58th HIGHWAY GEOLOGY SYMPOSIUM

POCONO MANOR, PENNSYLVANIA, OCTOBER 16-18, 2007

PROCEEDINGS

HOSTED BY
THE PENNSYLVANIA DEPARTMENT OF TRANSPORTATION AND
THE PENNSYLVANIA GEOLOGIC SURVEY

TABLE OF CONTENTS

Proceedings – Paper Abstracts

58th ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

Pocono Manor, Pennsylvania

October 15- 18, 2007

Welcome to the 58th Annual Highway Geology Symposium. The Pennsylvania Department of Transportation and the Pennsylvania Geologic Survey extends a cordial welcome back to the Keystone State.

Pennsylvania hosted the 38th Highway Geology Symposium in 1987, in Pittsburgh, highlighting the western area of the state. We now have the pleasure of acquainting you with the northeast area, the Pocono Mountains. The Pocono Mountains are divided into five regions, Delaware River, Upper Delaware River, Lake, Mountain, and the Lehigh River Gorge. The field trip will predominately explore the Lehigh River Gorge Region; however we encourage you to remain in the area and discover the incredible natural beauty of the other regions as well. The entire state holds the allure of geologic wonders and fascinating history so time permitting we hope you will discover these delights for yourself.

The local organizing committee has generated what we hope will be an informative, memorable and stimulating symposium. In keeping with the symposium's style, this year's authors will present practical and innovative papers, an interesting and delightful field trip is offered, and exhibitors with a variety of products and services available for discussion. Again, welcome and enjoy this year's Symposium in the Keystone State.

The 58th Annual Highway Geology Symposium Host Committee

HIGHWAY GEOLOGY SYMPOSIUM
HISTORY, ORGANIZATION AND FUNCTION

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

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

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

Since the initial meeting, 57 consecutive annual meetings have been held in 32 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, West Virginia, Ohio, Maryland, North Carolina, Pennsylvania, Georgia, Florida and Tennessee serving as host state.

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

List of Highway Geology Symposium Meetings

No.	Year	HGS Location	No.	Year	HGS Location
1st	1950	Richmond, VA	2nd	1951	Richmond, VA
3rd	1952	Lexington, VA	4th	1953	Charleston, W VA
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX

15th	**1964**	**Rolla, MO**	**16th**	**1965**	**Lexington, KY**
17th	**1966**	**Ames, IA**	**18th**	**1967**	**Lafayette, IN**
19th	**1968**	**Morgantown, WV**	**20th**	**1969**	**Urbana, IL**
21st	**1970**	**Lawrence, KS**	**22nd**	**1971**	**Norman, OK**
23rd	**1972**	**Old Point Comfort, VA**	**24th**	**1973**	**Sheridan, WY**
25th	**1974**	**Raleigh, NC**	**26th**	**1975**	**Coeur d'Alene, ID**
27th	**1976**	**Orlando, FL**	**28th**	**1977**	**Rapid City, SD**
29th	**1978**	**Annapolis, MD**	**30th**	**1979**	**Portland, OR**
31st	**1980**	**Austin, TX**	**32nd**	**1981**	**Gatlinburg, TN**
33rd	**1982**	**Vail, CO**	**34th**	**1983**	**Stone Mountain, GA**
35th	**1984**	**San Jose, CA**	**36th**	**1985**	**Clarksville, IN**
37th	**1986**	**Helena, MT**	**38th**	**1987**	**Pittsburgh, PA**
39th	**1988**	**Park City, UT**	**40th**	**1989**	**Birmingham, AL**
41st	**1990**	**Albuquerque, NM**	**42nd**	**1991**	**Albany, NY**
43rd	**1992**	**Fayetteville, AR**	**44th**	**1993**	**Tampa, FL**
45th	**1994**	**Portland, OR**	**46th**	**1995**	**Charleston, WV**
47th	**1996**	**Cody, WY**	**48th**	**1997**	**Knoxville, TN**
49th	**1998**	**Prescott, AZ**	**50th**	**1999**	**Roanoke, VA**
51st	**2000**	**Seattle, WA**	**52nd**	**2001**	**Cumberland, MD**
53rd	**2002**	**San Luis Obispo, CA**	**54th**	**2003**	**Burlington, VT**
55th	**2004**	**Kansas City, MO**	**56th**	**2005**	**Wilmington, NC**
57th	**2006**	**Breckenridge, CO**	**58th**	**2007**	**Pocono Manor, PA**

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

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

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

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

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

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

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

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

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

58th ANNUAL
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POCONO MANOR PENNSYLVANIA
October 15-18, 2007

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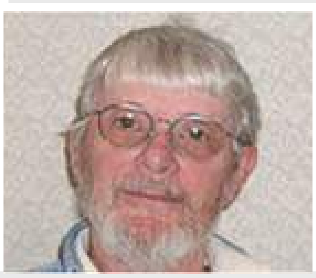

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***Deceased**

HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

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

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

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58th HIGHWAY GEOLOGY

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PAPERS

Landslides I have known in Northeast Pennsylvania and Northwest New Jersey

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Disclaimer

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ABSTRACT

Geologic mapping in the area of Delaware Water Gap and Lehigh Gap in northeast Pennsylvania and northwest New Jersey by the USGS and the Pennsylvania and New Jersey Geological Surveys, mostly at a scale of 1:24000, has shown the distribution and nature of Paleozoic sedimentary bedrock and Quaternary surficial deposits, and documented the orientation and extent of rock fractures. These data are useful for the understanding of landslide development in the area.

I have recognized four types of landslides, soil slips on glaciated-polished bedrock surfaces, debris flows in glacial till, debris slide in cemented glacial gravels, and rockfalls that originated along fractures that parallel roads. In the Delaware Water Gap National recreation Area a soil slip on the northwest-facing slope of Kittatinny Mountain occurred near Sambo Island on the Delaware River in October, 1995. Here, moderately dipping beds of the Bloomsburg Red Beds are covered with a thin veneer of soil and glacial till. A combination of heavy rain and lack of anchoring of the soil by tree roots that did not penetrate the glacially polished bedding surface resulted in the landslide. Similar geologic conditions (moderately steep bedding, glaciated-polished bedrock surfaces and shallow soil) can be used to determine areas of potential future landsliding. A debris flow in rain-saturated glacial till developed along a road cut in 1996 in a steep bank along a narrow tributary valley in the Pocono Plateau near Bushkill, Pa. Glacial till is common throughout the area and landsliding may be anticipated in areas where the bases of steep slopes are excavated. Sliding of cemented glacial gravels occurred along a steep stream-cut bank along a road paralleling Tom Brook. Two rockfalls occurred in New Jersey where the Old Mine Road parallels longitudinal joints near the crest of an anticline just north of Delaware Water Gap and on the northwest limb of an anticline opposite Tocks Island. These fractures are common in bedrock throughout the park. Stress measurements by the US Army Corps of Engineers on northwest-dipping bedding-plane faults in New Jersey near Tocks Island suggest that there is the potential for massive failure of rock above these structures should they be exposed by construction. Twenty five miles to the southwest at Lehigh Gap movement along cross joints in sandstone and conglomerate have created a rockfall hazard that required mitigation. This location is the site of stop 5 of this years filed trip. The run out from several of the joint-related landslides at Delaware Water Gap and Lehigh Gap have been mitigated with gabions.

INTRODUCTION

The landslides discussed in this report occur within folded sedimentary rocks along the Blue Mountain section of the Valley and Ridge physiographic province and the Pocono Plateau of northeastern Pennsylvania and northwestern New Jersey (fig. 1). Those in the Delaware Water Gap National Recreation Area have been affected by Wisconsinan glaciation where previous weathering materials have been largely stripped away. The landslide at Lehigh Gap is beyond the limit of the most recent glaciations and is in an area of deep weathering.

