allen, 1951). Younger, intrusive dikes and sills have been introduced to the Permian sedimentary rocks. The sills consist of a gray syenite and diorite. Thicknesses of the sills vary from a couple to tens of feet.

Soil deposits along the alignment are Quaternary in age. Stream and valley alluvial deposits are the predominantly located along the western portion of the alignment. The valley fill and fan deposits consist of clay, silt, sand, and gravel and cobbles that, in places, are partially cemented. Colluvium and talus are present along the alignment. The colluvium consists of unsorted and unconsolidated clay, silt, sand, gravel, cobbles and boulders resulting from debris flows and other debris movement along steeper slopes. Clay to boulder-size materials are also common in the arroyos. The talus deposits consist of residual soil deposits of degrading slopes and cuts.

**Design Challenges**

With respect to engineering the rock and soil roadway excavations (mostly located on the north side of the alignment), there were a number of geotechnical challenges. Those included: 1) Completing the geotechnical design just ahead of construction; 2) Completing geologic mapping and drilling on existing cut slopes up to 150-feet high; 3) Dealing with geology that varied from strong limestone to weakly cemented soils; 4) Preventing overbreak of the final backwalls of the cut slopes from blasting because of the weak interbedded sedimentary rock units. 5) Providing sufficient design information to satisfy Kleinfelder’s and the State’s design requirements while maintaining the team’s accelerated schedule.

These challenges were overcome using empirical methods to evaluate the stability of the weakly cemented alluvial soil slopes, pressuremeter testing to establish shear strength parameters for the weakly cemented soil and...
weak rock, completing kinematic evaluations in the field for the “hard” rock slopes, and using back analysis of failed slopes in the “weak” rock.

The following sections describe our geotechnical investigation and design methodologies. Finally, we discuss the construction issues involved with construction of the cut slopes along the alignment.

INVESTIGATION OVERVIEW

Geologic Mapping
During our field investigation, we completed a geologic map of the entire US 70 alignment from Ruidoso Downs to Riverside. A geological map at an approximate scale of one inch equals 100 feet (1:1200) was developed to estimate the locations of the various formations, alluvium and potential landslide features along the alignment (Figure 5).

Rock Slope Mapping
As part of our field reconnaissance at rock cut slope locations, we completed detailed geological and geomechanical mapping (Figure 6). Detailed line mapping was completed at 250 ft intervals (as recommended by FHWA, 1998) where rock cuts were required. The mapping windows were 50 feet long and as high as the existing rock outcrop. Much of the information collected during our outcrop mapping activities is attendant to the condition of discontinuities within the exposed rock masses including discontinuity information such as dip, dip direction, joint roughness and weathering characteristics. General rock mass information was collected to assess the quality of the rock mass and estimate the rock quality designation (RQD), Rock Mass Rating (Bieniawski, 1989) and Geological Strength Index (Hoek, 1997).

Because of the height and inclination of the existing rock cut slopes, much of our field mapping was completed using mountaineering (climbing and rappelling) techniques (Figure 7). Detailed mapping was completed at more than 100 locations along the US 70 alignment.
Exploration Drilling and Test Pits
The design team performed field investigations for the US 70 Hondo Valley Design-Build project from August 2002 to January 2003. A variety of drilling and excavation equipment was required to explore the variable and often difficult subsurface and surface conditions along the alignment. Explorations were performed for the cut slopes, embankment fills, retaining walls, and bridges (Figure 8). Approximately 500 explorations were completed (through drilling and test pit excavation), which equaled to approximately 10,000 lineal feet of drilling along the US 70 alignment.

In Situ Testing
Pressuremeter tests (PMT) were performed in selected bore holes in areas of weak rock and alluvium. The PMT was used to estimate the shear strength of the very weak rock or soil by inserting a flexible, fluid- or gas-filled membrane into a prepared drill hole. The membrane is pressurized and expands against the soil or weak rock until the desired strength results are reached (in the case of weak rock) or the soil or rock fails in shear. The amount of pressure required to displace or to fail the soil or rock in shear is recorded via computer and then the field data is reduced to estimate the shear strength of the soil or rock. The PMT data was supplemented by laboratory triaxial and index testing to develop strength correlations for the various strata along the corridor.

Laboratory Testing
Laboratory tests were performed to characterize the soils and rock and to develop indices and properties of the soils and rock. Laboratory tests performed during this investigation consist of:

- Atterberg Limits
- Grain Size Distribution
- Moisture Content
- Direct Shear
- Triaxial Strength
DESIGN OVERVIEW

Rock Slope Design
The development of recommended rock slope configurations was based on four different analyses or “checks”. These analyses included 1) kinematic evaluation, 2) rock mass durability, 3) global stability, and 4) rockfall hazard evaluation.

Kinematic Evaluations
The rock slope design was not overly challenging. The majority of the geology is sedimentary rock that is orthogonally-jointed and horizontally bedded and therefore kinematic release of blocks other than toppling that resulted from undercutting buy the weaker, sedimentary rock were not expected at most locations. Nonetheless, the first check on the rock slope stability was the kinematic potential for large scale rock blocks to fail out of a planned slope. Because of the geologic structure, the choice of discontinuity friction angles had little bearing on the kinematic evaluation. Based on the results of our analyses, kinematically stable slopes ranged in inclination from 45 degrees (1H:1V) to 76 degrees (0.25H:1V). Figure 9 displays a design cross-section showing a pole plot stereonet and a Markland Analysis (based on dip vectors).

Rock Mass Evaluations
Given the very weak nature of the younger sedimentary units that were going to be exposed in many of the rock excavation, we felt that it was prudent to evaluate the “global” stability of many of the planned slopes and therefore, the shear strength of the rock masses was estimated using the most recent Hoek Brown Failure Criterion published at the time of the investigation (Hoek et al, 2002). In the weaker Yeso formation, this was a valid failure mechanism and we supplemented the Hoek Brown Failure estimate with back analysis and also by PMT testing as described below. We completed our slope stability analyses. Based on the evaluation of the rock mass stability, we predicted stable slope inclinations ranged from 34 degrees (1.5H:1V) to 76 degrees (0.25H:1V) depending on the intact rock strength and anticipated condition of the exposed rock mass once constructed.

Rock Mass Durability
Rock mass durability of the Yeso Formation was a concern during cut slope design. The existing slope shown previously in Figure 3, exhibits erosion runnels common within the existing slopes constructed in the Yeso Formation “red beds.” Slake durability testing on the Yeso Formation indicated a low durability. With regards to the weak rock lithologies throughout the project, the rock mass durability ultimately controlled the slope design.
Rockfall Catchment Area Design
Rockfall catchment was paramount in view of the owner and after careful deliberation, the owner and the design team settled on using 90 percent rockfall catchment as a minimum design requirement. The catchment areas were evaluated using the Colorado Rockfall Simulation Program (CRSP) which was utilized to simulate rolling of the range of potential block sizes observed in the field. The civil design template required 13- to 16-feet wide traversable catchment areas that were and 3-feet deep. Where at least 90 percent catchment was not achieved using the prescribed roadway templates, a concrete wall barrier (CWB) was included to the outside edge of the catchment area. Figures 10 and 11 show constructed slopes in the San Andres and Yeso Formations, respectively.

Soil Slope Design
The Preliminary Geotechnical Interpretive Report characterized the materials within the existing (and planned) cut slopes as being “dense granular soil” that is weakly to moderately cemented. Standard penetration test blow counts were typically in the range of 30 to 50 and fines content was 15 to 40 percent. Initial attempts to sample the unsaturated, weakly cemented soils using conventional techniques were unsuccessful. Pushing Shelby tubes into the
soil rarely resulted in a testable specimen and driven SPT samplers yielded highly disturbed samples suitable only for index testing. With innovative and careful drilling and coring methods, the driller was able to collect relatively undisturbed samples by coring the soil.

Geological mapping and drilling suggested the presence of caliché within the gravel, sand and silt, which caused the soil to exhibit strength characteristics similar to that of an over-consolidated material. Rainfall data from the National Weather Service, observations within exploratory borings, and field mapping during the monsoon season suggested that the presence of a phreatic surface within any of the cut slopes was very unlikely.

The last major modification to the US 70 alignment in the project area occurred at least 25 years before the design build contract. This suggested that the existing alluvial (and for that matter exiting rock slope) would be a very good indication of the performance of future cut slopes along the project alignment. As a preliminary assessment of the unsaturated, weakly cemented, soil we observed and graphically plotted the height and slope inclination of the existing alluvial cuts. We completed back analyses based on those observations to establish baseline soil strengths to use in a more rigorous analysis. Based on this information alone, a conservative design was made. Figure 12 is a plot showing the height and inclination of many of the alluvial cut slopes that existed along the alignment at the beginning of the project.

By inspection of Figure 12, and assuming that the base friction angle of the granular soil is less than 40 degrees, it is clear that a portion of soil strength may be attributed to an apparent cohesion intercept (when considering the soil as a Mohr-Coulomb material).

We conducted triaxial and direct shear tests on the alluvial soil. It has been well documented within the geotechnical literature that undisturbed testing of unsaturated, weakly cemented soil yields highly ambiguous results because of wetting and disturbance of the soil during the sampling process (Walsh, 1997). Our field investigation and testing early in the project did not demonstrate the cemented soil’s strength to the satisfaction of Kleinfelder or the NMDOT reviewers. Thus, it was agreed that in situ testing would provide the most accurate representation of the soil strength, and would demonstrate that cut slope excavations similar to the existing cuts would have adequate safety factors.
Pressuremeter Testing

Pressuremeter testing (PMT) was used to estimate the insitu strength of the unsaturated and weakly cemented alluvial soils. The PMT proved to be effective and we were able to develop a numerical analysis method to estimate a cohesion intercept for the unsaturated soil and weak rock.

The pressuremeter measures the wall displacement for a given pressure. The general shape of the pressure expansion curve results from the material properties of the soil being stressed during the tests. If the unsaturated weakly cemented alluvial soils behave in a predominantly frictional manner with a cohesive component, the simplest model to describe this behaviour requires at least five parameters: (1) insitu lateral stress; (2) friction angle; (3) cohesion intercept; (4) the shear modulus; and (5) dilation rate.

We estimated the cemented soil’s friction angle based on published references (Bowles, 1968; Terzaghi et. al., 1996, & Peck et. al., 1974). We assumed that the addition of cementation would not affect the frictional component of strength (Clough et. al., 1981). Using our friction value, an estimated influence from insitu lateral stress, and taking a conservative estimate for the dilation rate, the cohesive intercept for a particular friction angle could then be estimated by comparing the field data to an ideal pressuremeter test in which the one variable, cohesion, is varied.

We conducted a number of consolidated-undrained triaxial shear tests on relatively undisturbed samples. Figure 13 is a plot of the triaxial test results and the pressuremeter results for coarse-grained unsaturated, weakly cemented soil. Note that the pressuremeter data was interpreted assuming a lateral earth pressure equal to one-half the estimated vertical pressure ($k_o$ of 0.5). The PMT tests were completed in normally consolidated soil and at the crest of excavations made twenty-five years ago. Therefore, we believe that the assumption that the lateral earth pressure is one-half the vertical pressure is conservative.
The PMT testing confirmed that designing the new cut slopes to “mirror” the existing slopes would provide an adequate factor of safety against global failure. In fact, based on the PMT results, higher, and much steeper slopes would have been justified. In the end, the slopes were constructed based on our empirical observations because those provided an indication of the weathering characteristics of the soil. Figure 14 displays a photograph of slope constructed as part of the design build contract in the weakly cemented alluvial soils.

More details regarding engineering of the weakly cemented soil slopes along the US 70 alignment can be found in Fisher and Hughes (2004).

**CONSTRUCTION OVERVIEW**

**Construction Challenges**

As stated previously, construction was started while the geotechnical design was being completed. Therefore, the construction was completed in sections. Maintaining safe passage for pedestrian traffic while providing right-of-way for

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Figure 13: Plot of PMT and Triaxial Test Data for Predominately Coarse-Grained Soil

Figure 14: Constructed soil cut slopes
exploration drilling, test pits and construction activities created multiple traffic congestion areas along the alignment.

Unexpected archeological sites were uncovered during the construction activities requiring civil and geotechnical design changes that resulted in higher and steeper cuts being required on the north side of the alignment or larger retaining walls on the southern side of the alignment. The cut slope design or alignment was changed to avoid disturbing the sensitive area.

Given the extent of the roadway improvements (38 miles), it was not feasible to perform geotechnical investigations at sufficient intervals to explicitly describe the all of the varying geological conditions that would be encountered during construction. Therefore, unexpected geologic conditions were encountered and the cut slope (and retaining wall) designs were updated during construction. This required “real-time” field engineering assistance and competent field personnel.

As stated above, rock slopes were designed based on not only the kinematic characteristics of the discontinuities but the weakest units within the rock slope. Reinforcement of the slope with rock anchors or shotcrete or draping the slopes was not an option. Therefore, our rock slope design had to consider geometry, kinematics and the strength characteristics of the rock mass. In some unexpected cases, weak and fractured sedimentary rock overlay more competent rock units requiring design field design of composite slopes. For instance, this would in some cases require laying the top portion of the slope back and while steepening the lower portion of the slope say to 0.5:1. In other cases such as a major cut slope at Picacho Hill, weak mudstones strata were sandwiched between more competent sandstones. At this location we steepened the sandstone units to about 0.5:1 above and below the weak mudstone while the mudstone unit was benched and flattened to account for weathering and prevent undercutting of the overlying sandstone which could lead to toppling failures.

Of all the cut slope construction issues encountered related to the varying geological conditions, excavations methods; especially rock blasting, presented a substantial challenge to the construction of the new alignment,

Blasting Challenges
Realignment of US-70 required major excavations using sidehill and through-cut construction blasting techniques. As with any excavation project, the geology played a major role in the blasting techniques and the shape of the backwalls of the road cuts (Figure 15).

In review, the rock slopes were excavated in Permian-aged sedimentary rock units. The San Andres Formation is characterized as strong and moderately weathered limestone and sandstone units which can exhibit massive beds. The Yeso Formation is another sedimentary unit and includes weak, interbedded highly weathered sandstones, siltstones, mudstones and shales with some gypsum strata. In some locations, igneous intrusions of diorite (strong and moderately weathered) interspersed the sedimentary units.

Because of the interbedded nature of the sedimentary rock units, where soft weak shales, mudstones and gypsum layers were interbedded with more competent sandstones or limestones, blasting problems were not uncommon. The most common challenge was control of overbreak of
the benches or final backwall of the rock cuts. Overbreak or backbreak is the rock volume broken beyond the plane defined by the last row of blast holes.

Wyllie and Mah (2004), Konya (2003) discuss slope instability as it relates to blast damage behind the face of the rock cut. Blast induced damage is often surficial and may possibly extend 15 to 30 feet behind the open face. The damage can result in rock fall over time as water enters the fractures, freezes and by expansion and opens the cracks and loosens the rock blocks. Blast damage can cause extensive damage where the rock slope contains persistent bedding planes that dip out of the slope face. In this case, explosive gases may travel up the planes resulting in displacement of blocks of rock. In addition, in the weak and fractured zones of the rock, gas pressure may be lost creating an inefficient shot and resulting in a ragged slope.