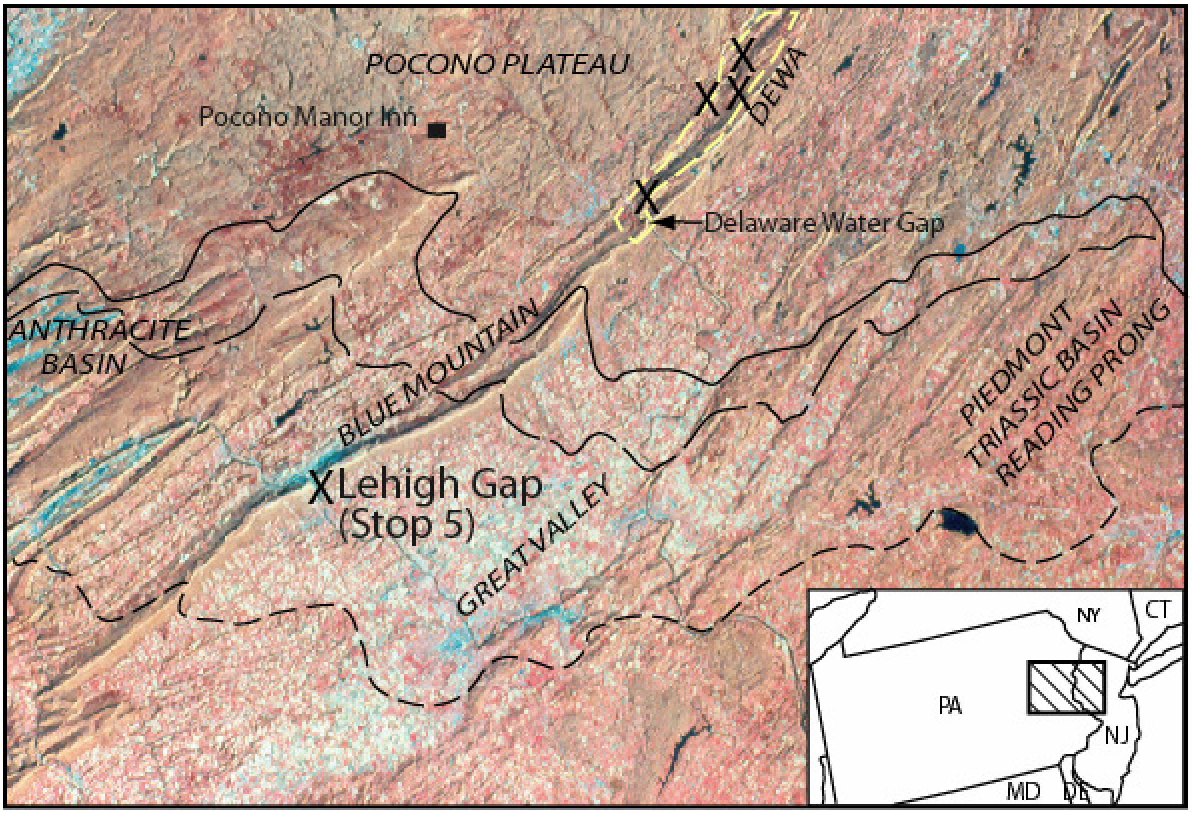

Figure 1. Satellite image showing physiographic provinces of eastern Pennsylvania and location of landslides (X) discussed in this report . The limits of major glacial advances is shown by solid line (Wisconsinan); long-dashed line (Illinoian), and short-dashed line (pre-Illinoian). DEWA is the boundary of Delaware Water Gap National Recreation area.

The Delaware Water Gap National Recreation Area (DEWA) lies within the heart of the Boston-Washington urban corridor. It is the largest National Park facility in the northeastern United States and is the sixth most heavily visited National Park Service facility in the country with about 4 million visitors yearly. It is about 40 miles long and includes a scenic and mostly undeveloped stretch of the free-flowing Delaware River between Port Jervis, New York, and the Delaware Water Gap in New Jersey and Pennsylvania. It occupies parts of eleven 7.5-minute quadrangles (Figure 2). The area offers a variety of recreational opportunities and opportunities to study the biologic diversity, cultural history, and geologic development of this part of the Appalachians.

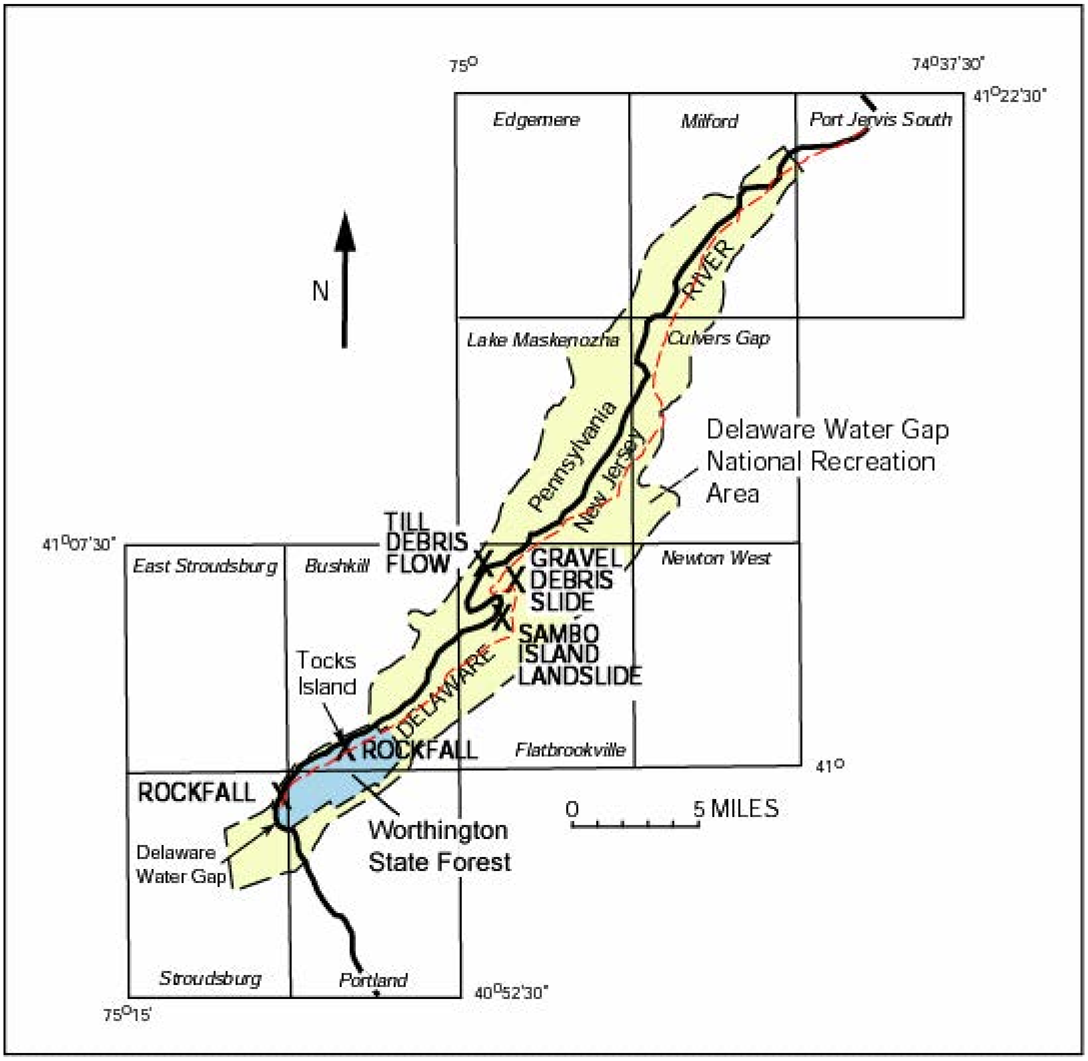

Figure 2. Index map showing landslides and topographic map coverage in the Delaware Water Gap National recreation Area and adjoining Worthington State Forest, Pennsylvania-New Jersey. Red-dahsed line is the Old Mine Road.

The park spans two major physiographic provinces, the Valley and Ridge and Appalachian Plateau, the latter locally known as the Pocono Plateau (fig. 3). The rocks within the park area aggregate more than 8,000 feet in thickness and range in age from the Middle Ordovician to the Upper Devonian, approximately 440 to 380 million years ago. The structure in the rocks and the resulting landscape features trend northeastward. These rocks are varied in lithology and structure and can be subdivided into four units. Table 1 summarizes the lithologic and structural characteristics of the component formations within these units. Wisconsinan glacial drift is the most widespread unit at the surface in the area.

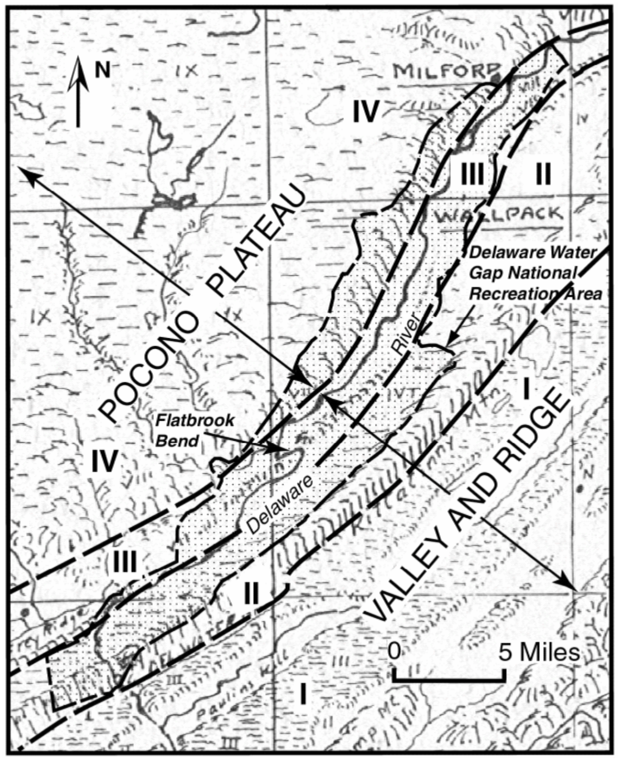

Figure 3. The Delaware Water Gap National Recreation Area spans two physiographic provinces, the Pocono Plateau and the Valley and Ridge. The boundary is marked by gently dipping rocks of the Marcellus Shale and Mahantango Formation (Unit IV) of Middle Devonian age along cliffs northwest of the Delaware River, separated from Ordovician through Middle Devonian rocks (units III– I) to the southeast that are more complexly folded.

Unit I comprises slate and sandstone of the Martinsburg Formation of Middle Ordovician age, forming rolling hills that slope down to the southeast towards the Paulins Kill which is underlain by carbonate rocks of Cambrian and Ordovician age. No significant landslides are present in the Martinsburg in the immediate area. Unit II comprises resistant sandstone and conglomerate of the Shawangunk Formation which holds up Kittatinny Mountain, rising to altitudes above 1600 feet, with less resistant shale, siltstone, and sandstone of the Bloomsburg Red Beds forming the northwest slopes. Slope failures include soil slips and debris flows in the Bloomsburg and rockfalls in both formations. Bedding-plane faults in the Bloomsburg have the potential for causing mass movement of overlying rock if construction daylights this zone of weakness.

Table 1. Generalized description of rock units in the Delaware Water Gap National Recreation Area, Pennsylvania and New jersey

Geologic Unit	Age	Formation	Thickness (feet)	Description
4 Flat-lying to gently dipping rocks of the Pocono Plateau.	Devonian	Catskill Formation	500+	Sandstone and lesser shale; forms uplands in Pocono Plateau
		Mahantango Formation	2,000	Siltstone and silty shale; forms steep slopes northwest of the Delaware River
		Marcellus Shale	800	Shale; underlies the Delaware River northeast of Flatbrook Bend
3 Asymmetric generally overturned folds with wavelengths averaging about 1,000 feet.		Buttermilk Falls Limestone	275	Limestone, calcareous shale, and chert
		Schoharie Formation	100	Calcareous, argillaceous siltstone
		Esopus Formation	180	Shaly siltstone and shale
		Oriskany Group	100	Sandstone, calcareous shale and siltstone, chert
		Helderberg Group	300	Limestone, calcareous shale, calcareous sandstone
		Rondout Formation	30	Dolomite, calcareous shale, limestone
	Silurian	Decker Formation	80	Arenaceous limestone, calcareous sandstone, dolomite
		Bossardville Limestone	100	Limestone
		Poxono Island Formation	500	Dolomite, shale, limestone; underlies Delaware River southwest of Flatbrook Bend
2 Asymmetric upright and overturned folds with wavelengths averaging about 1 mile		Bloomsburg Red Beds	1,500	Red sandstone, siltstone, shale; forms northwest slope of Kittatinny Mountain
		Shawangunk Formation	1,400	Sandstone and conglomerate; holds up Kittatinny Mountain
1 Assymetric similar overturned folds with wavelengths about 2,000 feet; pervasive slaty cleavage	Ordovician	Martinsburg Formation	1,000+	Slate and greywacke; forms southeast slope of Kittatinny Mountain

The course of the Delaware River in the northeast section of the Recreation Area is on cherty limestone of the Buttermilk Falls (Onondaga) of Unit III and shales of the Marcellus of Unit IV. After cutting through unit III at the S-shaped Flatbrook Bend, the river flows on the weak shale and carbonate rock of the Poxono Island Formation (Unit III) and upper part of the Bloomsburg Red Beds (Unit II). Other than large talus blocks, probably of Pleistocene age, there are no major landslides in unit III, although a Wisconsinan gravel debris slide, cemented by calcium carbonate derived from the underlying limestones of this unit, is found above this unit. Unit IV lies northwest of the Delaware River and forms the base, steep slopes, and tableland of the Pocono Plateau. The basal shales of the unit, which form the escarpment and steep slopes along US 209, are over steepened because of Wisconsinan glacial erosion and are constantly spalling off, forming shale-chip rubble at the base of the cliffs. One debris flow is reported here in glacial till overlying this unit.

Four distinct types of landslides were identified: (1) a soil slip-debris flow on moderately dipping, glacially polished bedrock surfaces in the Bloomsburg Red Beds, (2) a debris flow in till on a steep slope, (3) a debris slide on glacial gravels, and (4) rock falls or slips generated along joints along the Old Mine Road (fig. 2). The variety of slope instabilities within the park boundary poses a risk to people and property. Mitigation costs for these landslides in the park have been an estimated $150,000.