On US-70, the blasting contractor planned for and mitigated the blast damage to the final walls by implementing proper production blast designs and employing controlled blasting techniques. Production blasting was designed to limit rock fracturing behind final wall. In addition, controlled blasting techniques such as preshearing (presplit) and cushioning blasting techniques were employed to define the final faces. Controlled blasting allowed for steeper slopes which resulted in reducing excavation volume and additional land impact resulting in cost savings to the project.

To improve slope stability and avoid overbreak or backbreak during production blasting, the following was addressed in the blast design:

1. The front row of blast holes behind the free face was designed to account for and move the burden. Burden is the distance between the free face and the first line of blast holes. Typical shot patterns were 6-ft by 6-ft with 12-ft lifts (Figure 16).
2. Stemming in the shot hole was designed to account for the burden, diameter of the blast hole and the unconfined compressive strength of the rock. Stemming is inert material such as crushed rock inserted into the collar of the drill hole to confine the explosive gases. Typical stemming thickness in the shot holes was about 4-ft.
3. The shots were designed with adequate delays and timing intervals for movement of the rock to the free face and creation of additional free faces for the blast holes behind the present free face. The ratio of the timing between shot rows to the burden ranged between 4 and 6 to minimize overbreak.
4. Delays were employed between blast holes and rows to control the maximum instantaneous explosive charge and reduce vibrations.
5. The back row of shot holes and “buffer holes” were typically drilled about 8-ft from the final face and line of holes to facilitate excavation and minimize damage to the final wall.
On all blasting projects, it is paramount that the blasting contractor or engineer conduct test blasting to establish if the blasting design will work as designed. Geology plays a critical role in the blasting design and outcome. After conducting a series of test blasts in the weak interbedded sedimentary rock, it was established that controlled blasting using cushion blasting techniques as opposed to preshearing (presplit) techniques would be required to minimize overbreak of the final backwall of the cuts.

As a review, in controlled blasting: closely spaced parallel holes drilled at the final face and are lightly loaded with an explosive that has a diameter smaller than the drill hole. Explosive suppliers manufacture and distribute special explosives for controlled blasting. (On US-70 Dynosplit C was the standard explosive for this job). The air-gap between the explosive and the drill wall provides a cushion that attenuates the explosive shock wave transmitted to the rock. The pressure formed from the explosion is insufficient to crush the rock around the hole. However, the radial fractures will preferentially create a clean break or shear zone between blast holes forming a clean rock face.

The difference between preshearing (presplit) blasting and cushion blasting is the sequence of the shot and the loading and spacing of the holes. Presplitting employs lightly loaded that are closely spaced (≤ 30 inches) Detonation of presplit shot holes occur milliseconds before the production blast creating a clean stable face before the production blast fires; hence the term preshear or presplit. In homogeneous rock the results are generally very good. In addition, presplitting is a protective measure to keep the final wall from being damaged by the production blast (Konya, 2003).

Conversely, with cushion blasting, the final rows of explosives are detonated milliseconds after the final production line of blast holes. The holes are separated further and have a higher explosive load to move the burden. Cushion blasting is typically employed for trimming and attaining a smooth wall or when the burden thickness is less than the bench height (Wyllie and Mah, 2004).

On US-70, results from cushion blasting were mixed (Figure 17). Where the rock units were competent and massive, results were good. However, overbreak was still observed in units that exhibited beds that we less than 4-ft thick interbedded with strata of shales and mudstones or highly fractured units. According to Konya (2003), since the cushion blast is the final row of holes along the cut face, they are last to detonate in a production shot and they are not as efficient to protect the stability of the final wall. In the cases where cushion blast exhibited poor results on US-70, the discontinuities may have channeled the explosive gasses into the backwall from the
production blast creating overbreak. In other areas overbreak occurred, because the bedding or structure dipped out of slope and some of the explosive gasses followed the structure beyond the final face.

**CONCLUSIONS**
The US 70 project was the first design build highway project for the State of New Mexico. The project was professionally rewarding as well as challenging. Construction challenges were overcome using empirical methods and pressuremeter testing, completing kinematic evaluations in the field for the “hard” rock slopes, using back analysis of failed slopes in the “weak” rock and employing controlled blasting techniques to mitigate overbreak of the benches and backwalls. Without finalizing the design prior to some of the widening, construction proceeded at the risk of the contractor. Our initial design was updated (and changed appropriately) once the all geotechnical information was available and during a stringent review process by the owner.

**REFERENCES**


A NEW GUIDANCE DOCUMENT FOR MITIGATING IMPACTS FROM ACID-PRODUCING ROCK FORMATIONS IN TENNESSEE ROAD CONSTRUCTION PROJECTS

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ABSTRACT

When portions of the Chattanooga shale and other pyrite-bearing or sulfide-bearing rock formations are exposed in Tennessee Department of Transportation (TDOT) road projects, there is a potential for runoff to become polluted with sulfuric acid and metals (mostly iron) when the pyrite/sulfide rock weathers. As a part of surface water pollution management, TDOT recently updated its 18-year-old standard operating procedure (SOP) for dealing with this important issue to create a new guidance document. In the process, a team of geologists and GIS experts developed a GIS database of information that TDOT could use to quickly identify projects that might need to follow the new guideline to avoid impacts. This information includes zones of geologic formations known to contain pyrite and formations containing acidic pH-neutralizing rocks such as carbonates. The GIS database was also configured to not only receive the wealth of analytical data that TDOT has assembled over the past decade on pyrite-related road projects but to allow addition of new information in the future. The project team geochemists further compiled the latest research on pyritic rock characterization and testing and compared it to protocols found in TDOT’s existing SOP.

The new guideline document, building on years of TDOT’s actual experience, was also based on mining industry experience in mitigating pyrite-derived impacts. It was recognized that despite the implementation of up-to-date Best Management Practices (BMPs), some residual acidic/metal runoff may occur. For these situations, the guideline provides passive treatment system (a.k.a. constructed wetland) BMPs, again based on mining industry derived experience. TDOT’s new guidelines are the most comprehensive construction related acidic rock drainage BMPs of any state DOT.

Additional Keywords: pyrite remediation, passive treatment, geographic information systems

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INTRODUCTION

The Tennessee Department of Transportation (TDOT) has in recent years been involved in the detection, testing, and mitigation of rock material containing minerals that, under certain conditions, are capable of producing acidic runoff. In late 2006, a focused effort began to replace an earlier Standard Operating Procedure [SOP] (TDOT, 1990) regarding this issue. The new guideline was based on existing literature and published practices by others faced with the challenges of encountering Acid Producing Rock (APR) which can lead to acid rock drainage (ARD). The new document [Guideline for Acid Producing Rock Mitigation] [1] was developed, based on years of TDOT’s actual experience with APR-related issues. As part of this effort, a GIS database was created to efficiently identify problem sites for the new guideline to be implemented. This database includes information on pyrite-bearing formations, as well as formations containing acidic pH-neutralizing rocks, such as carbonates. The database was configured to receive and incorporate new data as it becomes available, ensuring that the guideline remains up-to-date. The project team further compiled the latest research on pyritic rock characterization and testing, allowing for a thorough comparison with established protocols. The new guideline, which builds on the experience of previous efforts, has been developed to be comprehensive in addressing all aspects of APR-related issues, including those related to constructed wetland BMPs. This new guideline is the most comprehensive construction-related acidic rock drainage BMPs of any state DOT.
Rock Investigation, Testing, Monitoring, and Mitigation], (TDOT, 2007), the “APR Guideline”, was designed to provide consistent guiding principles, rather than strict analytical/procedural protocols, to be applied to TDOT projects for investigation, prevention, and mitigation of potential ARD. Thus, it considers professional judgment as acceptable input in decision-making. Notably, it was produced in cooperation with the Tennessee Department of Environment and Conservation (TDEC).

While the primary focus of the new guideline was ARD prevention, it also included a secondary focus on ARD treatment not addressed by the original SOP.

The new APR Guideline provides direction on the following topics or phases of road construction projects:

- Project Screening and Site Assessment (Visual and Geographic Initial Assessment),
- Sampling and Testing,
- Triggers and Thresholds,
- Mitigation (Prevention and Treatment), and
- Monitoring.

Figure 1 summarizes the overall structure for projects and phases of investigation.

The first phase, Project Screening, will be conducted using geographic information system (GIS) based data, TDOT personnel professional experience, and other available geological literature and maps. The goal of this phase is to determine if a project, or a project’s components, is located in Medium- or High-Risk APR potential zones. Figure 2 shows the various risk zones as developed by the APR Guideline team. These GIS data are based on a bedrock geology map at 1:250,000 scale produced by the Tennessee Division of Geology (Hardeman 1966) and were deemed sufficiently comprehensive for use as a general guide for site geology and potential risk. Project components in Low-Risk APR zones could likely be exempted from additional phases such as sampling and testing. However, all project sites would require an initial site visit and/or knowledge from previous visits to determine that potential APR materials are not present. While primary purpose of the site visit is to verify the accuracy of the GIS mapping data and expected geology of the site, the visit may be combined with other tasks related to geotechnical data needs.
Projects with components located in Medium- or High-Risk APR zones are to follow sampling and testing guidelines during the life of the project and monitoring at the conclusion of the project. Data generated would be examined using guidance provided in the “Triggers and Thresholds” sections of the APR Guideline to identify if further sampling or mitigation measures are warranted. In addition, if potential APR materials are identified at any point during the project, the APR Guideline provides direction for...
appropriate APR mitigation design approaches. Mitigation is divided into two methodologies: prevention and treatment.

The realm of prevention and treatment technologies has two logical endpoints as shown graphically in Figure 3. At one end, an APR situation might be completely mitigated by implementing a “walk-away” prevention design remedy that is nearly permanent, requiring little or no maintenance with just cursory post-construction monitoring. The upfront costs of implementing this approach may be much more than Tennessee taxpayers are willing to spend for a new transportation project. At the other end, it may not be practical to implement APR prevention measures in which case a commitment to perpetual treatment of acidic drainage and monitoring will be required in the event that acidic drainage actually forms. The long-term costs and problems of this approach may be equally unacceptable. Some projects may have components encompassing both endpoints, and the vast number of combinations in between. The proportioning of prevention and treatment risk is to be resolved by the professional judgment of qualified engineers and/or geologists based on project- and site-specific circumstances.

![Figure 3 – Relative Costs within the Realm of APR Mitigation Strategies](image)

Guideline-based recommendations may vary within a given project depending on the current project phase and with changes in geology, site conditions, and disturbance area. Pre-, mid-, and post-construction activities may require different levels of sampling and testing. Also, due to the linear nature of highway construction projects, guideline applicability may vary with milepost/stationing as a function of the geology combined with the depth of construction and other site conditions. Lastly, the type of project might influence APR assessment and response procedures. These include:

- Building a new-alignment road in undisturbed terrain;
- Widening or modifying an existing road segment; and
- Implementing ARD mitigation at a previously-constructed project.
The principles and general direction included in the new APR Guideline were derived from existing literature, previous TDOT professional experience, past practices, and experience reported by others, such as the US EPA, other states, and the Federal Highway Administration. Germaine TDOT experience was obtained from recent TDOT highway projects involving acid producing rock material in Blount, Carter, Sevier, and Unicoi counties.

Based on literature searches as well as direct contact with the transportation departments in other states, it appears that no other state transportation agency has a guideline document for dealing with APR at this time, though several other states are aware of these issues and are researching them as well. The US EPA, many state agencies, and mining companies are confronted with APR situations related to existing and abandoned mines; therefore, it was appropriate to consider some of this experience in identifying and characterizing APR and in developing mitigation guidelines for potential APR from TDOT projects.

While the new APR Guideline attempts to provide up-to-date and state-of-the-art practices for APR and roadway construction, new tests, standards, or mitigation technologies may be developed in the future. Factors affecting APR generation include mineralogy, weathering rates, climate, material size and surface area, mineral occlusion or exposure, exposure of the material to air and water, hydrologic regime, and material placement method and location (EPA 1994; Nordstrom and Alpers 1998); these factors and their complex interactions are being continuously studied and researched in a variety of settings. Therefore, it is anticipated that the new APR Guideline will be reviewed and updated periodically to account for new developments, including findings developed in-house by TDOT based on site-specific observations at Tennessee road projects.

Those observations may include the assessments of mitigation strategies that TDOT implemented at the outset of dealing with APR issues over a decade ago. There is no better gauge of a mitigation design’s effectiveness than the test of time. The protocols developed in the new APR Guideline should facilitate this ongoing process into the future.

**APR RISK MAP GENERATION**

The GIS/APR map (GIS Dataset) was developed by researching the geology of the State of Tennessee to identify known geologic units that have the potential to be sources of APR. In addition, the APR Guideline team also identified geologic units that contain neutralizing materials for APR. As noted earlier, the GIS/APR Map was based on the Geologic Map of Tennessee (Hardeman 1966). Geologic units shown on the map legend were researched as well as a general internet web-search for APR and pyrite-containing formations in the State. While pyrite is the most common component in APR, other sulfide-bearing minerals can also be present. In addition, the team researched available hard-copy publications to complete the effort.
The research identified individual geologic formations and groups of formations that contain known APR sources, potential APR sources or sources of neutralizing materials. The team defined seven categories for the GIS/APR map (five with APR potential to varying degrees and two with APR neutralizing potential) and color coded them as follows:

- **Red** – Individual Formations which are known sources of APR.
- **Light Red** – Groups and supergroups that include formations which are known sources of APR.
- **Orange** – Formation that may contain potentially APR.
- **Yellow** – Formations that are potential sources of APR.
- **Navy Blue** – Fort Payne and Chattanooga Shale (specific, historically problematic, high APR potential rock formations)
  - Green – Limestone (material with comparatively high neutralizing ability).
  - Light Green – Dolomite (material with comparatively lower neutralizing ability).

The details supporting these categorizations would constitute a separate technical paper and are not discussed here. An example of the individual formation data is provided in the table below which is an excerpt of the geological data that was inserted in the GIS metadata table (TDOT 2007).

<table>
<thead>
<tr>
<th>Geo. Formation ID</th>
<th>Geological formation(s)</th>
<th>APR Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pcm</td>
<td>Cross Mtn Formation</td>
<td>Includes formations that may contain acid producing rock</td>
</tr>
<tr>
<td>Pco</td>
<td>Crab Orchard Mountains Group: Contains Whitwell Shale</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>Pcg</td>
<td>Crab Orchard Mountains and Gizzard Groups: Contains Whitwell Shale</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>PCo</td>
<td>Ocoee Supergroup</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>pCw</td>
<td>Walden Creek Group: Contains Sandsuck Formation</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>pcss</td>
<td>Sandsuck Formation</td>
<td>Formation that contains acid producing rock</td>
</tr>
<tr>
<td>pCrb</td>
<td>Rich Butt Sandstone</td>
<td></td>
</tr>
<tr>
<td>pCg</td>
<td>Great Smoky Group</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>pCs</td>
<td>Snowbird Group</td>
<td>Includes formations that contain acid producing rock</td>
</tr>
<tr>
<td>pEm</td>
<td>Mount Rogers Group</td>
<td></td>
</tr>
<tr>
<td>pCr</td>
<td>Roan Gniess</td>
<td></td>
</tr>
</tbody>
</table>
The GIS/APR Map includes a number of layers that contain political or geographical information for orientation and to make the map more useful. These layers include the following:

1. TDOT Regions
2. TDOT jurisdictional roads
3. County Names, boundaries, and County seats
4. Waterways
5. 303(d)/305(b) waters impaired by pH and/or metals
6. 7.5 Minute (1:24,000 scale) USGS quadrangle map names and boundaries
8. Latitude and Longitude

The GIS/APR Map is intended to be an evolving “living” tool which can be updated and refined with more detailed information which can be incorporated into the GIS database to supersede the existing database. Several directions for additional effort or further refinement of the GIS database were identified during the research that may be of particular value. For example, a significant amount of published geologic mapping exists that could be evaluated, digitized and incorporated into the GIS/APR database to provide more detail and precision. Several areas of 303(d)/305(b) impaired waters are covered by 1:24,000 scale geologic mapping which could provide additional detail in these critical locations. In addition, site specific geologic mapping of APR data could be incorporated into the existing database. Another opportunity for refinement would be to incorporate the Soil Survey Geographic (SSURGO) Database developed by the United States Department of Agriculture National Resource Conservation Service (USDA-NRCS). This soils mapping is complete and available in digital format for much of the State. The SURRGO soils mapping provides soil properties based on shallow (60 inch deep) soils borings and laboratory testing which includes classification testing, basic soils mechanics properties, erosion characteristics, permeability and soil pH.