SAMBO ISLAND LANDSLIDE

During October 20-21, 1995, heavy rain fell in the Delaware Water Gap area generating a landslide on the anticlinal ridge south of Sambo Island along the Delaware River (fig. 4). The ridge is composed of red siltstone, shale and sandstone of the Bloomsburg Red Beds (unit II, Figure 3). The northwest slope of the ridge is a dip slope with bedding dipping gently at the crest at about 850 feet altitude, increasing to nearly 40^{o} farther down the slope (fig. 5). The bedding surface exposed at the landslide site has been polished and striated by glacial erosion (fig. 6). Scattered outcrops of bedrock dot the ridge, but in most places bedrock is covered by three types of surficial materials: (1) soil composed of shale chips and organic matter; (2) large blocks of sandstone, and (3) till. The glacial erosion and till are the result of action by the last glacier that departed from this section of the Delaware River less than 20,000 years ago. The soil and sandstone debris formed subsequent to glacial retreat.

Figure 4. Sambo Island landslide, developed on glacially polished bedrock bedding surfaces, extends for more than 600 feet on a slope that averages about 36^o. It originated as a soil slip, entrained additional soil, glacial till, and trees downslope, and spread out into a debris fan at the bottom after encountering a rock rib formed by a stratigraphically higher bedding surface. Note the leveed channel on the lower right.

The soil cover in the slide area averages about two feet thick (fig. 7), ranging up to 8 feet thick. It consists mostly of rock chips, weathered from Bloomsburg shale and siltstone, averaging about 1 inch in length, and mixed with fine organic matter. Where the underlying bedrock is sandstone, angular blocks as much as 20 feet long are produced by spalling off from the bedrock surface and by transportation down slope. Slow mass wasting by creep is evidenced by many trees that are bowed at their bases. Production of the weathered fragments is facilitated by cleavage, bedding parting and joints in the shale-siltstone, and by bedding parting and joints in the sandstone. Most trees in the immediate slide area have an accumulation of rock chips plastered on their up-hill side to a height of two feet, indicating fairly recent downward movement of these materials.

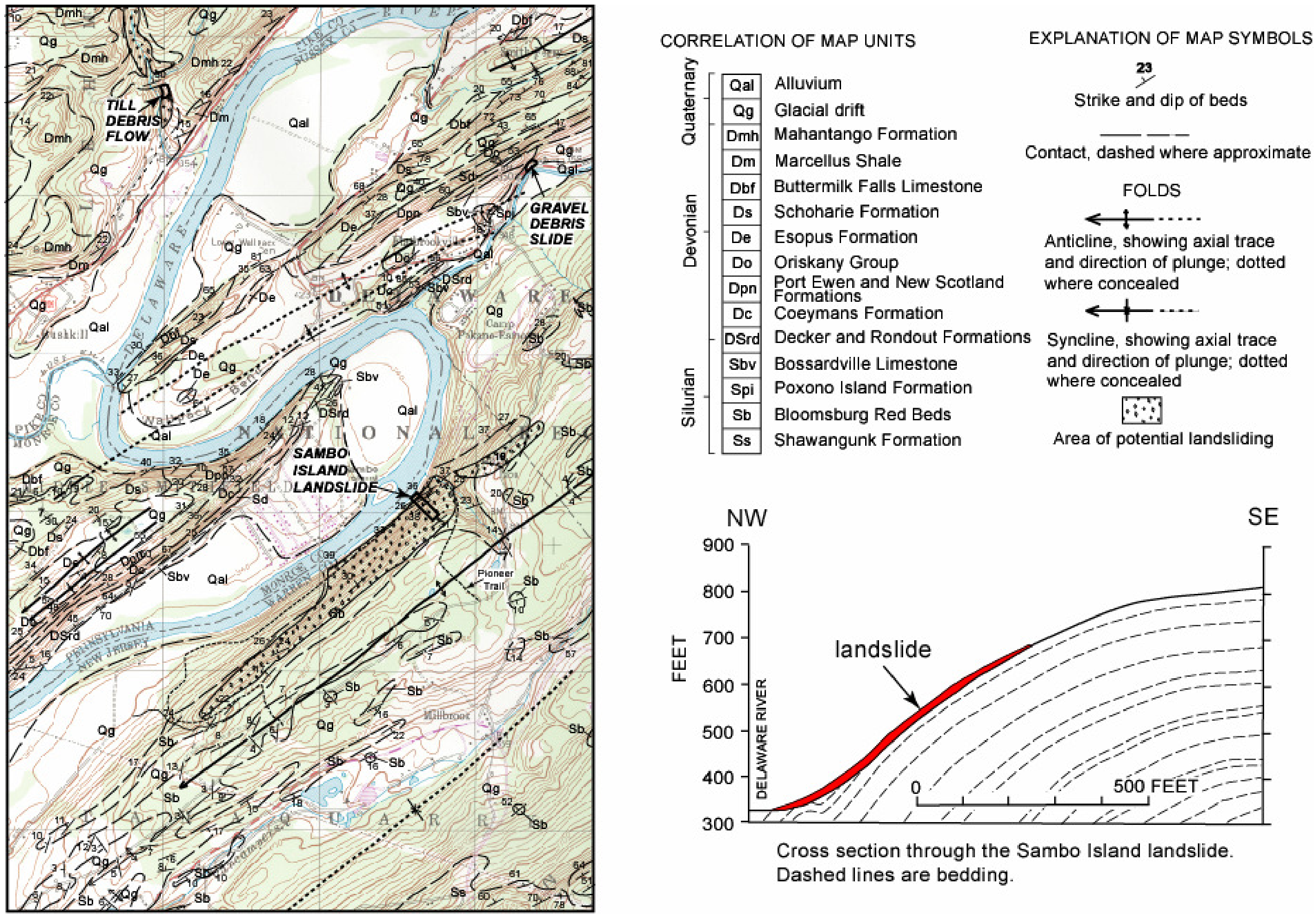

Figure 5. Geologic map of the Wallpack Bend area, Flatbrookville quadrangle, New Jersey-Pennsylvania, showing location of Sambo Island landslide (with a cross section) and till debris flow. The area of potential landsliding south of the Delaware River is delimited.

Figure 6. Glacially polished Bloomsburg bedding plane in the initial failure area of the Sambo Island landslide. Bedding slopes 32°NW. Glacial striae trend S. 40° W., parallel to the trend of the ridge.

Figure 7. Pressure ridge (arrow), about 3 feet high, showing downslope slip of soil adjacent to the initial failure area of the Sambo Island landslide. Note the lack of penetration of tree roots into the polished bedrock.

Glacial till (fig. 8) is present in patches along the entire slope of the mountain and is more than 20 feet thick in places where it blankets large areas of bedrock. Hummocky topography along much of the lower slope of the mountain is suggestive of old landslide deposits. At the slide area the till is thickest at the base of the slope where it is nearly 5 feet thick (fig. 9). The material is a moderate brown (5YR4/4) clay-silt till with subangular to rounded cobbles and boulders as much as four feet long in the slide area. The clasts include a variety of rock types, including quartzite and sandstone from the Shawangunk Formation, Bloomsburg Red Beds, and Catskill Formation; and limestone and chert from various stratigraphic units. Some igneous boulders are syenite derived from an intrusion at Beemerville, NJ, 17 miles to the northeast. A thin veneer of soil containing red shale chips overlies the till in many places. For several hundreds of feet downstream, riffles in the Delaware River are due to large erratic till boulders that may have been emplaced by older landslides. Some of these boulders are as much as six feet long.

Figure 8. About 4.5 feet of glacial till overlies Bloomsburg bedrock just below the initial failure area. A thin veneer of soil containing red shale chips overlies the till.

Figure 9. Four-foot deep channel developed in debris fan in glacial till at the base of the Sambo Island landslide.