PROJECT SCREENING AND SITE ASSESSMENT

The APR Guideline prescribed three preliminary phases to be conducted as a part of a potential APR evaluation. The first phase is Project Screening. Project Screening should identify whether a project or project components are located in areas of Low-, Medium-, or High-Risk APR zones. These zones are based upon the geology of Tennessee and professional knowledge of Tennessee geologic formations with respect to APR. Tennessee formations have been classified as those with Known Potential (High-Risk zones), Likely Potential (Medium-Risk zones), or Minimal to Rare Potential (Low Risk zones). Locations of these zones are determined using a GIS database, published geological literature and maps, as well as internal institutional or professional knowledge.
The second phase includes a dedicated site visit and/or assessment of observations noted in previous site visits, referred to here as a Visual and Geographic Assessment. The purpose of the Visual and Geographic Assessment is to confirm that site conditions match those predicted by the database or other existing information, to assist with development of a Sampling Plan (SP), and to identify areas or zones that should be targeted for future sampling and testing.

The third phase is the development of a SP. For projects with components containing Medium- or High-Risk APR zones, a SP, or multiple SPs if necessary, should be developed. The SP(s) should be prepared at the conclusion of the Project Screening and Visual and Geographic Assessment using project site-specific information and information collected as a part of the screening. The SP, or SPs, should incorporate recommendations presented in the APR Guideline.

**SAMPLING and TESTING**

The sampling and testing section of the TDOT APR Guideline provides details for Pre-Construction and Construction Phase planning and sampling if the project is located in Medium- or High-Risk APR zones. Sampling of water and rock is required for those areas of the project that are located in Medium- or High-Risk APR zones, or in areas identified by the Visual and Geographic Assessment. Sampling may vary throughout the project or in different areas, depending on the project type, phase of the project, and results from earlier phases. Figure 4 provides a summary of the recommended water, rock, and geophysical sampling programs.

![Figure 4 – Summary of Water, Rock, and Geophysical Sampling Programs (TDOT 2007)](image)
For completeness, the APR Guidance document provides recommendations for specific sampling methods and guidance for analytical testing methods. Results from the sampling and testing would be assessed using the information contained in the Triggers and Thresholds discussion to determine if additional actions are required.

**GUIDELINES FOR MATERIAL CHARACTERIZATION AND MITIGATION THRESHOLDS**

The APR Guideline provides direction for examination and use of data collected during the sampling and testing phases of a project, as well as for the initial screening and monitoring phases. Numerical thresholds are provided for each of the testing methods, or a combination of the testing methods. If these thresholds are exceeded, additional effort, such as sampling or mitigation designs and appropriate material handling during construction, are therefore “triggered.” However, these numerical thresholds must be considered with the site-specific conditions and past or known behavior of the materials. Actual known field behavior of materials may be considered more reliable than laboratory testing performed in a sterile environment. To facilitate understanding and communication regarding this complex issue, the APR Guideline provided figures that summarized recommendations if thresholds are exceeded during the initial screening as well as flow charts for decision-making based upon water and rock sampling results.

**Visual and Geographic Assessment Thresholds**

The Visual and Geographic Assessment can provide an excellent indication that Potential–APR (P-APR) or APR materials are present or information about the field behavior of these materials. Site thresholds include the following that are associated with site geologic conditions:

- **Waters** of distinctive colors, such as iron/red, yellow, white, or black stained streambeds, or iron/red staining with large amounts of algae;
- Staining of rocks or surface materials, particularly on hillsides, streambeds, road cuts, roadways or sidewalks, or other surfaces;
- Low pH values (<5) or elevated conductivity values (>2,000 microsiemens per centimeter (µS/cm) depending on background) or;
- Kill zones, or areas devoid of vegetation;
- Cementation crusts or areas of mineral precipitation from evaporating water;
- Geologic formations at the site, as outcrops or on geologic maps—of particular interest are those known to be rich in sulfides (e.g. pyrite), have a history of APR impacts, or are carbonate materials (e.g. limestone); and
- Proximal P-APR sites, such as coal mines or, road cuts; and
- Proximal road fills and any seeps emanating from the fills.
Rock Thresholds

Laboratory test results drive the following thresholds and categories for rock materials. Based on initial laboratory Acid-Base Accounting (ABA) testing, including paste pH and pyritic sulfur values, materials will fall into one of four categories, as listed below.

- APR-Neutralizing Materials
- Non-APR Materials
- Potential APR Materials
- APR Materials

Flow charts for identifying materials falling within these categories were developed and provide the foundation of the APR thresholds; one two sets of guidelines may be applicable depending on the TDOT’s experience at a given site or geological situation.

The primary rock characterization guideline is based on existing institutional knowledge. TDOT has been actively and progressively working with Potential-APR and APR materials for many years and their practices to date have not resulted in significant ARD problems. Therefore, a primary set of guidelines has been provided based on practices to date. These guidelines may be more appropriate for sites and materials for which TDOT has previous experience where previous material handling and placement procedures have not resulted in ARD.

A second set of guidelines provides thresholds that represent state-of-the-art practices with respect to ARD evaluations applicable to the geologic setting of Tennessee but where institutional knowledge may be lacking. These thresholds are necessarily conservative in order to account for the wide variety of factors that can influence ARD development. An appropriate future course of action for TDOT may be to collect and analyze historical and current data on handling and placement of Non-APR, Potential-APR and APR materials to date in order to formally calibrate the thresholds proposed.

Selection of the particular set of guidelines should be made by a qualified engineer or geologist based on site-specific and material-specific information based on the previous experience with a site or material.

**APR Characterization Overview**

The characterization of a particular geologic horizon falls with a continuum ranging from APR-Neutralizing Materials to APR Materials. The behavior of a geologic horizon is dependent on a number factors, such as its mineralogy, weathering rates, material size and surface area, mineral occlusion or exposure, exposure of the material to air and water. Therefore, characterization of a material relies upon several tests or aspects of the material to classify it as APR-neutralizing, APR, or somewhere in-between. Paste pH, Net Neutralization Potential (NNP) or Neutralization Potential Ratio (NPR) values, and sulfur values are all considered in the APR-Guideline to determine whether a given material needs to be fully or partially encapsulated or blended. In general, avoidance of construction in APR horizons should be the preferred action.
Water Thresholds

There are several water chemistry indicators for the presence of APR. As described by Skousen et al. (1987), water affected by APR in the Appalachian region (Alabama, Indiana, Illinois, Kentucky, Maryland, Ohio, Pennsylvania, Tennessee, Virginia, and West Virginia) area generally has pH values less than 5.0 or a combination of the following:

- total iron greater than 7 mg/L,
- total manganese greater than 4.0 mg/L,
- other dissolved metals greater than EPA MCLs,
- elevated acidity,
- elevated conductivity (>2,000 µS/cm, depending on background), and
- elevated sulfate concentrations.

If these conditions are observed, then APR conditions may have developed. It is worth noting that not all of these water geochemistry indicators may be present to indicate that APR conditions are developing; professional judgment and understanding of site geology should be used to determine if all or some of these conditions present indicate the development of ARD. Additional sampling should be performed in anticipation of development of APR mitigation. If the above thresholds are observed in surface water or groundwater, this should trigger periodic measurement of flow rates, which are necessary for design of mitigation systems.

In addition, trends in water chemistry through time are just as important as the stated values above. Coupled with visual assessment clues (e.g., fresh iron staining), a professionally-judged increase in metals, sulfate, or acidity concentrations, or a coincidental decrease in alkalinity or pH values with time may be an indication that ARD is occurring. Increasing sulfate and decreasing alkalinity of the water, without increasing metals concentrations, may indicate that oxidation of sulfides and subsequent consumption of neutralizing potential (NP) is occurring. If the NP becomes fully depleted then ARD conditions may occur. Therefore, if these trends are observed, increased monitoring should be performed and APR mitigation designed if ARD conditions have occurred.

MITIGATION MEASURES

Mitigation techniques are needed for two general situations: excavated material and cut slopes. Several mitigation techniques, referred to here as Best Management Practice (BMPs) are provided for both situations. For excavated materials, techniques range from blending to full encapsulation, with an intermediate of partial encapsulation. The techniques may be viewed as distinct methods or as a continuum that may be adjusted to fit site specific conditions or materials.

Mitigation of Excavated Material
Techniques for the mitigation of APR excavated material have been proposed by the Federal Highway Authority (Byerly 1990) and TDOT (TDOT 2005). TDOT has had significant experience with APR mitigation and has published research on updated mitigation methods (Moore 1992). The current APR Guideline expands or furthers these publications and experience. Techniques or BMPs of several phases of road construction are provided below.

Design Phase Best Management Practices (BMPs)

If Pre-Construction Sampling and Analysis indicates the presence of P-APR/APR, the APR Guideline indicates that:

- Excavation of P-APR/APR should be avoided where possible and always minimized.
- The expected quantity of P-APR/APR should be estimated from construction drawings.
- Sites for disposal of all anticipated P-APR/APR should be identified.
- On-site borrow areas from which adequate quantities of cover material for burial of the APR should be identified.
- Logistics for hauling P-APR/APR, the lime and limestone, and cover material to the disposal sites during construction should be developed to eliminate, if possible, temporary storage of the P-APR/APR.
- Drainage should be diverted away from all excavations and encapsulating embankments if possible.
- Drainage ditches or other water conveyances along excavated and encapsulated APR should be lined with geomembrane or other impervious material such as clay.
- Underdrains, pipe culverts, and storm drains in areas of excavated and encapsulated APR should be constructed of inert plastic.

Blasting BMP’s

If blast hole sampling and testing indicate the presence of P-APR/APR, blast designs may be adjusted to minimize the production of “fine-grained” P-APR/APR. This BMP is implemented only if it results in blasted fragments that may be safely and cost-effectively loaded into haulage vehicles or moved into encapsulation zones.

Construction Phase BMPs

Three different Construction Phase BMPs are described in this section, including blending, partial encapsulation, and full encapsulation. These three methods are appropriate for different thresholds; however, variations or modifications to or between the methods may be appropriate given site-specific conditions or site-specific materials. These BMPs should be selected in consultation with TDEC. Four major BMPs were developed in the APR Guideline:
• Blending of P-APR and APR with APR-neutralizing material [i.e., limestone, calcareous shale, or rock material with a net neutralizing potential (NNP) value greater than 50 Tons of calcium carbonate (CaCO$_3$) per kiloton (kT) of rock]. Grain sizes and mixing recommendations are provided.
• Partial Encapsulation (See Figure 5).
• Full Encapsulation (See Figures 6 and 7) which may occur within the roadway or at a dedicated waste site repository.

Figure 5 - Partial Encapsulation Cross Section View (TDOT 2007)

The Full Encapsulation BMP conceptual design includes both clay and geomembrane liners; Figure 6 shows the geomembrane liner option.

Figure 6 - Roadway Full Encapsulation Cross Section View (TDOT 2007)
Cut Slopes - ARD Prevention

Cut slope ARD prevention BMPs include designing the slopes to be as steep as possible within geotechnical stability constraints and public safety. Pre-split blasting to minimize rock face over-break is a BMP that limits exposure of APR to water and oxidizing conditions. If near-vertical slopes are not recommended, the slopes would be flattened to allow placement of Non-APR and plant growth medium. Bactericides, which are considered a temporary BMP, may be used in this effort to suppress pyrite oxidation as the plant community matures.

Other cut slope BMPs include: attention to bench designs, stabilizing friable rock slope covers, and rapid revegetation protocols. Post-construction BMPs include the placement of oxic limestone channels and mixing of limestone into native soils/plant growth medium prior to revegetation.

WATER TREATMENT

While the goal of the guideline is to avoid generation of ARD, and if proper planning and mitigation BMPs have been followed, the likelihood of generating ARD should be minimized. However, treatment of ARD would be necessary if other implemented prevention measures have not achieved the level of control required. Water treatment is costly, and in some cases, must be continued in perpetuity. In addition, this may not have occurred at some older sites that pre-date effective mitigation methods.

The spectrum of ARD treatment ranges from active to passive and includes a “semi-passive” category. Active treatment processes typically require mixing and settling tanks, pumps, electricity, chemical addition and some level of filtration in addition to the labor required to operate and maintain these systems. Active treatment plants also generate sludge which requires disposal on a regular schedule. Because of these
permanent infrastructure requirements, active treatment systems are deemed inappropriate for TDOT projects.

Passive treatment, on the other hand, consists of oxic limestone channels, free water surface wetlands, and bioreactors that treat water without electricity, day-to-day labor, or chemical addition. Passive treatment systems (PTS) require occasional maintenance and must be refurbished, depending on the type of system, every 10 to 20 years. The primary limitation of the PTS technology is that large areas may be required to treat high flow rates and/or high metal concentrations. Some types of PTS may require National Pollutant Discharge Elimination System (NPDES) Permits.

Semi-passive treatment is an off-the-shelf technology that uses water-powered chemical feeders to add reagents either continuously or intermittently to ARD. The reservoirs of chemical reagents require refilling perhaps on a monthly to bi-monthly schedule, depending on the ARD treatment situation.

The new APR Guideline was not intended to be a PTS design manual but instead to offer direction for situations in which PTSs are appropriate. If a site requires water treatment, a qualified professional engineer should evaluate the site water, and design the appropriate PTS. Public-domain software, AMD Treat©, is available from the internet to assist the project engineer in sizing and designing a PTS and/or a semi-passive treatment system in typical situations.

**Water Treatment Implementation Triggers**

Water treatment should be initiated based on the following triggers:

- The source of the ARD cannot be eliminated or remediated; or
- Water leaving the site is in violation of TDEC water quality criteria for Fish and Aquatic Life; or
- Water leaving the site has a pH of less than 5 (site dependent).

The decision to treat water at a particular site will be based on a variety of site factors including background water quality, flow rate, land ownership, historic land use, and future land use. In the case of background water quality, it is possible for streams to have naturally-occurring pH values less than 5. In this situation, TDOT and TDEC could waive the water treatment requirement. This document attempts to provide generalized guidance for initiation of water treatment at potential ARD sites. The final decision to treat water at any particular site should be made based on the triggers listed above and TDOT and TDEC recommendations.