Between 11 PM, October 20, 1995, and 3 PM, October 21, a total of 3.3 inches of rain fell near the site of the landslide (fig. 10), during two periods separated by about 6 hours. Immediately following the rain the river's discharge increased nearly ten-fold, peaking at about 27,000 cfs (fig. 11). During the 16 hours, there were two separate periods of heavy rain which saturated the thin soil making it unstable and causing it to slide on the polished bedrock surface. The contrast between the loose porous soil and the hard smooth bedrock surface was favorable for such failure because of a buildup of pore-water pressure over the less permeable bedrock surface. During recent periods of rainy weather, water flows along the smooth bedrock from under the soil cover.

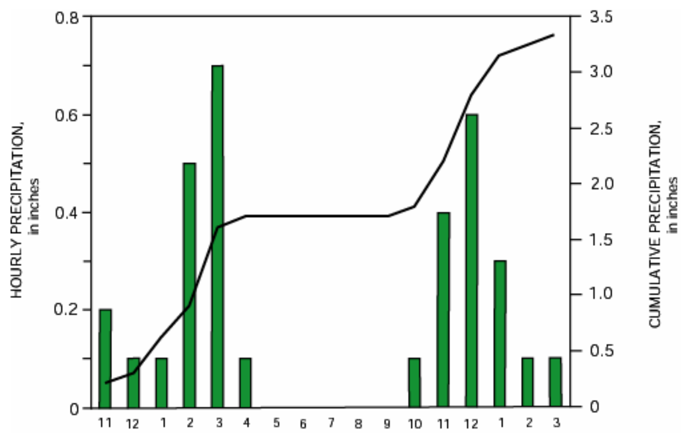

Figure 10. Bar graph showing hourly precipitation and curve showing cumulative precipitation between 11 PM, October 20, 1995, and 3 PM, October 21, 1995, at dingmans Ferry, 11 miles northeast of the Sambo Island Landslide.

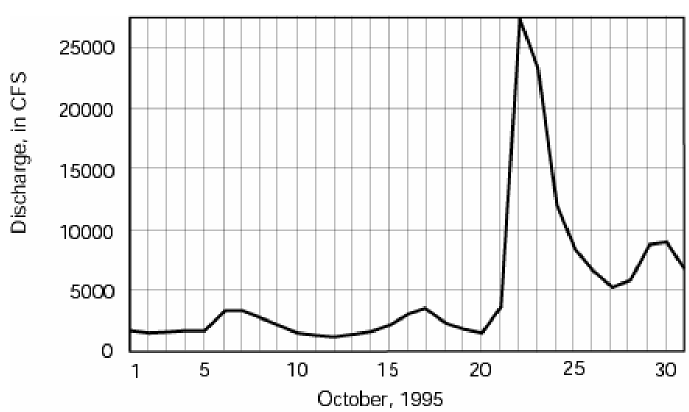

Figure 11. Streamflow for the Delaware River at Tocks Island during October, 1995.

The slide initiated in about 1.5 feet-thick, moderate-reddish-brown (10R4/4) shale- and siltstone-chip soil and organic matter at an altitude of 680 feet, encompassing an oblong area 37 feet wide and 90 feet long (fig. 12). The bedrock here forms a dipslope (34^{o}), is composed of red siltstone, and is partly burrowed and mudcracked. The surface is highly polished and striated (fig. 6), glacial movement towards S40^{o} W is indicated by the striae. At the bottom of the initiating area, the slide narrows to 28 feet at a three-foot-high pile of a debris-flow levee on the bedding surface, below which there is about 4.5 feet of moderate brown (5YR4/4) clay-silt glacial till overlying the bedrock (fig. 8). The soil thickens to about three feet in a pressure ridge (fig. 7). Initial movement of the slide compressed the soil to form the pressure ridge, then movement was retarded at this debris dam, and finally failure progressed by entrainment of material in a narrow slide area averaging about 40 feet wide, but reaching 50 feet in width, and for a total length of 600 feet to the bottom of the ridge. It extended out into the Delaware River for an additional 60 feet. Fortunately, no canoeists were present in the river at the time.

Figure 12. Initial failure area at the top of the Sambo Island landslide. This area is 37 feet wide and extends down 90 feet to a pile of debris. The bedding dips 32o, paralleling the slope. The slide area extends 600 feet down to the Delaware River.

For most of the slide area, the soil has slipped off a single bedding plane, which remains fairly constant in dip (38^{o} near river level, although complicated by a small fold there) down to an altitude of 440 feet. At about 130 feet above the river, higher bedding planes are exposed and the slope is offset upwards by about 10 feet. Glacial till (fig. 9) was encountered and the material spread out as a gullied debris fan (figs. 4 and 9), 40 feet wide at the top, and 125 feet wide at the bottom where it removed the Pioneer Trail just above the Delaware River. The debris included bedrock fragments, soil, till, and trees, which slid out into the river. The total volume of soil displaced is calculated to be about 48,000 cubic feet (1,800 cubic yards).

The instability that produced the landslide probably began as sheet wash of soils, as evidenced by many small (less than 20 feet long) debris fans made of forest litter and piling up of soil up to two feet high on the uphill side of trees. Initial failure may have been sited at fractures, similar to ones present near the slide (fig. 13). Some of these tension cracks were developed along animal trails.

Figure 13. Irregular tension cracks (below dashed line) several inches high and a few tens of feet northeast of the initial failure area. Arrows show slight downward movement of the soil.

In summary, the landslide near Sambo Island is a soil slip of a thin veneer of shale-siltstone-chip gravel and glacial till that merged into a debris flow at the bottom. It is sited along the steep cutoff northwest slope of Kittatinny Mountain at the outer bend of a meander in the Delaware River. The presence of old landslide deposits indicate the potential for a landslide hazard here and elsewhere where similar conditions exist. The factors that make some of the area prone to instability and landsliding are (1) fairly steep slopes, more than 30 degrees in many places, (2) a thin soil cover, including glacial till, that rests on glacially polished bedrock surfaces, (3) tree roots, which otherwise could anchor the soil to bedrock, that do not penetrate the bedrock, (4) water seeps along the soil-bedrock boundary, making for a low-stress condition at this interface, (5) removal of the toe of the potential failure area by stream erosion (or the works of man), and (6) tension cracks similar to those seen near the landslide. Figure 5 outlines the slope adjacent to the Sambo Island landslide that meets these criteria for instability.

ROCKSLIP NEAR TOCKS ISLAND ALONG THE OLD MINE ROAD

The Old Mine Road in New Jersey (fig. 2) cuts through siltstone and very fine-grained sandstone of the Bloomsburg Red Beds for a distance of 2,400 feet adjacent to Tocks Island. Smooth longitudinal joints whose strike parallels the road (averaging about N. 70° E.) and which dip steeply towards the road (50-70° northwest) produce slabs averaging about 1 foot thick (fig. 14). These joints are smooth and regular and suggest sheet jointing or exfoliation resulting from expansion due to release of confining pressure due to rapid erosion along this stretch of the Delaware River.