**AMD Treat© Public Domain Software**

AMD Treat© is a computer application for estimating remediation costs for mine drainage or generic ARD. Version 4.0 of AMD Treat© can be downloaded from the internet from the Office of Surface Mining website (http://amd.osmre.gov/amdtreat.asp); the website also offers an on-line tutorial in learning how to use the software. The software can be
used to estimate construction quantities and costs (capital and operating) for a variety of passive and chemical treatment methods; including:

- vertical flow ponds,
- anoxic limestone drains,
- anaerobic wetlands,
- aerobic wetlands,
- bio reactors,
- manganese removal beds,
- limestone beds,
- settling ponds,
- oxic limestone channels,
- caustic soda,
- hydrated lime,
- pebble quicklime,
- ammonia,
- oxidation chemicals, and
- soda ash treatment systems.

The treatment estimating modules in bold above have been identified as preferred treatment methodologies at TDOT sites. However, these preferences are not necessarily all-inclusive and other methodologies may be appropriate.

**Water Treatment Methods**

**Short-term Semi-Passive Treatment**

If ARD is discovered during construction, immediate capture and semi-passive treatment of the water should begin to prevent off-site impacts. This short-term treatment method will be employed until a permanent system is designed and built. Short-term semi-passive treatment measures follow.

- Retention Pond Sizing - The flow rate of the ARD should be measured. If the ARD flow is the result of precipitation events, a qualified hydrologist/engineer should estimate the 10-yr, 24 hr. runoff volume. A geomembrane-lined retention pond with a 24-hr retention time should be constructed to capture the ARD. See the following modules in AMD Treat©: Ponds, Flow Calculation Tools, and Acidity Calculator. Periodic sediment and/or sludge removal will be required for the retention pond. Clean stormwater should be diverted from the retention pond.

- Aquafix™ Treatment – Aquafix™ units are water-wheel powered pebble lime-dosing machines. Aquafix™ systems require neither electricity nor constant monitoring but function better under continuous flow conditions. If the ARD flows are intermittent but can be stored and released as a continuous feed, an Aquafix™ unit may be appropriate. Contact and ordering information for Aquafix™ units can be found at http://www.aquafix.com/. See the following modules in AMD Treat©: Ponds, Pebble Lime, Flow Calculation Tools, and Acidity Calculator.

- Wheel-treater™ Treatment – Wheel-treater™ units are water-wheel powered caustic soda (sodium hydroxide solution)-dosing machines. These units require neither electricity nor constant monitoring. They function well under both continuous and intermittent flow conditions. Contact and ordering information for Wheel-treater™
units can be found at http://www.chemstream.com/. See the following modules in AMD Treat©: Ponds, Caustic Soda Flow Calculation Tools, and Acidity Calculator.

- Other Semi-Passive Units – Vendors offering semi-passive units that feed limestone or other acid-neutralizing reagents should be investigated on a case-by-case basis.

- Water Treatment Sampling Program – A water quality sampling program should be initiated as soon as the retention pond receives water. Pond influent and pond water samples should be collected and analyzed for the parameters listed on the Advance Sampling Suite. The pond influent sample should be collected upstream of the pond and the semi-passive unit. The pond water sample should be collected from the surface of the pond near the pond discharge point. If the ARD flow is driven by precipitation events, samples should be collected after significant precipitation events (rainfall > 1 inch in 24 hours). A sampling quality control plan should be developed in accordance with TDEC regulations to ensure a successful sampling program.

- Semi-Passive Reagent Feed Rate Adjustment – The target pH for pond water should be 8 or less, depending on the pH of the receiving stream. Increasing the pH to this level should remove a significant portion of metals. Based on the pH levels measured in the pond water, the lime feed of the Aquafix™ unit or the caustic soda feed of the Wheel-treater unit should be adjusted to provide the target pH level.

- Constituents of Concern and Reporting – Sampling results should be reported to TDOT and TDEC on a quarterly basis and after the completion of construction. Based on the sampling results, a list of contaminants of concern should be developed upon which to base future sampling efforts.

Long-term Passive Treatment Implementation

After the short-term semi-passive treatment system is in place, the long-term PT implementation phase begins and consists of the design and construction of a suitable PT system to address mitigation of ARD at the site. After an appropriate PT system has been constructed and commissioned, the operation of the semi-passive unit can be suspended. However, retaining the semi-passive unit on site in standby status is recommended for at least six months. The APR Guideline provides decision criteria for three different types of PT systems as listed below. Some types of PTS may require National Pollutant Discharge Elimination System (NPDES) Permits. Long-term passive treatment measures addressed in the APR Guideline follow.

- Analyze water quality data from the Short-Term Semi-Passive Treatment phase.
- If the site water has a pH < 5 or if any metals concentrations exceed the TDEC water quality criteria, long-term PT will be required. The site conditions and water quality will dictate which PT system (PTS) is appropriate among the options listed below.

1) PTS I – Settling Pond, Open Limestone Channel (OLC)
2) PTS II– Settling Pond, Surface Flow Wetland (SFW)
3) PTS III – OLC, Setting Pond, two Sulfate-Reducing Bioreactors (SRBRs), and SFW.

Detailed descriptions of these systems, sizing criteria, and installation guidance are provided in the APR Guidance document. A decision tree diagram for choosing the most appropriate PTS is shown on Figure 8. Sulfate reducing bioreactors are discussed in more detail in Gusek (2002).

![Decision Tree for Selection of Long Term Passive Treatment System (PTS)](image)

Figure 8 – Decision Tree for Selection of Long Term Passive Treatment System (PTS)

**POST-CONSTRUCTION MONITORING**

If P-APR/APR materials are identified during the course of the project, then the guideline indicates that Post-Construction monitoring should be performed for a minimum of two years following construction to ensure that mitigation and design measures are working effectively. If a PT system is constructed, monitoring should be performed as long as the system is in operation. If adverse impacts from APR disturbance/exposure develop, they would most likely be detected in surface water, runoff, or groundwater associated with the project. Sampling of rock in the Post-Construction phase is impractical relative to water sampling.
Monitoring Locations

The guideline recommends that any area of construction that contains P-APR/APR materials should be monitored. Monitored areas include, but are not limited to: road cuts fill zones, constructed or exposed embankments, and blended fill areas associated with APR; structures designed for encapsulation, mitigation, or remediation of P-APR/APR; and PT systems. In order to monitor these areas, designated sampling points should be established to capture groundwater, seepage, and runoff from these areas. Surface water sampling points should include provisions for flow rate measurement, if this data requirement is triggered. Monitoring locations should be established in a site-specific monitoring plan to monitor areas associated with APR materials.

The sample locations should be accounted for during the Pre-Construction design phase to ensure that the sampling sites will provide representative samples of water leaving the site. If impacts are noted down-gradient, appropriate up-gradient samples should be collected.

Monitoring Period

The monitoring period should be established in a site-specific monitoring plan that accounts for the specifics of each project. It is recommended that water sampling should be performed on a quarterly basis for the first year following construction, or in accordance with permitting, and semi-annually until one year after vegetation is established on cut faces, graded areas, slopes, and embankments; however, this frequency may be varied based on site conditions and professional judgment. If no indication of ARD generation is shown in this time, sampling may be discontinued. Background groundwater should be sampled on the same frequency as down-gradient waters.

If no indication of ARD generation is observed during these monitoring periods, sampling may be discontinued. If indications of ARD are observed, sampling should be increased to bi-monthly in order to evaluate the ARD generation. PT systems should be monitored on a quarterly basis for the first year following construction and on a semi-annual basis thereafter. Treatment systems should be monitored as long as they are in operation. If a PT system is regulated by a NPDES permit, the permit will specify the monitoring frequency.

Monitoring Suite

The analysis suites for post-construction monitoring are the same as those presented in the water testing methods section of the APR Guideline. Two sampling suites are specified there: if ARD is not present, the analysis suite should include an “abbreviated” set of parameters. If ARD is known to exist or if a PT system is in operation, an extended sampling suite is recommended. The sampling suites can be modified based on site conditions and professional judgment.
CLOSING REMARKS

TDOT’s APR-Guideline represents the collaboration of many individuals who generated the document itself and an extended list of engineers and scientists who researched ARD, its prevention and mitigation, and shared their data in publications and on the internet. While the APR Guideline is a living document that should improve over time provided that the findings of others can also be incorporated, it still allows professional judgment to override prescriptive controls.

REFERENCES


TDOT. 1990. State of Tennessee Special Provision Regarding Acid Producing Materials, revised 5/30/03. Location: Tennessee Department of Transportation.

ROCK CREEK CROSSING PART II
LEGAL ISSUES PART I

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ABSTRACT

Rock Creek Crossing (RCC) is a housing project sliced out of the side of a mountain in northern New Jersey. The first phase of the RCC slope stability project saga was presented at the 57th HGS. A number of the suspect areas discussed lay behind two three-story condominium structures and a parking lot. Potential slope failure modes mapped included the ravelling of glacial materials atop the slope as well as wedge and plane failures along bedrock discontinuities (foliation, faulting and fracturing).

Subsequent to the remediation work in this area, an adjacent landowner (Applicant) proposed to construct three large homes with associated roadways, utilities, septic systems and recharge basins above that slope. A review of the development plans prepared by the engineering firm headed by the Applicant indicated that the property encompassed the slopes behind three RCC facilities (Buildings B and C, as well as the intervening parking area).

Subsequent discussions, and meetings with, as well as sworn testimony from, various professionals for the adjacent project indicated little knowledge of rock slope engineering and ground water movement within a fractured rock medium. During the progress of the hearings, it also began to appear that the existing slope on the other side of the proposed development that was thought to be part of a third project was also on the Applicant’s property. This slope, along the entrance road to a large housing development, was also mapped and evaluated by the authors. Despite taking the developer’s professionals on guided tours of the two slopes of interest, their apparent lack of understanding of potential rock mechanics failures and ground water flow out of the slopes strongly contributed to the local Planning Board denying the developer’s application.

INTRODUCTION

At the 2006 HGS in Breckinridge, we presented the first installment of this tale of rock slope woe. The site, as developed, has many unstable bedrock slopes with a number of structures, parking areas, recreation facilities and the primary entrance road located in harm’s way. That paper presented the geology of the site (see Figure 1), a brief history of the development (Figure 2) and some examples of slope failures (Figures 3 and 4). Figures 5 and 6 show examples of some slopes that threaten property.
Based upon a triage evaluation using the slope safety factor versus the hazard to inhabitants and the available funding, remediation consisting of scaling, bolting and grouting was performed in the fall and winter of 2005 and 2006. This work covered about a third of the mapped rock slopes. A series of boulder “retaining walls”, which presented an additional hazard to entrance road users, was not addressed during this work except to note signs of falling rock.

With the satisfactory mitigation of immediate danger to the residents and, to some extent, their property, the homeowner’s association entered into further discussion with their law firm. The collection of property owners, banks, investors and professionals involved in the project was apparently a jumbled maze. Establishing the responsibility for the various phases of design, construction and inspection added to the concerns regarding shoddy construction and unfinished work.
At this time (2006), the many boulder retaining walls throughout the site were finally addressed. The boulder walls in the Clubhouse area were relatively well-constructed (Figure 7). The boulder walls above the main entrance road appeared to be placed indiscriminately rather than as a designed retaining structure (Figure 8). A portion of the entrance road was built upon a boulder retaining wall with the level of craftsmanship exhibited by the boulder walls near the clubhouse. However, other portions of the roadway support structure were constructed over a man-made boulder field.

Thus, the complete set of geotechnical concerns resulting from the work of a variety of prospective developers, engineers, architects and constructors are:

a. Boulders and cobbles ravelling from the surficial tills at the top of slopes and cuts affected by precipitation and the resultant loss of the sandy soils downslope.

b. Wedge and plane failures within the faulted, jointed and weathered metamorphic rock slopes that resulted in a risk to people and damage to property.

c. Poorly constructed boulder walls along the entrance road with the potential for failure above and downslope of the roadway.

All in all, a complex of geotechnical concerns exist that are not amenable to evaluation and remediation without incurring significant costs.

TIME LINE - ROCK CREEK CROSSING

- 172 townhouse/condominium units moderately priced (for NJ) at $150,000 to $400,000.
- Final design and layout (1995 to ~2000).
- Township approval (~2001 to 2002).
- Geotechnical firm hired to look at “rock face overhang”. Virtually every rock face and numerous boulder walls are suspect (2004).
- Remediation (2005 to 2006)

Figure 2 - Time line of notable events at Rock Creek Crossing.
THE NEIGHBORS

Sometime after the initial rock slope remediation phase of the work was performed, a developer that owned the adjacent, uphill property presented a plan to construct three large homes above Buildings B and C. The location of their property is shown on Figure 1. The remediated area behind Building C is shown on Figures 9 and 10.

From a geotechnical standpoint, the planned construction atop these slopes could result in;

1. Loading on the slope from the planned structures (likely minor),
2. Additional precipitation flowing over the RCC slope and through fractures, shear zones and foliation within the bedrock reducing near-term and long-term stability, and
3. Potential effects from construction blasting on the RCC slope and nearby structures from vibrations and fly rock.
Hence, the RCC Condominium Association became concerned with the planned construction and hired another arm (land-use law) of the legal firm already engaged in locating the perpetrators of the original slope stability concerns (together with the other shoddy construction practices). The group’s purpose was to either stop construction of the three planned units above RCC, or alter the design and construction enough to eliminate the potential for future slope failures and minimize blasting effects. The concerns were legitimate and there did not appear to be a vendetta against the Applicant.

Just prior to the Planning Board meeting for the new project, a set of site development plans were provided to RCC. While reviewing the plans the next day, we noted that their adjacent property apparently included much of the slope behind a heavily used parking area and Building C (see Figure 11). Initially espousing nothing but goodwill towards RCC, the Applicant hired an “experienced geotechnical engineer” that he had previously worked with to assure that the proposed development would not effect RCC in a detrimental manner. The pre-meeting discussion seemed amiable and we presumed that it would be a cooperative effort to solve the geotechnical concerns for RCC.
There were a series of meetings at the site with the Applicant (a Professional Engineer) and his consultant (also a Professional Engineer) and the Authors’ firm. Guided tours of the various slopes and walls were given, including another roadway slope just north of the Applicant’s property that the authors’ had mapped for another client (see Figure 1). Subsequently, we became aware that this nearby roadway cut slope was also likely part of the Applicant’s property. Similar fractures occurred along these slopes.

As a result of this inspection and a series of shallow test pits (3 to 6½ feet in depth) on the RCC side of the uphill site, their geotechnical consultant issued a series of reports (unsigned) that basically said the slopes were sound, no water would issue from the slopes in question because all the precipitation that fell on their site would percolate straight down, and blasting...
would not be a problem as all of the planned excavations (basements, utility lines and recharge pits) were shallow.

Photos such as Figures 12 and 13 did not sway him from his professional opinion as to the lack of water flowing from the slopes after precipitation.

The Planning Board turned down the application after six months of hearings.

THE CURRENT PROJECT

While still a geotechnical project, the scope of work has broadened. No longer a straight forward evaluation of rock slope and boulder retaining wall stability, a review of the plans for the adjacent property to lessen the threat from the RCC slopes and concerns for remediating someone else’s property; it now involves providing techno-legal assistance to a large law firm.

To complete the 2007 and early 2008 portion of this morality tale, the discovery portion of the RCC lawsuit showed that the Applicant’s engineering firm was also the lead civil engineer for the main construction phase of the RCC project. That firm hired the same geotechnical consultant that they originally used to evaluate the RCC slope and foundation stability concerns. In their 1995-1996 reports, both the engineering
firm and the geotechnical consultant recognized the instability of the slopes behind Buildings B and C; however, it appears that little was done toward mitigating the suspect areas.

It has been a most interesting, but convoluted project so far, technically, ethically and legally, and it is not yet over.
NEW MEXICO’S RAIL RUNNER EXPRESS PHASE II:
GEOLeGIC OVERVIEW AND
GEOTECHNICAL EVALUATION OF THE MANCOS SHALE

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ABSTRACT

New Mexico’s Rail Runner Express is a commuter rail line that after completion of Phase II will connect the city of Belen, Albuquerque, and the state capital-Santa Fe. To make the connection from Bernalillo to Santa Fe a notable distance of 17 miles of new track is currently under construction. To meet the project completion target of December 2008 a design-build delivery method is being utilized.

The Rail Runner project resides in North-Central New Mexico, an active and diverse geologic setting with a rich mining history. The project site is in the Rio Grande Rift, passes through the Cerros del Rio volcanic field, and immediately adjacent to Cerrillos Hills, a mined out laccolithic intrusion. The preliminary geotechnical investigation encountered geologic units that include Cretaceous shales, basalt lava flows, Quaternary basin fills, and recent alluvium.

The new track alignment requires 7 new bridges, 5 concrete boxes over/under pass structures, and numerous retaining walls to be constructed. Two of the concrete box structures bring the rail in and out of the I-25 highway median. A noteworthy portion of the alignment is where the new track will climb the La Bajada escarpment through Waldo Canyon, this area was avoided by historic rail alignments and requires construction of deep cuts and fills. In this area the Mancos Shale is prevalent and is known to be a highly degradable. Due to concerns with the quality of the Mancos Shale as a “soil” or “rock” material preliminary design considerations disallowed its use as a fill material.

PROJECT OVERVIEW

New Mexico’s Commuter Rail Project, the Rail Runner Express, under control of the Mid-Region Council of Governors (MRCOG) and the New Mexico Department of Transportation (NMDOT) is being extended from Bernalillo to Santa Fe, NM. The extended line will follow the existing Burlington Northern Santa Fe (BNSF) line between Sandoval City and Santa Domingo Pueblo and then follow an entirely new alignment up La Bajada Hill into Santa Fe. Seventeen miles of new track begins at the BNSF split near the base of La Bajada Hill, enters the median of interstate 25 (I-25) just north of the La Bajada rest area, and ends at the a tie in with the existing SFS rail line near the I-25 St Francis Avenue interchange. The December 2008 target date for revenue operations is requiring a fast track, design-build project delivery method.
GEOLOGIC DATA AND SUBSURFACE DATA REVIEW

Regional Geology

The project area for the Rail Runner Express Phase II Extension is located in north-central New Mexico and resides within the Rio Grande Rift. According to published literature, the Rio Grande Rift began to form when tension developed along two roughly parallel fault zones approximately 30 million years ago (mya) during the Oligocene Epoch. The fault zones dropped the earth’s crust down as much as 30,000 feet creating a chain of north-south trending basins. Volcanic, alluvial, and colluvial materials from the neighboring highlands have since been filling the rift valley. The project area is located at the southern end of the Espanola Basin.

Other significant geologic features near the project site are the Sangre de Cristo Mountains, the Southernmost tip of the Rocky Mountains, uplifted from the late Cretaceous through the Eocene; the Cerrillos Hills, a laccolithic, igneous complex emplaced during the Oligocene (~32 mya); the Cerros del Rio volcanic field formed during the Tertiary (~2.7 mya); and the most recent feature, the La Bajada Fault Zone, that developed during the Quaternary Period (<1.6 mya).

Geologic Summary of the New Alignment

The new alignment from the base of La Bajada Hill to the SFS connection for this phase of the Rail Runner will encounter three geologic corridors. They include: the southern end of the alignment, where the track diverges from the BNSF line north to Waldo Canyon Road, Waldo Canyon, and the top of the canyon across the mesa the SFS connection. Across these areas the preliminary geotechnical investigation encountered geologic units ranging from Cretaceous shales, to Quaternary basin fills, and recent alluvium and artificial fill. Below is a brief summary and description of the specific units encountered.

BNSF Split to Waldo Canyon Road

A review of the geologic maps of the Madrid and Tetilla Peak quadrangles indicates that the Cretaceous aged Mancos Shale (Km) formation exists within the project limits at the southern end of the alignment (Maynard, et al., 2001; Sawyer et al., 2002). The Mancos Shale is exposed where the new track alignment diverges to the north from the existing BNSF rail line and along the sidewalls of Waldo Canyon for approximately two-thirds its length. The geologic maps show that Pediment gravels (Qp), Quaternary Sheetwash deposits (Qsw), and Ancha Formation (QTa) are exposed at the ground surface and overlie the Niobrara Member of the Mancos Shale (Kmn) in this area.
A significant feature at the BNSF split is an unmapped pile of spoils from a cut through the Mancos Shale during the 1960s realignment of the BNSF line (Figure 1). The spoils pile is approximately 30 to 40 feet in height. Also, observed at this location is a relatively thick sandstone unit within the Mancos Shale and an andesitic igneous dyke. In the literature, the sandstone unit is referred to as a 336 foot thick sandstone lentil and the dyke is estimated to be 25 to 50 feet thick.

The pediment and sheetwash deposits were generally be described as being composed of wind blown sand and reworked sand and gravel from other nearby sources. The Niobrara Member is described as sandy marine shale with concretions up to 2 feet in diameter and sandstone interbeds ranging from 2 to 20 feet thick. Additionally, numerous igneous dykes are mapped within this area.

Figure 1: Photos of the BNSF take off point prior and during construction.

Waldo Canyon Road to Straight Street

The alignment climbs through Waldo Canyon at one point crossing from the east to the west side of the canyon on approximately 50 feet of fill above the canyon floor (Figure 2). The canyon sidewalls are composed of Mancos Shale overlain by the Ancha Formation, the upper most unit of the Santa Fe Group. The Ancha Formation is described as moderately consolidated and caliche cemented, moderately to well stratified, pebble to cobble conglomerate and pebbly to cobbly sandstone with scattered boulders and muddy sandstone interbeds. The matrix is described as fine to very coarse grained, very poorly sorted sandstone and gravel.
Figure 2: Photos of Waldo Canyon prior and during construction.

Straight Street to SFS Connection

The Tetilla Peak Geologic Map and the Generalized Geologic Map of the Southern Espanola Basin show that the remainder of the alignment from Straight Street at the top of Waldo Canyon to its connection with the SFS rail line crosses the Upper Santa Fe Group (Sawyer et al., 2002; Read et al., 2004). The Upper Santa Fe Group at this location is comprised of the Tuerto Gravel and the Ancha Formation. The Tuerto Gravel is described as having the same characteristics of the Ancha Formation with the addition of abundant subrounded to subangular clasts of igneous rocks, primarily diorite, monzonite, and andesite porphyry derived from the Cerrillos Hills. The thickness of the Tuerto Gravel ranges from a few feet near hills to 150 to 200 feet in some drainages. Along the mesa top and before entering the median the alignment also runs through fringes of the Cuerbio basalt (Figure 3).

Figure 3: Photos of Waldo the Mesa Top and cut through the Cuerbio basalt.
DESIGN AND CONSTRUCTION OF SHALE EMBANKMENTS

Due to numerous, large-scale failures of shale highway embankments throughout the eastern United States during the 1970’s the Federal Highway Administration (FHWA) conducted a comprehensive research study to investigate the underlying problems and to develop appropriate remedies. The investigation culminated in a five volume report with a summary report titled FHWA-TS-80-219 Design and Construction of Compacted Shale Embankments: Summary. The findings of this research, as well as others, is that many shale embankment failures are caused by slaking deterioration of certain shales creating excessive settlement (1 to 3 feet) and the potential for slope stability failure. The most severe settlements were found to be due to:

1. Use of non-durable shales as rock fill that allows for water infiltration and slaking.
2. Mixing shale and overburden soils with harder rock preventing improper compaction.
3. Lack of benching and drainage of underlying slopes allowing water build-up at the embankment base.

To avoid these problems, in some instances, highway departments have taken a conservative approach of treating all shales as non-durable, soil-like materials compacting them in thin, 8-inch lifts. Therefore, many shale embankments have been over-designed with durable shales being placed as soil. However, in other cases the lack of reliable criteria and testing has caused non-durable shales to be under-designed and inadequately compacted.

The solution to the problem is determining which shales are durable enough to be placed as rock fill, in thick lifts, and which shales must be broken down and compacted as soil, in thin lifts. Researchers have indicated that with proper evaluation of the shale material and proper construction techniques, embankments can be less conservatively constructed and still be safe.

Classification of Shale Formations

The main concern for development of proper shale embankment construction is classification of the borrow material. Many classification systems have been developed to categorize the durability of shale materials in order to predict their long-term engineering behavior. Design of Shale Embankments, prepared by Kenneth Huber for the Virginia Department of Transportation, summarizes 16 different shale classification systems. The various systems utilize 17 different laboratory tests where the most common is some form of slaking durability test.

The main goal of durability testing is to categorize the major shale strata to be encountered as: soft-nondurable, hard-nondurable, or hard-durable. The nondurable materials are to be treated as soil-like and the durable as rock-like. The shale materials that are most problematic are intermediate shales, categorized as hard-nondurable. These materials to not break down readily during construction, are difficult to compact, and degrade with time causing excessive settlement.
GEOETECHNICAL CHARACTERIZATION OF THE MANCOS SHALE

Geotechnical Problems with Existing Mancos Shale Embankments

During the preliminary investigation discussions with representatives of the New Mexico Department of Transportation (NMDOT) revealed that embankments in New Mexico constructed of the Mancos Shale, as well as other shales, have resulted in settlement and stability issues. Based on the NMDOT’s considerable geotechnical problems with current highway embankments composed of shales, the construction of new embankments using Mancos Shale materials for the Rail Runner alignment was disallowed in the Phase II, design-build Request for Proposal documents under the direction of the NMDOT.

From personal communications with the NMDOT it was learned the primary reason for deeming Mancos Shale unsuitable for Rail Runner embankment construction is due to settlement and stability issues associated with the southbound lanes of I-25 at La Bajada Hill. This area is of very close proximity to the Rail Runner alignment. The NMDOT reports that the southbound lanes are currently settling excessively and early signs of slope movement is indicated by their observation of tension cracks. Based on review of 1953, 1954 construction plans for US-85, now known as I-25, the embankments can be estimated to be as much as 64 years old. Although the plans do not indicate what material was placed in the embankment, it is assumed by the NMDOT that the embankment was constructed as a rock fill from Mancos Shale bedrock. The quality control of the embankment construction is not known.

Geotechnical investigation by the NMDOT at the problem areas along I-25 indicates the failure mechanism of these lanes is due to excessive settlement due to degradation of rock fill into a soil material. During the installation of inclinometers, the borings encountered perched groundwater, soft clay of high moisture content and very low shear strength, and high void ratio.

Mancos Shale along the Rail Runner Alignment

The profile along the proposed rail alignment indicates that significant cuts and fills are required through the southern section where the Mancos Shale is located to achieve an acceptable grade. A large cut in the first 2200 feet from the BNSF take off resides directly adjacent to the largest fill section of the alignment (Table 1).
Table 1: Summary of preliminary cut and fill volumes for alignment and within project area where Mancos Shale will be encountered.

<table>
<thead>
<tr>
<th>Station</th>
<th>Proposed Earthwork (Cut-Fill)</th>
<th>Cut (cy)</th>
<th>Fill (cy)</th>
<th>Net (cy)</th>
<th>Notes</th>
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<tr>
<td>Entire Alignment</td>
<td></td>
<td>1,884,990</td>
<td>1,785,175</td>
<td>99,185</td>
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<tr>
<td>106+50</td>
<td>Soil-249,220 Rock-36,915</td>
<td>--</td>
<td>-286,135</td>
<td></td>
<td>Mancos Shale spoils pile, Residual Soil and Bedrock.</td>
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<tr>
<td>129+50</td>
<td></td>
<td>861,782</td>
<td>+861,782</td>
<td></td>
<td>Proposed Embankment fill up to 50 ft tall</td>
</tr>
</tbody>
</table>

NOTE: Volumes based on preliminary alignment, interpretation of the boring logs, field observations, and Bentley end-roads average end area volume calculations with soil and rock volume factors set at 1.0.

Table 2: Summary of the subsurface materials encountered at the boring locations from Station 107+87 to 215+17 and interpreted soil stratum.

<table>
<thead>
<tr>
<th>Test Boring Data</th>
<th>Stratum Thickness (ft)</th>
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</thead>
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<tr>
<td>Test Boring</td>
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<tr>
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<tr>
<td>2-16</td>
<td>215+17</td>
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</table>
Determining Soil versus Rock

To assess the viability of Mancos Shale materials for embankment construction it was imperative to determine if the material should be treated and act as a soil or rock fill. According to the NMDOT 2005 Interim Specification, Section 203.21, Classification of Materials, is determined by a Ripping Test or Seismic Test, not hollow-stem auger drilling methods. That is why for this investigation the SPT results were employed as a preliminary measure to delineate residual soil from bedrock. Top of bedrock was considered the to be the depth at which SPT refusal was first encountered; SPT refusal is defined as achieving 50 blows or more when driving a 2-inch O.D. sampling spoon 6-inches or less. Refusal in cemented soil materials is not considered bedrock.

Subsurface Conditions at the Southern End of the Alignment

During the preliminary investigation the Mancos Shale geologic formation was encountered at the location where the proposed Rail Runner alignment splits from the Burlington Northern-Santa Fe (BNSF) line through Waldo Canyon. Seventeen hollow stem auger test borings and the three CME continuous push borings were drilled along the proposed alignment where the Mancos Shale is expected to be encountered. The subsurface conditions were interpreted and the soil strata encountered were delineated as spoil, residual soil, sheetwash, alluvial soil, and bedrock. A summary of the subsurface stratum encountered and approximate thickness is included below as Table 2. The materials encountered were tested for grain size distribution, Atterberg limits, natural moisture content, in-place density, moisture-density relationships (modified Proctor), shear strength (direct shear), and collapse/swell. All of the spoil and residual soil is assumed to be a derivative of the Mancos Shale.

Of significant geotechnical concern was what to do with the cut soils and the stability of the cut slopes through the large spoil pile located adjacent to the existing BNSF rail cut. Photos and cross section of the spoil pile and BNSF rail cut are included as Figure 2 and Figure 5. The spoil was described as dark gray to olive, clayey sand with gravel and sandy lean clay. The classification of the spoils samples ranged from lean clay, sandy lean clay, sandy silt, to clayey sand. The PI of these samples in three cases was 13 to 18, and three others were 20 or 21. In one instance, the sample was non plastic (NP).
Figure 5: Cross Section at the BNSF split, existing alignment is at approximately +150, centerline of spoils pile is at approximately -60, natural ground surface is at approximately elevation 5660.