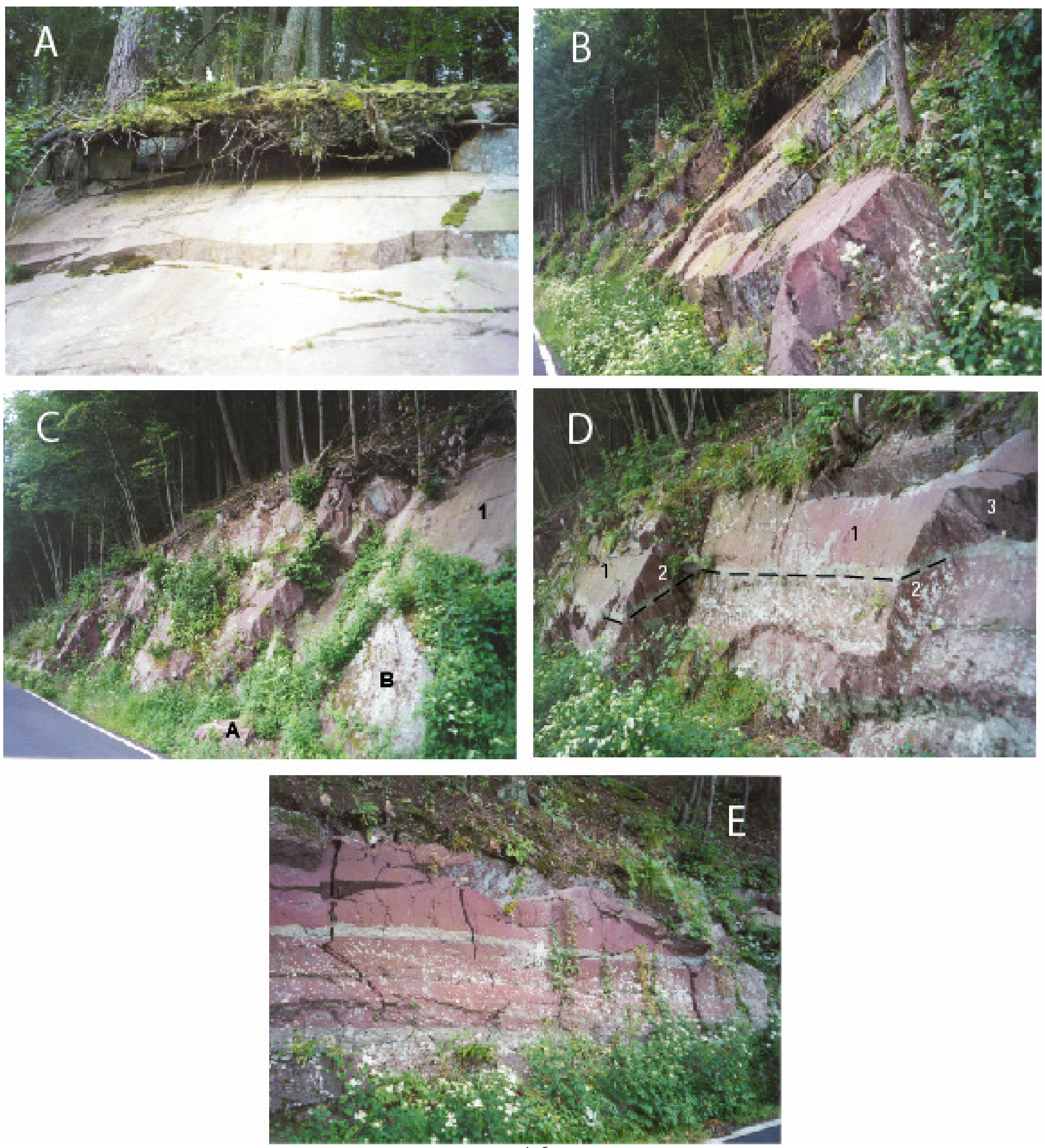

Figure 14. Exposures of the Bloomsburg Red Beds along the Old Mine Road in New Jersey, opposite Tocks Island, Delaware Water Gap National Recreation Area, showing how joints, sheeting, and cleavage affect development of rock slabs prone to sliding. *A*. Longitudinal sheeting joints dipping 48° NW and producing slabs between 10 inches and 3 feet thick. Shallow tree roots do not penetrate bedrock. *B*. Bare Bloomsburg bedrock due to rockfall along sheeting joints. Note shallow soil and tree roots that do not penetrate the bedrock. *C*. Sheeting joints (1) dipping steeply to the northwest and two rock slabs (A, 8 feet long; B, 10 feet long) that slid off during 1999. Other slabs that covered the road have been removed. *D*. Longitudinal (sheeting) joints (1) dipping 57° NW and cross joints (2) separate the rock into slabs between 2 and 8 feet long. Rock cleavage (3) dips steeply southeast and bedding (dashed line) dips gently NE. *E*. Irregular cross joints cutting steep longitudinal joint surface and separating rock into masses 2-8 feet wide.

The rock is further cut by irregular steeply dipping cross joints (striking 2°-53° northeast; fig. 14 C,D, and E) and fracture cleavage (averaging N60°E, 65°SE;fiog. 14D). The road cut is as much as 25 feet high along this stretch. The slope above the road cut comprises ribs of bedrock outcrop and colluvial debris on slopes of about 38°. The bedrock is massive and bedding is generally indistinct; it is recognized mainly by green reduced layers. The bedding dips more gently than the longitudinal joints, ranging between 16° and 44° northwest (fig. 14D). Bedding has not influenced slope instability along this road cut. Figure 15 illustrates the geologic features at the site and their control on rock slips at this locality.

Because the toe of the rock mass has been removed by road construction (fig. 15), rock masses as much as 10 feet long have slid down the smooth joint surfaces. Figure 14C shows two masses that fell off the outcrop, partially blocking the road in 1999. The soil above the bedrock is thin, about 1 foot thick, and tree roots do not penetrate into the bedrock (figs. 14A and B). Thus, the soil is not stabilized by the trees and, just as at the Sambo Island landslide, there is potential for soil slip along this stretch of steep slopes. These small rock falls are a continuing problem.

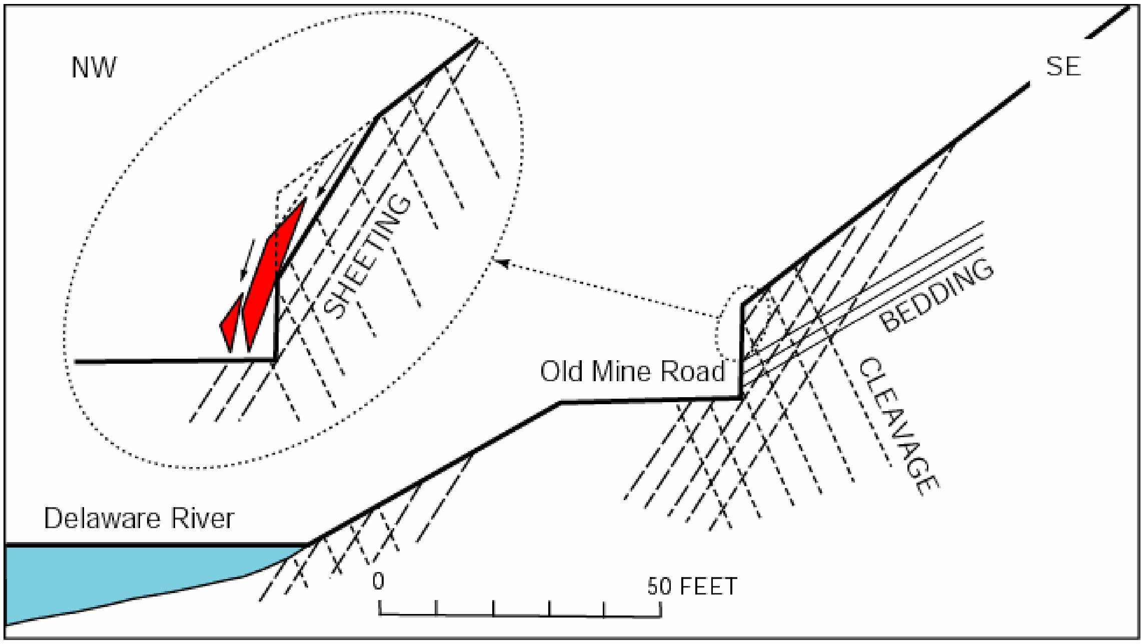

Figure 15. Cross section showing orientation of longitudinal joint set (sheeting) and cleavage in the Bloomsburg Red Beds along the Old Mine Road opposite Tocks Island on the Delaware River, New Jersey. Because the road cut along the Old Mine Road is steeper than the dip of the joints, the toe of the rock mass has been removed making the area susceptible to rock falls along the joint surfaces. Cross joints, parallel to the plane of the section, aid in breaking the rocks into slabs as much as 10 feet long. Along the Delaware River, the surface slope is less than the dip of the joints, so there is little likelihood that rockfalls will form.

ROCKFALL IN DELAWARE WATER GAP ALONG THE OLD MINE ROAD

Worthington State Forest includes part of the gap of the Delaware River through Kittattiny Mountain in New Jersey (fig. 2) and is surrounded by the Delaware Water Gap National Recreation Area. During the early 1980's a landslide, presumably a rockfall, removed a part of the west side of the Old Mine Road in the park, one thousand feet northeast of where I-80 crosses the Delaware River (fig. 16). The failure occurred in interbedded siltstone and shale and minor sandstone of the Bloomsburg Red Beds on the crenulated southeast limb of the Cherry Valley anticline which, 800 feet to the northeast, brings gray quartzite and conglomerate of the Shawangunk Formation to the surface. A 50-foot cliff parallels the road here and the slope above is steep, averaging about 43^{o}. Figure 17 is a diagrammatic cross section at the site. The landslide along the road probably occurred along sheeting fractures, but failure may have also been by rock toppling along steep cross joints.

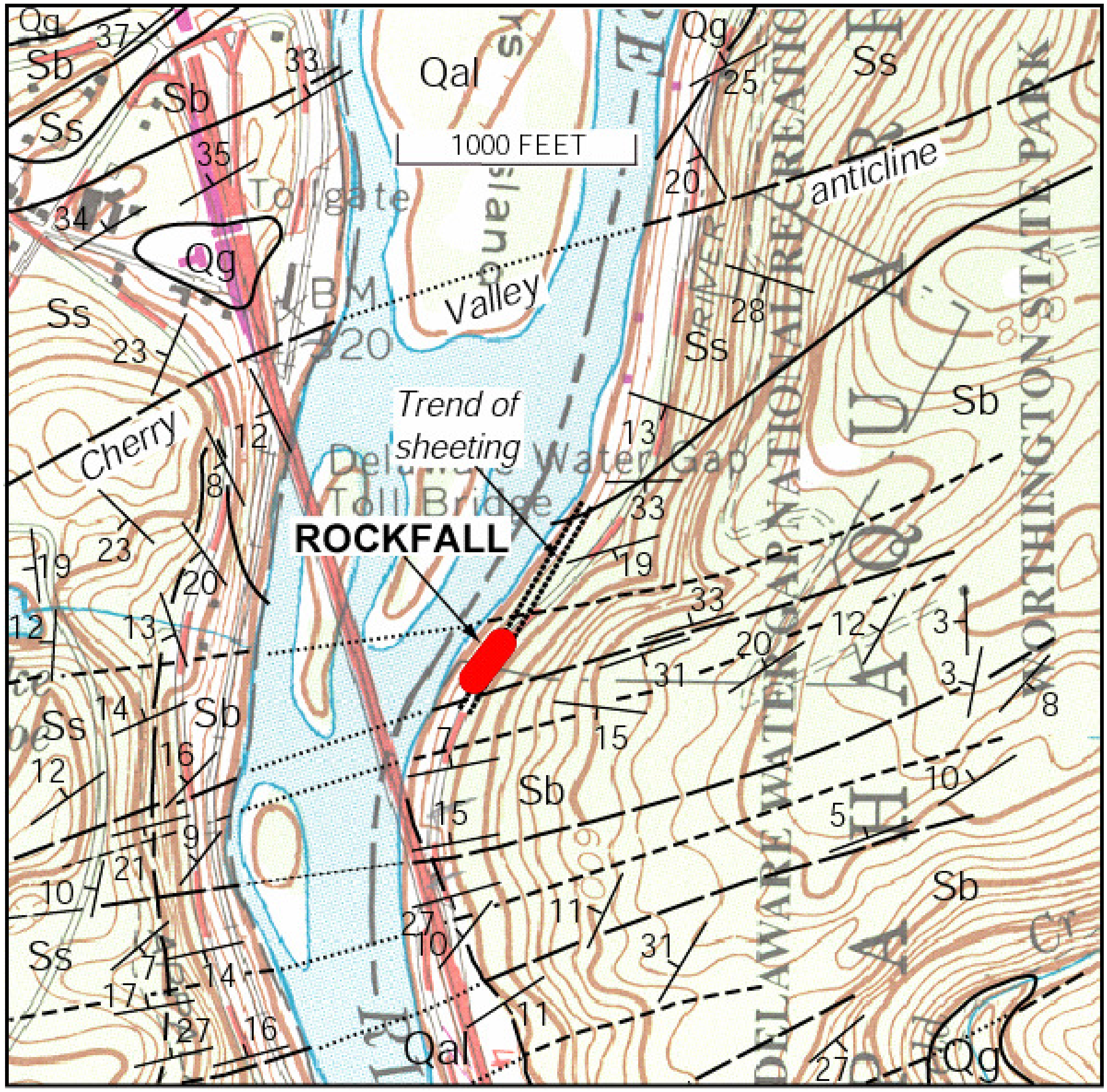

Figure 16. Geologic map of part of Delaware Water Gap showing location of rockfall along Old Mine Road and relation to geologic structure. Heavy dotted line is trend of sheeting fractures. Qal, Holocene alluvium and stream terrace deposit; Qg, Wisconsinan glacial drift; Sb, Bloomsburg Red Beds; Ss, Shawangunk Fromation. Solid lines, contacts. Long dashed lines, anticlines. Short dashed line, synclines. Modified from Epstein, 1973.