Sheetwash or alluvial soil underlying the spoil pile was more difficult to identify in the borings, but can be identified based on a decrease in n-values and more homogenous silty sand appearance. Samples considered to be alluvial soil were generally classified according to the USCS as sandy silt or sandy clay, and silty sand. The alluvial soil was generally non-plastic. The soil samples interpreted to be sheetwash were primarily classified as sandy clay or sandy silt and two samples clayey sand. These samples generally had plasticity indexes (PI) below 10. One notable exception is the sheetwash sample that was encountered below the BNSF spoils pile; this sample had a PI of 19.

Residual soil or decomposed shale is generally described as olive to dark gray or dark brown; and is described as lean clay, lean clay with sand, sandy lean clay, silty clay, silty sand, silty clay, and silt. The samples interpreted to be decomposed shale were very similar to the spoil samples. According to the USCS they were generally clayey sand, silty sand, or sandy lean clay. The PI of these samples ranged from 9 to 20 with an average value of 17. A notable difference between the spoil and residual soil is the average sand content of the classified samples. The sand content of the spoil was generally lower than the residual soil samples, approximately 28% versus 45%, respectively.

The bedrock cored within the relevant project limits is generally described as very weak to moderately strong, highly to moderately weathered, dark gray shale. One sample that was designated as bedrock according to a SPT refusal criteria of greater than 50 blows for six inches of penetration was also submitted for USCS classification. The sample from a depth of 7.5 feet below the ground surface (bgs) classified as sandy lean clay with gravel, and the PI was 14. Unconfined compressive strength tests were performed on two Mancos Shale core samples. The strengths were 452.7 psi at a depth of 54.5 feet bgs and 42.1 psi for a sample from 13.3 feet bgs.

Collapse-swell testing indicates that the Mancos Shale can swell up to 2.3% or collapse as much as 5.1%. The results of a direct shear test indicate that the angle of internal friction of Mancos Shale-residual soil is 40 degrees with an apparent cohesion of 150 psi. The residual friction angle was determined to be 35 degrees.
**Additional Mancos Shale Laboratory Testing**

Due to the NMDOT’s current problems with shale embankments the purpose of the additional lab work was to evaluate the Mancos Shale at the project site and determine the feasibility of using Mancos Shale materials for embankment construction. The goal was to develop appropriate design, risk, and construction mitigation conclusions and recommendations for the NMDOT to compare to any proposal from the DB teams.

The major concern with using the Mancos Shale within the project embankments is that the material will degrade over time after placement and compaction, and will result in excessive long-term settlement of the embankments. To evaluate the Mancos Shale materials for embankment construction two-cycle slake durability tests ASTM D4644) were performed and potential settlement of compacted Mancos Shale samples was evaluated by performing modified collapse tests.

The Mancos Shale materials tested for durability and settlement were spoils from the existing BNSF rail cut, decomposed shale-residual soil, and NQ sized rock core samples. The samples tested were generally from proposed cut areas, but based on the small amount of proposed cut through Mancos Shale bedrock and limited sample availability within this area, core samples from non-cut areas (the proposed Waldo Canyon Road overpass) were also tested to better characterize the general engineering properties of Mancos Shale bedrock.

A total of 25 two–cycle slake durability tests (ASTM D4644) were performed, consisting of nine relatively undisturbed CME continuous push samples and 16 on NQ bedrock core samples. To better define the behavior of the cut materials at the completion of each slake durability test, Atterberg limits and gradation analyses were performed on all of materials broken down from the slake durability testing. Atterberg limits tests were performed on the material passing the #10 mesh drum and gradation tests were performed on the material retained within the #10 mesh drum. This testing was subcontracted to Advanced Terra Testing, Inc.

**Modified Collapse Test Results**

In order to characterize the settlement behavior of embankments composed of Mancos Shale derived materials a modified collapse test were developed and performed to investigate potential degradation of shale placed and compacted as an embankment material. This modified collapse test was performed in order to estimate potential collapse of compacted samples due to loading, wetting, and drying cycles.

To estimate potential embankment settlement, collapse tests were performed on samples compacted to 95% of Maximum Dry Density and near the optimum moisture content. The samples were loaded incrementally to 2.5 tsf, approximately ¾ of the maximum embankment height of 37.5 feet; the samples were saturated for 24 hours, drained and left loaded for 144
hours; the samples were resaturated for 24 hours, then drained; collapse was measured one final time; after each loading, saturation and draining cycle, the sample height was recorded.

RESULTS

The main goal of durability testing is to categorize the major shale strata to be encountered as: soft-nondurable, hard-nondurable, or hard-durable. The nondurable materials are to be treated as soil-like and the durable as rock-like. As described previously, two-cycle slake durability tests were performed to classify the shale into the soil-like or rock-like category. The slake durability test results in the slake durability index, $I_d(2)$, and also a qualitative rating of the post test materials.

Atterberg limits and gradation of the post slake durability materials were performed to further characterize the weathered shale materials. The results are included on the next page in Table 3, Figure 6.

\[ I_d(2) = \text{slake durability index (second cycle)} \]
\[ I_d(2) = (W_F-C)/(B-C) \times 100 \]
\[ B = \text{dried specimen weight before the first cycle} \]
\[ W_F = \text{oven-dried specimen weight retained after the second cycle} \]
\[ C = \text{mass of drum} \]

Type I—Retained specimen remain virtually unchanged
Type II—Retained specimen consist of large and small fragments
Type IIII—Retained specimen is exclusively small fragments

As was suspected based on the field classification and USCS results all the materials considered Mancos Shale spoil and residual soil materials broke down completely during slake durability testing. The average $I_d(2)$ value for these samples is 7.2% and are rated Type III material. The sixteen bedrock core samples tested had varying durability ratings. Their $I_d(2)$ value ranged from 22.0 to 95.7% and were rated as Type I, II and III materials. The average $I_d(2)$ value was 67.1.
Figure 6: Photos showing the various states of the Mancos shale materials after the 2-Cycle Slake Durability test. Note Type I, II, and III shale rating.
<table>
<thead>
<tr>
<th>Test Boring</th>
<th>Approximate Sample Elevation (ft)</th>
<th>Approximate Sample Depth (ft bgs)</th>
<th>Samples</th>
<th>Sample Type</th>
<th>Two Cycle Slake Durability (ASTM D4644)</th>
<th>Shale Classification</th>
<th>Laboratory Tests</th>
<th>Mechanical Sieve</th>
<th>USCS Classification</th>
<th>FHWA Classification (Strohm, Bragg, and Ziegel, 1980)</th>
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<td>5641</td>
<td>15</td>
<td>31</td>
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<td>100.0</td>
<td>38.1 CL</td>
<td>35</td>
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Table 3: Summary of slake durability test results and associated shale classification.
Table 3: Summary of slake durability test results and associated shale classification.

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<tr>
<th>Test Boring</th>
<th>Approximate Sample Elevation (ft)</th>
<th>Approximate Sample Depth (ft bgs)</th>
<th>Sample Type</th>
<th>Origin</th>
<th>Slake Durability Material Type</th>
<th>Material ID</th>
<th>Material ID</th>
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<th>Laboratory Tests</th>
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<th>Atterberg Limits (ASTM 4318)**</th>
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<td>2-3</td>
<td>5657</td>
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<td>Medium High</td>
<td>35.7</td>
<td>2.6</td>
<td>Not enough Material</td>
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</tbody>
</table>

Notes: **Bolded** sample depths indicate samples tested from proposed cut locations.

- **SAMPLE TYPE**: Bag = sample collected from auger cuttings, CONT = continuous push sample, CORE = NQ core sample collected using air rotary with down hole water for dust control
- **ORIGIN**: S = Sheetwash, F = Fill, R = Residual Soil, B = Bedrock
- **USCS Classification of Post Slake Material Passing #10**
- **Atterberg limits tests to be performed on the material passing the #10 mesh drum and gradation tests to be performed on the material retained within the #10 mesh drum**
Results Modified Collapse Test Results

The modified collapse tests were performed on composite samples of the spoils-residual soil samples were classified as clayey sand (SC) according the USCS. The plasticity of the spoils composite sample was 18, the residual soil sample was 15. The Modified compaction tests indicate that the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of the samples are 125.9 pcf and 10.1% for the spoils composite sample and 133.8 pcf and 5.9% for the residual soil composite sample.

The estimated collapse of the shale samples compacted to MDD, loaded, and saturated ranged form 1.0 to 1.75%, three of the four samples showed 1.75% of collapse. There was no appreciable collapse reported upon draining and resaturating the samples after a time period of 144 hours.

DISCUSSION

The subsurface conditions were interpreted and the soil strata encountered were delineated as spoil, residual soil, sheetwash, alluvial soil, and bedrock. All of the spoil and residual soil is assumed to be a derivative of the Mancos Shale. Review of the test boring logs and proposed alignment reveal that most of the cut material will be spoil or residual soil and a very limited amount of bedrock will be encountered. Therefore, classification of the durability of shale bedrock along the rail runner alignment is less critical than suspected prior to this investigation. Regardless, the durability of the shale materials is discussed with respect to the using the FHWA 1978 and Gamble’s 1971 shale classification systems (Huber, 1997).

FHWA-Strom, Bragg, and Ziegler 1978 Shale Classification System

The FHWA shale classification system classifies shales based on three Id(2) categories, <60%, 60% to 90%, and >90%. It also uses a I, II, or III shale rating; and another qualitative hard or soft description. According to the FHWA system the current laboratory test results show that all of the cut soils are to be considered soil-like, non-durable Type III materials (Table 3 and Figure 7). The histogram presented as Figure 7 shows the variability of the Mancos Shale materials tested and that they are generally considered soil-like, non-durable, although some of the samples are rock like-durable and two are intermediate-hard, the most problematic shale type.

Based on the results of the slake durability testing of the Mancos Shale cut soils, they appear to be generally suitable for use as fill if properly placed, broken down, and compacted in thin lifts as a soil-like material. Rock-like material that does not break down readily should be wasted. Further, the post slake durability Atterberg limits and gradation analysis indicates that the materials are generally medium plasticity to non-plastic, where plasticity decreases with depth. The materials retained in the #10 mesh slake durability basket were generally medium to coarse gravel, indicating that moisture conditioning does not completely break down all shale materials.
The rock core samples from non-cut zones that were tested in order to generally characterize and evaluate the engineering properties of the Mancos Shale formation have much more interesting results. The “rock” core samples are very diverse ranging from soil like, non-durable to rock like, durable.

Due to the variability of the “rock” cores if the cuts were at different locations or deeper in the subsurface much more attention and evaluation would have to be taken in considering the use of the Mancos Shale materials for fill. This is illustrated in Figure 8 shows that the durability of the materials tested generally increases with depth which is the expected, but also varies laterally. The slake durability test results of the “rock” core samples tested from below the cut zone indicate that fresher, unweathered Mancos Shale can be considered intermediate hard, non-durable or rock like and durable according to the FHWA classification system.

**Gamble 1971 Shale Classification System**

Gamble’s system plots PI against slake durability Id(2), where each property has a qualitative range from very low to very high for Id(2) and low to high for PI. Gamble’s research indicates that low durability (Id(2)<60) and high plasticity (PI>25) shales have slope stability problems. Another researcher later suggested that high or very high durability (Id(2)>95) and low plasticity (PI<10) shales can be placed as rock fill. According to Gamble’s system none of the materials are considered susceptible to slope stability problems, although, some are marginal and a number

---

**Figure 7: Histogram of the FHWA shale classifications based on the 25 samples consisting of spoil, residual soil, of “bedrock”**.

No. of Occurrences, Total Sample 25

<table>
<thead>
<tr>
<th>Category</th>
<th>No. of Occurrences</th>
</tr>
</thead>
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<td>Soil, Non-Durable</td>
<td>18</td>
</tr>
<tr>
<td>Intermediate-Hard, Nondurable</td>
<td>10</td>
</tr>
<tr>
<td>Rocklike-Durable</td>
<td>6</td>
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</tbody>
</table>

FHWA Shale Classification

- Soillike, Non-Durable
- Intermediate-Hard, Nondurable
- Rocklike-Durable

---
of them have low-durability. Two of the samples tested have high enough durability to be placed as rock-fill, but both are below the cut zone.

**Summary of Mancos Shale Embankment Construction Alternatives**

Three potential alternatives were developed for mitigating problems associated with the use of Mancos Shale for the rail embankments that we believe are feasible based on this investigation. The alternatives were to encapsulate the shale with suitable material, blend the shale with suitable material, or waste the shale completely.

Ultimately the selected DB decided to completely waste the Mancos Shale. Our original recommendation was that if the NMDOT decides to allow Mancos Shale materials to be used as embankment material that the material be blended at no greater than 10% with suitable non-shale material. This blending recommendation assumes that material will be placed, blended with reclaimers to ensure complete breaking down of non-durable rock like materials, and proper moisture conditioning and compaction. The shale bedrock materials estimated to be approximately 37,000 cubic yards should be not be considered suitable for blending and shall still be wasted.
CONCLUSIONS

Since the original field and laboratory testing was performed, the shale spoil pile has been excavated, exposing a profile of this material. It is apparent from the various strata that the wasted materials were placed and compacted to some degree, and not simply dumped. In addition, the spoil pile has been sitting since the late 1960’s, for some forty years, and the shale materials do not exhibit much degradation. This is likely why the side slopes of the spoil pile are stable at or steeper than 1.5 horizontal to one vertical slopes. This is in contrast to the I-25 embankments, which are exhibiting stability problems. The major difference in the two is likely the drainage conditions. The spoil pile has positive drainage in all directions, while the I-25 embankment was constructed against a natural slope that has seeping groundwater present.

The results of this testing may also indicate what is occurring at the I-25 embankment. As is the case with materials tested as part of this study, the shale materials at I-25 likely ranged in durability and now after a period of 60 years the intermediate and rock-like materials are breaking down and causing noticeable settlement. Another potential reason for collapse of the I-25 embankment soils is the subsequent wetting and saturation of properly placed and compacted soil materials. The modified collapse test indicate that a 100 feet thick, well placed fill that saturates slowly over a period of time could show up to 1.75 feet of settlement.

REFERENCES


NMDOT. *New Mexico Department of Transportation 2005 Interim Specifications*, 2005.


Personal communications with Tom Brown, P.E. from the NMDOT Geotechnical Design Section and Bob Meyers, P.E. the State Materials Bureau Engineer.


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<td>Keaton, Romo</td>
<td>Subsurface Void Detection in Oklahoma Evaporite Terrestrial Photogrammetric Models for Virtual Structural Mapping Compared to Traditional Geologic Measurements</td>
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<td>Peter Scholle NM State Geologist</td>
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**Session 4 Barriers and Field Trip**

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**Session 5 MISC and Computer Applications**

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**Session 6 Case Histories and Other**

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**Symposium on Problem Soils and Surficial Deposits**
1:30 – 5:00 P.M. Tuesday May 6th, 2008 Sante Fe, New Mexico
Sponsored by TRB Section on Geology and Properties of Earth Materials

**AGENDA**

1:30 – 1:35 P.M. Introduction  
Vanessa Bateman, *Tennessee Department of Transportation*

1:35 – 2:05 P.M. Mechanical properties of carbonate-cemented sands of arid regions  
*Paola Bandini, New Mexico State University*

2:05 – 2:35 P.M. Characterization of expansive soils using simple index properties  
*Claudia Zapata, Arizona State University*

2:35 – 3:05 P.M. Characterization and construction with gypsum  
*John Lommler, AMEC Earth and Environmental*

3:05 – 3:25 P.M. break

3:25 – 3:45 P.M. Gypsum-rich soils: problems and remediation  
*Bob Henthorne and Carrier Denesha, Kansas DOT*

3:45 – 4:05 P.M. Granular trenches for roadway embankment foundation treatment  
*David Harwood and Lok M. Sharma, Terracon Consultants, Inc.*

4:05 – 4:25 P.M. Subsurface void detection in Oklahoma evaporite deposits using geophysical methods  
*Justin Rittgers, David Butler and Phil Sirles, Zonge Geosciences*

4:25 – 4:45 P.M. Problematic soils in Oklahoma  
*Butch Reidenbach and Jim Nevels, Jr., Oklahoma DOT*

4:45 – 5:05 P.M. Stabilization of the expansive red clay soils in Socorro County by using fly ash  
*Abibata Essilfie and Mehrdad Razavi, New Mexico Institute of Mining and Technology*
Presenters

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Mechanical Properties of Carbonate Cemented Sands of Arid Regions

Paola Bandini, Ph.D.

Sand deposits that have been cemented with calcium carbonate are common in semi-arid regions of the southern United States. This presentation describes the main features, origin and mechanical properties of these naturally occurring cemented sands. Results of a series of direct shear tests and oedometer tests performed on undisturbed specimens from three typical profiles of New Mexico are compared and discussed. Undisturbed specimens were tested air-dried, moist, and saturated to determine the effects of moisture on the shear strength and compressibility of these soils. Sand samples were also tested undisturbed and remolded (disturbed) to determine the contribution of cementation on the shear strength. It was found that the shear strength of these cemented sands was significantly affected by wetting and degree of cementation. Despite the presence of carbonate nodules in the specimens and the inherent variability of the sand, it was possible to observe the tendency of the soils to lose shear resistance and become compressible as moisture content increased. The peak shear stress of saturated specimens was approximately half of that measured for air-dried or slightly moist specimens. Unlike the dry or moist specimens, saturated specimens did not dilate during shearing. Oedometer test results also showed that soaking caused this soil to become more compressible compared with dry soil. The characteristic features of the macro and micro fabrics of these soils are also described.
CHARACTERIZATION OF AND CONSTRUCTION WITH GYPSUM

John Lommler, Ph.D., P.E

In southeastern New Mexico near White City and Carlsbad Caverns there are deposits of gypsum. These materials are not gypsum contaminated soils; rather, they appear to be quite pure gypsum. Construction of roadway subgrades on this material and construction of embankments built of this material were studied. Complexities of the chemical composition of this material were encountered. The primary "problem" was structural or chemical water content which was difficult to separate from geotechnical water content. For paving design purposes the stiffness or modulus of this material was required, but construction with this material required "density" and uniformity of fill placement measurements in addition to stiffness considerations.
GYPSUM-RICH SOILS: PROBLEMS AND REMEDIATION

Robert Henthorne, P.G. and Carrie Denesha, R.G.

The Kansas Department of Transportation is currently finishing up a comprehensive highway program. This program consisted of over 3.5 billion dollars in roadway improvements. Many of the proposed improvements were located in the gypsum rich region of the state.

This region is found in the central portion of Kansas and covers approximately 1/3 of the state. Bedrock in this area is from the Permian System. Kansas Permian System deposition is characterized as shallow land locked seas to sub-aerial exposures. This sequence has given rise to vast deposits of evaporites, mainly salt and gypsum with minor accumulations of anhydrite. Many of these deposits are currently being mined. Even though we may have some mine related problems, they are not the major concern when dealing with our evaporites. The Kansas Department of Transportation’s major concern is the reaction of minor amounts gypsum with our lime treated sub-grades. If undetected the chemical reaction can produce a new mineral up to 250 percent larger than the parent materials, thus creating major pavement failures.

We have problems with our soils as well as bedrock material. The presentation will discuss our geologic investigations procedures, testing, and remediation followed by case histories.
Design and construction of The New I-64 in St. Louis, Missouri presented numerous geotechnical challenges including loess and compressible alluvial foundation soils. One of the more challenging conditions encountered included planned 25 to 45-foot high embankments with mechanically stabilized walls to prevent fill slope encroachment. The embankments were underlain by 15 to 20 feet of compressible soils. The embankments were critical path components of the first phase of construction. Due to project constraints, including maintenance of existing traffic, space limitations and project schedule, the embankments required a relatively rapid construction. The short-term bearing capacity failure of the foundation soils and the anticipated settlement would not permit a rapid construction of the embankments. Some form of treatment of the foundation soils thus became necessary. Initial options considered included overexcavation and foundation soil replacement, stone columns and grout injection. The design-build team including the contractors, geotechnical engineers, and structural engineers collaborated on the available options as well as local resources.

The stone column option was further evaluated. The design of stone columns resulted in a relatively close spacing and would require a specialty contractor. After reviewing the stone column spacing requirements, the team explored the potential of using the local resources and implementing a series of trenches in the foundation soils backfilled with compacted 4-inch minus granular material. The design calculations of the trench behavior indicated improvements in the bearing capacity as well as reduced settlements, thereby permitting the objective of rapid construction of the embankments. The team selected the granular trench option.

The granular trenches were installed to penetrate the soft foundation soils to the full depth. The embankments have been completed as per design and have been performing satisfactorily. This paper presents the evolving processes that led to the selected unconventional approach, analysis and design of the foundation treatment measures.
SUBSURFACE VOID DETECTION IN OKLAHOMA EVAPORITE DEPOSITS USING GEOPHYSICAL METHODS

Justin Rittgers, David Butler and Phil Sirles

Surface sinks, distressed highway sections, voids and evaporite formations with variable weathering have complicated highway redesign in western Oklahoma. More than 46,000 linear feet of Direct Current (DC) Electrical Resistivity (ER) imaging data were collected, using the Dipole-Dipole technique, along Highways US-64 and US-412 in Major County near Woodward, Oklahoma. The main purpose of the survey was to identify and discriminate between sections of highway underlain by solid gypsum or gypsum containing voids (resistivity > 1000 ohm-meters) and sections containing combinations of claystone and weathered gypsum (resistivity <100 ohm-meters). Zonge Engineering and Research Organization’s ZETA system was evaluated as an effective tool in mapping subsurface geology.

Preliminary geophysical results were furnished to Terracon, Inc. and, in consultation with the Oklahoma Department of Transportation (ODOT), the locations for eighteen confirming borings were selected. Borehole data correlated very well with the resistivity models and allowed for the assignment of resistivity ranges to specific lithologies, this correlation became the basis of all geophysical data interpretation for the duration of the survey.

The results presented here show that DC ER offers an accurate and cost-effective approach to mapping lateral and vertical variations in material properties that can be directly associated with lithology. This can help alleviate the common issues confronted when making geologic interpretations based on limited and widely varying data from adjacent borings. Two useful generalizations can be drawn about this specific project area: 1) The highest values of resistivity more often correlate with gypsum hosting numerous smaller (0.5-1.5 feet diameter) voids than with large voids, and 2) Large sections of the surveyed area (several 1000’s of feet along US-412 and US-64 are underlain by clay, weathered gypsum and gypsum-clay as confirmed by the borings, and will not likely pose as many issues with regards to required mitigation efforts. In summary, the ER technique, as confirmed by borings, successfully separated the surveyed areas into sections underlain by claystone and weathered gypsum and into sections with potentially karst gypsum formations requiring different mitigation tactics.
Problematic Soils in Oklahoma

Vincent Reidenbach, Ph.D., P.E. and James B. Nevels, Jr.

The context for the Oklahoma Department of Transportation (ODOT) presentation is based on the ODOT geotechnical specifications for Roadway Design. The problem soils listed therein include the following: organic soils, dispersive clays, normally consolidated clays, expansive clays, collapsible soils, river or stream (meander loops, cutoffs, and oxbow lakes), and soils containing sulfates. Surficial deposits include the following: degradable and expansive shales, mine spoils, shales and siltstones containing gypsum, and karst features (gypsum and limestone).

Organic soils occur primarily in the southeast part of Oklahoma in Division 2. They are of limited extent but have traditionally been handled by undercut and removal. Dispersive clays are more of a problem in recent years because of reductions in some maintenance practices. They are usually found in the southeast and southwest and sporadically elsewhere but not in the northwest or panhandle areas of Oklahoma. Normally consolidated clays are not in them selves a particular geotechnical problem; however, their occurrence is. Typically we find these types of clay deposits in eastern Oklahoma, and they occur in thin layers or zones under over consolidated alluvial clay crusts. Expansive clays are an extensive and wide spread problem in Oklahoma. Regardless of nationally published maps indicating ranges limited of expansive soil deposits in Oklahoma, all expansive clay mineralogies are found. Natural occurring collapsible that have had significant past problems occur in central and western Oklahoma and are usually derived from silt deposits. They may or may not be partially cemented with calcium carbonate. River or stream (meander loops, cutoffs, and oxbow lakes are not in them selves but their occurrence is. Finally, mapped soils containing sulfates are of limited extent in western Oklahoma.

The most extensive surficial deposit in Oklahoma is shale. Shales occur as mudstones, clay shale, siltstone, and rock-like shale. Some shales are highly expansive, and they can be highly degradable. Mine spoils that are of major concern and significance are primarily located in Ottawa County in northwest Oklahoma. They are a nationally recognized environmental hazard from former lead and zinc mining. There are five shale and/or siltstone formations that contain gypsum which can have soluble sulfates. There significant karst features (sinkholes, caves, etc.) in gypsum and limestones formations, and they are primarily located western, south-central, and northeastern Oklahoma.
The production of fly ash from coal combustion has been increasing but only a negligible portion of the product is used. The environmental issues associated with fly ash disposal have made it necessary to find effective techniques in solving this problem. Fly ash has been widely used to modify the engineering characteristics of subgrade soils. This paper presents a study on the effectiveness of fly ash as an additive to improve the engineering properties of expansive red clay soils in Socorro County in New Mexico. An experimental study was performed to evaluate the effect of adding fly ash on plasticity, swell potential, compaction, and strength characteristics of the soil. Experimental results showed a significant decrease in the plasticity index as well as swell potential but an increase in unconfined compressive strength with increasing the fly ash content. Changes in moisture-dry density relationship resulted in higher optimum water content and lower maximum dry density of the soil.
Subsurface Void Detection in Oklahoma Evaporite Deposits Using Geophysical Methods

By Justin Rittgers*, David Butler and Phil Sirles, Zonge Geosciences, Inc.

Abstract

Surface sinks, distressed highway sections, voids and evaporite formations with variable weathering have complicated highway redesign in western Oklahoma. More than 46,000 linear feet of Direct Current (DC) Electrical Resistivity (ER) imaging data were collected, using the Dipole-Dipole technique, along Highways US-64 and US-412 in Major County near Woodward, Oklahoma. The main purpose of the survey was to identify and discriminate between sections of highway underlain by solid gypsum or gypsum containing voids (resistivity > 1000 ohm-meters) and sections containing combinations of claystone and weathered gypsum (resistivity <100 ohm-meters). Zonge Engineering and Research Organization’s ZETA system was evaluated as an effective tool in mapping subsurface geology.

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The results presented here show that DC ER offers an accurate and cost-effective approach to mapping lateral and vertical variations in material properties that can be directly associated with lithology. This can help alleviate the common issues confronted when making geologic interpretations based on limited and widely varying data from adjacent borings. Two useful generalizations can be drawn about this specific project area: 1) The highest values of resistivity more often correlate with gypsum hosting numerous smaller (0.5-1.5 feet diameter) voids than with large voids, and 2) Large sections of the surveyed area (several 1000’s of feet along US-412 and US-64) are underlain by clay, weathered gypsum and gypsum-clay as confirmed by the borings, and will not likely pose as many issues with regards to required mitigation efforts. In summary, the ER technique, as confirmed by borings, successfully separated the surveyed areas into sections underlain by claystone and weathered gypsum and into sections with potentially karst gypsum formations requiring different mitigation tactics.

Introduction

Highway-related construction maintenance is often complicated by subsidence, the presence of sinkholes and natural and man-made voids such as dissolution caves and abandoned mine shafts and addits (Sheets, 2004). Mitigation of these issues often involves timely and expensive drilling programs and use of indiscriminant engineering measures.

Zonge Geosciences, Inc. of Lakewood, CO. conducted extensive geophysical surveys along three sections of highway under subcontract to Terracon, Inc., and under cooperation of the Oklahoma Department of Transportation (ODOT). The survey encompassed three independent areas including two sections of US-412 and one section along US-64 in Woodward and Major Counties, Oklahoma. An initial survey covering a total of approximately 20,500 feet of data coverage was completed, and preliminary processing and interpretation was performed. These preliminary results were used to identify confirmation borehole locations for “ground-truthing” of the geophysical data and to identify areas warranting further investigation. Additional data was later collected, resulting in approximately 46,000 linear feet of data coverage.

The objective of the geophysical investigation was to map the vertical and lateral extent of the gypsum unit associated with these dissolution features to help ODOT mitigate possible adverse impacts on planned highway expansion projects. The near-surface
geology in the vicinity of US-412 and US-64 consists primarily of clays, claystone, sandstone, limestone and evaporate deposits (gypsum). The later is often extremely susceptible to dissolution from the infiltration of storm water and the movement of groundwater. There are numerous small dissolution features that can be seen outcropping on the sides of the existing highway, and multiple caves, including the well known Alabaster Caves, are located near the existing US-412.

The geophysical method referred to as Direct Current (DC) electrical resistivity (ER) was selected to map subsurface geology and identify the presence of gypsum and potential subsurface dissolution features. This method is commonly used for void detection, as the resistivity contrast between soil/rock and dissolution features is typically very large. The ER surveys along US-412 and US-64 were designed to provide data to depths of about 60 feet below ground surface along the approximate locations of the proposed new highway alignments.

Along the western section of US-412 (referred to here as section #1), the planned highway lanes are approximately 130 feet (40 meters) and 40 feet (13 meters) offset to the north of the existing road. Along the eastern section of US-412 (referred to here as section #2) the planned highway lanes are approximately 105 feet and 45 feet offset to the south of the existing highway (see Figure 1). The actual locations of the ER surveys were determined by accessibility and terrain, and were selected in consultation with Terracon and ODOT personnel. Terracon provided the locations and elevations of the
planned highway in Station-Offset-Elevation (SOE) file and hard copy formats. The ER surveys were referenced to survey markings on the existing road, and were measured by compass and chain methods relative to the centerline of the existing road.

The ER surveys along US-64 (referred to here as area #3) were conducted on a single transect located with a 25 foot offset along the south side of the existing road. The objective of the ER survey in this area was to determine if subsurface geologic features were causing observed roadway damage. The results of geophysical surveys in conjunction with geotechnical borings suggest that the road damage is likely due to clay-rich expandable soils, and it is believed there are no dissolution features in this area. The results of this portion of the investigation are not further discussed herein.

**Purpose and Scope**

As acidic rain and groundwater permeates through fractures and between layers, dissolution of gypsum results in widening fractures, weakened sections of highly weathered material, large caverns and sinkholes due to the removal of materials that previously supported overlying rock and soils. Large systems of caverns are known to exist in the vicinity of US-412, and this cave system traverses beneath US-412 in one known location where people have reported hearing traffic passing overhead.

This paper presents the results from an extensive geophysical investigation conducted for Terracon and ODOT along Oklahoma highways US-412 and US-64. This case study lends itself as an example of the effective application of 2-D DC resistivity for mapping of subsurface geo-electric structures related to lithology and detecting anomalous zones such as highly resistive features caused by the presence of vadose dissolution features and large caverns.

**Methods**

The geophysical technique utilized for this project’s electrical resistivity survey is referred to as the Double-dipole, or more commonly, the dipole-dipole technique (Telford et. al., 1976). In the dipole-dipole electrode configuration, a controlled electrical signal is transmitted into the ground via a grounded dipole consisting of two current electrodes (A and B). At varying distances from the midpoint of the current dipole, the electrical potential drop is measured and recorded at a different grounded dipole, called a “receiver” or “potential” dipole (M and N). This potential difference measured by the receiver dipole is due to the electric field created by the source current dipole. For this survey, an axial (or polar) dipole configuration was used, where the receiver dipole is inline with the transmitter dipole (Al’pin, 1950).

![General electrode configuration for the dipole-dipole DC resistivity technique.](image)

The signal is normally measured, digitized and recorded to the instrument’s internal memory or directly on an external drive or computer. Both the current and potential dipoles have two electrodes with constant spacing, referred to as the “a” spacing; and, the distance between the transmitting and receiving dipoles is varied by multiples of “a”. Here, “n” is normally an integer value between 1 and 6. For this survey an a-spacing of 20 feet was used.

The main material property of earth materials measured by electrical methods is resistivity (ρ), which is the reciprocal of conductivity (σ). Electrical resistivity is a quantitative measure of how difficult it is to send current through a material. The mechanisms that allow electric current flow include the movement of free electrons through a metallic lattice referred to as electronic conduction, the movement of ions through an aqueous solution referred to as electrolytic conduction, the movement of ions through a solid crystal lattice referred to as solid electrolytic conduction (Yungul, 1996). Displacement current is the last means of transferring charges, however, this phenomenon is only present in high-frequency time-varying situations and does not apply here (Telford, et. al., 1976).

Variations in subsurface porosity, fluid content, fluid chemistry, permeability and soil or rock type all affect resistivity measurements. Cultural features (i.e., man-made items) such as fencing, power lines, and pipelines can also significantly affect resistivity measurements if not properly insulated from the ground or adequately avoided.

From Ohm’s Law, the ratio of the measured potential drop across the receiver dipole (M and N) to the measured output current across the transmitter dipole (A and B), the method yields the apparent resistivity (ohm-meters) at a certain “point” below the array:
Figure 3: Sequence of data collection in a dipole-dipole ER survey, depicting the construction of a pseudo-section: (a) The first measurement and associated apparent resistivity value, (b) the first diagonal completed with the transmitter dipole at its first station, (c) the transmitter dipole advanced to the 7th transmitter location, and receiver dipole collecting additional soundings, and (d) the completed pseudo-section with the two dipoles at the end of the survey line.

\[ \rho_a = k(\Delta V/I) \]

Where \( \rho_a \) is the apparent resistivity (ohm-meters), \( k \) is the geometry factor (meters) which is equal to \( 2\pi a \) for this dipole-dipole array configuration, \( \Delta V \) is the measured potential drop across M and N electrodes (volts), and \( I \) is the measured output current (amps).

Apparent resistivity is an average value for the non-homogeneous volume sampled by each measurement, and does not necessarily represent the true resistivity of earth materials at a certain lateral location or depth (Abraham, et. al., 2004). This is the raw data to be modeled in order to obtain a true resistivity model of the earth below the dipoles.

As depicted in Figure 3, each measured and calculated apparent resistivity value is plotted at the center-point (or station) between the two dipoles and at a “depth” equal to the “n” value to create a pseudo-section. The pseudo-section is a generalized way to plot data coverage and quickly detect major anomalous readings prior to processing. The processing method employed to resolve final resistivity models is discussed further in the Data Processing section below.

**Instrumentation**

The instrumentation used to perform this geophysical survey is the Zonge Electrical Tomography Acquisition (ZETA) system produced by Zonge Engineering and Research Organization (ZERO). The ZETA system consists of 6 primary components: 1) a 24-volt main power supply to the power-booster, 2) the power-booster unit that outputs up to 400 volts to the transmitter, 3) the transmitter unit that outputs current to the multiplexor (MUX), 4) the MUX unit that coordinates the dipole-dipole array geometry over a 30-channel array, 5) the geophysical data processor (GDP) unit that sets transmitter parameters, controls the transmitter and directly records all essential data (transmitter output currents and receiver-dipole potentials) onto an internal hard-drive, and 6) a laptop computer with the ZETA200 program installed and running.

The ZETA200 program allows the user to set all desired parameters, and is used to synchronize and coordinate the GDP, transmitter and the MUX units. ZETA200 also utilizes a user-written schedule file that controls the MUX unit, allowing the user to use any number of arbitrary electrode geometries and perform a complete contact-resistance check for all active
channels prior to performing data collection. Figure 4 illustrates the physical configuration of this system.

The GDP allows data to be recorded on all available channels (multiple receiver dipoles) simultaneously, allowing for fast data acquisition of an entire diagonal in the pseudo-section prior to advancing the transmitter dipole and repeating. This allows the operator to quickly obtain full data coverage for a given spread before advancing along the survey line. Overlapping data coverage at the end and beginning of each spread ensures seamless depth coverage along a given survey line with multiple spreads.

For this survey, the transmitted signal was a 0.5 Hz time domain signal (50% duty cycles). This frequency is low enough to perform a DC ER survey while avoiding significant displacement current and SP effects due to polarized electrodes (Yungul, 1996). Eight cycles were stacked and averaged to comprise one measurement. All measurements were repeated at least one additional time to establish repeatability of data. Adverse affects from cultural features were minimized through proper placement of survey lines.

At each station, electrodes consisting of tin-coated copper grounding braids were buried approximately two-inches deep in the soil. Once a spread of 30 electrodes (290 feet per spread) was in place and connected, data were acquired. Relative elevations were recorded at every station (electrode) using a hand level and stadia-rod, and these elevations were converted to absolute elevations via tying to survey marks that were measured using a Trimble RTK GPS, normally with sub-decimeter accuracy.

Data Quality

For this survey, the receiver operator made multiple measurements of each data point while monitoring real-time standard-error values displayed on the screen of the receiver. During ZETA data acquisition, multiple waveforms are stacked and averaged to reduce random noise in the data blocks, and all data blocks are repeated at least twice to establish data repeatability. All individual blocks are recorded and saved digitally, along with standard error of the mean (SEM) values. The receiver operator monitors data quality in the field, and contact resistance issues are resolved and data acquisition is repeated if necessary. Data quality for this project ranged from fair to excellent with respect to standard error of the mean (SEM) and block repeatability for ZETA.

Some cultural features such as roads, signs, pipelines and fencing were encountered during the course of the survey; however, these did not have significant affects on the data quality. One survey line in area #1 required the removal of metal posts and barbed wire fencing by ODOT personnel to avoid undesired current-paths between electrodes from forming through the fence. The dipole-dipole method is most sensitive to regions directly between the two dipoles; however, there are occasionally strong contrasts in resistivity near, but not directly under, the survey line that affect measurements.

Data Processing

Processing for electrical resistivity data acquired using the ZETA system was performed using software developed by Zonge Engineering and Research Organization (ZERO). The flow chart sequence shown in Figure 5 outlines the main steps in reducing and processing the ER data collected for this project. These programs are made available for commercial use and are sold on worldwide basis with Zonge equipment systems. The data are processed through the SHRED program initially to pre-process raw field data, then TDAVG and TS2DIP to computationally model (in 2D) the resistivity data. Two-dimensional plots are
Program names are CAPITALIZED
File names are Boxed

SHRED
TDAVG
AVG-GDAT
.GDAT-file
ZPLOT
GEOSOFT PROGRAMS
Other files read or written:
.MDE-file
.LOG-file
.Xnn-files

Figure 5: ZETA data processing flow using SHRED, TDAVG, AVG-GDAT, ZPLOT and GEOSOFT Programs

generated using either standard (over-the-counter) GEOSOFT tools (for example) or through Zonge’s ZPLOT package.

Briefly, smooth-model inversion mathematically “back-calculates” (or “inverts”) from the measured data to determine a likely distribution of true resistivity values. Comparison of the observed field data and the calculated pseudo-section plots is a useful method for evaluating how well the mathematical model fits the observed data. The results of the smooth-model inversion are intentionally gradational, rather than showing abrupt, “blocky” changes in the subsurface. The inversion results should not be considered a unique solution, and some ambiguity remains in any mathematical representation of the data. Confidence in any interpretation increases with corroborating information.

Results

Preliminary results of the modeled data were provided to Terracon/ODOT in order to determine boring locations to assist in the interpretation of the geophysical data. In the preliminary interpretations, an arbitrary color scale was selected for the resistivity sections, as shown in Figures 6 and 7. This color scale was selected to highlight high resistivities (>500 ohm-m) that might be associated with karst features. Based on these results, and in consultation with Terracon/ODOT, 18 borings were completed. The locations and results of these borings are tabulated in Table 1 below, and their locations are also annotated on the final model cross-sections presented in Appendix A.

After the 18 borings were completed, correlations were made between lithology and calculated resistivity values in the final models. This allowed for the assignment of a range of resistivity values expected for a given lithology. The three main materials encountered in the 18 borings are; 1) clay and clay/weathered-gypsum mixtures, 2) gypsum, and 3) large voids and highly fractured gypsum with many small voids.

Once ground-truthing of the models was complete, the ranges of model resistivities within a given material were plotted, and are presented on Figure 8. Each of these three material types correspond to a range of resistivities, and a unique color was assigned for each range on the color scale shown in Figure 6. This color scale was used for all final models, and it became the foundation of all interpretations for this project. The letters “C” and “G” and “V” were annotated on the color scales of all final models to indicate the interpreted lithologies.

Survey station numbers, offsets from the center of the existing road and proposed highway elevations are annotated on the final figures, horizontal axes indicate station location (feet) and the vertical axes indicate elevation (feet).
Figure 7: Preliminary results presented with an arbitrary color scale and no interpretations for resistivity lines along US412 highway section #1. The figure includes the proposed highway SOE plotted on the figure as a red-dashed line. The figure also includes relevant field observations, confirmed boring locations and proposed boring locations annotated along the top of the sections. The red regions indicate highly resistive areas most likely containing gypsum or karst geology.
Figure 8: Correlation of final model resistivity ranges with borings used in the final interpretation of along Highway US-412.
<table>
<thead>
<tr>
<th>No.</th>
<th>Location</th>
<th>Purpose</th>
<th>Completion Depth (feet)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>54+500 (150' North)</td>
<td>Geotech</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>54+810 (150' North)</td>
<td>Void and Geotech</td>
<td>25</td>
<td>Void Detected 11 to 13', then small voids to 14' bgs</td>
</tr>
<tr>
<td>B-3</td>
<td>55+017 (150' North)</td>
<td>Void and Geotech</td>
<td>38</td>
<td>Vooids Detected between 22-24, 25-26 and 26-35' bgs</td>
</tr>
<tr>
<td>B-5</td>
<td>55+210 (85' North)</td>
<td>Void and Geotech</td>
<td>40</td>
<td>Small Voids between 15 and 22' bgs, water loss @ 15.6' bgs</td>
</tr>
<tr>
<td>B-6</td>
<td>55+290 (20' North)</td>
<td>Void and Geotech</td>
<td>40</td>
<td>Small Voids @ about 22.5' bgs, water loss @ 9.5' bgs</td>
</tr>
<tr>
<td>B-7</td>
<td>55+450 (146' North)</td>
<td>Void and Geotech</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>B-8</td>
<td>55+720 (150' North)</td>
<td>Void and Geotech</td>
<td>36</td>
<td>Small Void @ 9.5' bgs, water loss @ 9.5' bgs</td>
</tr>
<tr>
<td>B-10</td>
<td>56+450 (75' North)</td>
<td>Void and Geotech</td>
<td>30</td>
<td>Water loss @ 14' bgs</td>
</tr>
<tr>
<td>B-9</td>
<td>56+450 (150' North)</td>
<td>Void and Geotech</td>
<td>42</td>
<td>Water loss @ 4.5' bgs</td>
</tr>
<tr>
<td>B-11</td>
<td>56+520 (85' North)</td>
<td>Void and Geotech</td>
<td>35</td>
<td>Voids from 32-33.5' bgs</td>
</tr>
<tr>
<td>B-12</td>
<td>56+680 (150' North)</td>
<td>Void and Geotech</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>B-13</td>
<td>2046+050 (150' South)</td>
<td>Void and Geotech</td>
<td>36</td>
<td>Water loss @ 12.7' bgs</td>
</tr>
<tr>
<td>B-14</td>
<td>2047+000 (85' South)</td>
<td>Void and Geotech</td>
<td>34</td>
<td>Small Voids 19-21' bgs, water loss @ 21' bgs</td>
</tr>
<tr>
<td>B-15</td>
<td>2050+047 (150' South)</td>
<td>Void and Geotech</td>
<td>13.5</td>
<td>Abandoned Due to Void @ about 13.5' bgs</td>
</tr>
<tr>
<td>B-16</td>
<td>2050+050 (150' South)</td>
<td>Void and Geotech</td>
<td>64</td>
<td></td>
</tr>
<tr>
<td>B-17</td>
<td>2079+000 (150' South)</td>
<td>Void and Geotech</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>B-18</td>
<td>618+000 (21' North)</td>
<td>Geotech</td>
<td>50</td>
<td></td>
</tr>
</tbody>
</table>

Table 1: Boring locations along with completion-depth and some data that were used for ground-truthing of resistivity results.

**Discussion**

There is excellent correlation between resistivity distributions on adjacent transects, revealing linear trends of anomalously high resistivity zones. These correlations are interpreted as linear problematic sections of gypsum trending across the existing road and proposed project area. There is also excellent correlation between the modeling results and outcropping caves and other observations noted in the field.

As seen in Table 1, and the final resistivity modeling results presented in Appendix A, the survey has mapped areas with concentrated anomalies, identifying sections of the highway redesign project area where problematic materials may be encountered at depth. The west section of Highway 412 contains the majority of anomalously high resistivity distributions while large expanses along the east section #2 have relatively low resistivity values. There is some overlap in resistivity values for each lithology type discussed here; however, the general distributions of material types have been mapped successfully.

The results presented here show that DC ER offers an accurate and cost-effective approach to mapping lateral and vertical variations in material properties that can be directly associated with lithology. This helps alleviate common geotechnical issues confronted when making geologic interpretations based on limited and widely varying data from adjacent borings.

More specifically, this project has shown that DC ER can be used to map geology that likely contains subsurface voids. The results presented here demonstrate the usefulness of DC resistivity profiling in helping to effectively mitigate and prevent future highway-related issues related to the presence of and formation of new dissolution features and subsurface voids.
References

Appendix A

Terracon/ODOT
Highway 412 West Section
West End to Station 1500 (Line 1 reference)
2D Inversion Model Resistivity (ohm-m)
Figure 6-3
Terracon/ODOT
Highway 412 West Section
Station 4500 to 6000 (Line 1 reference)
2D Inversion Model Resistivity (ohm-m)
Figure 6-6
Terracon/ODOT
Highway US-64
2D Inversion Model Resistivity (ohm-m)
Figure 6-16