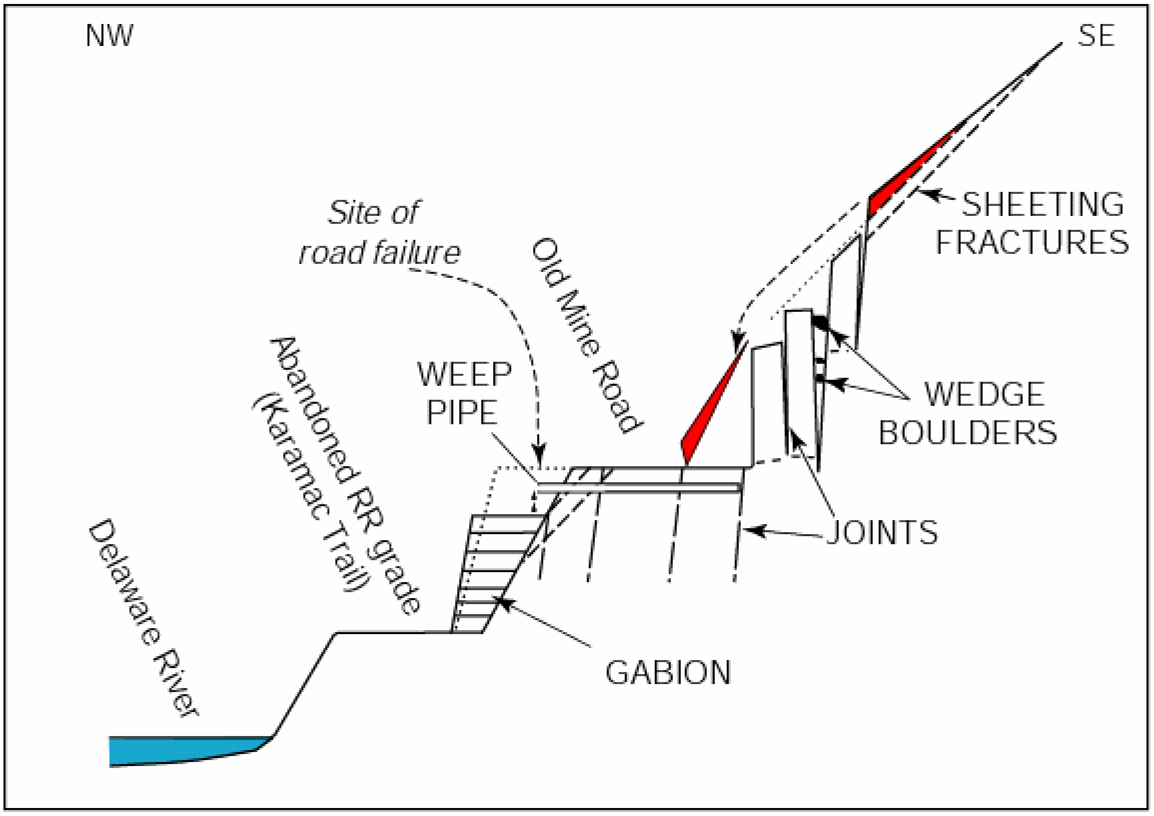

Figure 17. Generalized profile showing the relations of joints, sheeting, wedge boulders, and topography that resulted in rock sliding and toppling along the Old Mine Road in Worthington State Park, New Jersey. Dashed lines beneath road bed indicate joints and sheeting fractures that probably were related to the road collapse about 50 years ago. Additionally, the dotted block contributed to the failure by sliding off the sheeting joint. Dotted line indicates configuration of pre-rockfall topography.

The collapsed section of the road is now supported by a concrete gabion, about 100 feet long and 15 feet high (figs. 17, 18), located along the abandoned railroad grade below the road, now used as a foot trail. A one foot diameter weep pipe, about 30 feet northeast of the gabion, drains a wet area along the road. At the time of one visit, water coming out of the pipe was much less than the water falling from the rocks above and into a drainage along the road. At this locality the rock is highly fractured and veined in a zone ten feet high that parallels a bedding-plane fault. Bedding trend is N. 68° E., 41° SE. and slickensides plunge 27°, S. 34° W., with steps indicating movement downdip to the southwest. Much water soaks into the substrate below, a possible concern for future failure. Just above the fractured zone is a set of sheeting fractures, similar to those at the Tocks Island rockfall, along which the failure probably took place (figs. 17, 19). The strike of sheeting parallels the road at the failure site (fig. 20) and dips

45° NW. Because it parallels the slope of the ridge and because it□s orientation is different than most other joints produced by tectonic forces, exfoliation is believed to be the origin of these joints.

Figure 18. Concrete gabion along abandoned railroad grade below the Old Mine Road in Worthington State Park.

Figure 19. Sheeting joints along which the landslide along the Old Mine Road near Delaware Water Gap probably occurred. The shear zone is often wet and water seeps into the road bed below, more than is emitted from the weep pipe shown in figure 17.

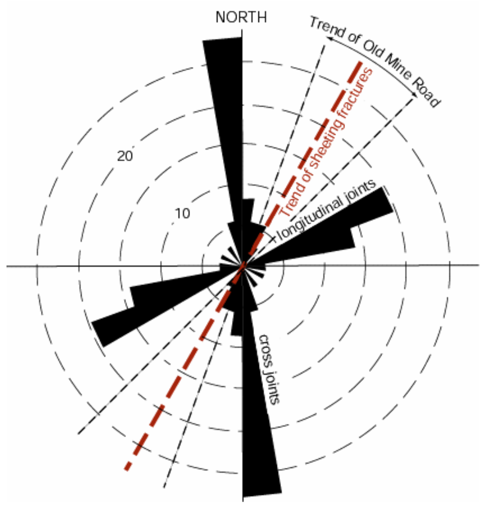

Figure 20. Rose diagram showing trend of longitudinal joints, cross joints, and sheeting fractures along Old Mine Road in Worthington State Park, New Jersey.

Failure may have also occurred by separation and toppling along cross joints (fig. 21) that trend slightly west of north (fig. 20). These joints, prominent south of the gabion, are aligned about 25^{o} from the trend of the road. Rock fragments and angular boulders have fallen into the cracks and over time have wedged the rock apart. The potential for future toppling is obvious, especially for those rocks on the steep hill high above the roadway.

Figure 21. Irregular cross joint, trending slightly west of north and at an angle to the trend of the Old Mine Road (see figure 20). Boulders in the fracture are wedging the block apart, creating the potential for toppling.

Thus, several geologic factors appear to be involved in two distinct types of failure and potential failure. These include joint sheeting dipping towards the roadway, shearing along a fault, and rock masses being forced apart by wedging along cross joints. Additionally, rock cleavage, which averages about N. 60^{o}E., 65^{o}SE., is nearly parallel to the Old Mine Road, creating another plane of weakness along which the rock may separate.

BRODHEAD ROAD DEBRIS FLOW IN GLACIAL TILL

The Wisconsinan glacier retreated from the Delaware Water Gap area less than 20,000 years ago. It left behind a variety of geologic deposits, including sorted sand and gravel, mainly in the valleys, and glacial till, consisting of a heterogeneous mixture of clay, silt, sand, and boulders. The till has the potential for flow when it becomes water saturated on moderately steep slopes. A debris flow developed about 1996 on the east side of Brodhead Road, 1.3 miles northeast of Bushkill, PA, and 1500 feet north of US 209 in the Flatbrookville quadrangle (figs. 3, 5, and 22).

Fig 22. Scarp and debris flow in clay-silt till along Brodhead Road. The largest boulder in the till is two feet long.

The bottom of the slope was excavated by construction of the road, cutting out the toe, and expediting the sliding. The landslide is 60 feet high with the steeper head scarp 20 feet high. The angle of the original slope was 37^{o}, now steepened by the sliding in the upper part. The landslide occurred in a moderate- to dark-yellowish-brown (10 YR 5/4-4/2), poorly sorted, compact, clay-silt till with scattered boulders as much as 2 feet long, but averaging about 3 inches long. Boulders more than 5 feet long have been noted nearby. Stones were derived from underlying siltstone bedrock and gray and red sandstones from northerly rock units including the Trimmers Rock and Catskill Formations. The most recent slide is 140 feet wide, but there are older slide scars extending for an additional 300 feet along the road. The slope is steep, having been eroded along the outside meander of the creek that now lies west of the road. Another smaller slide area lies about 200 feet south along a cut bank on the opposite side of the creek. Two wire-mesh gabions, each 200 feet long and separated by 50 feet, were constructed along the road along the total slide area to prevent future movement onto the road (fig. 23). The potential for future landsliding is shown in figure 5 where till is present on slopes greater than about 30 degrees. The slide along Brodhead Road is presently stabilized and overgrown.

Figure 23. Gabion, about 450 feet long, along the glacial debris flow along Brodhead Road.

FLATBROOK DEBRIS SLIDE

A landslide involving cemented gravels of Wisconsinan age occurred during the past year along a steep slope 0.3 mile northeast of the road intersection in the town of Flatbrookville, Pa. (figs. 5, 24). The head scar is about 60 feet above the road surface and about 10 feet high (fig. 25). The gravel is composed of well-rounded clasts as much as 10 inches long, comprising a variety of rock types, including sandstone, limestone, red beds, and shale, in a coarse-sand matrix. (fig. 26) It was able to maintain a steep slope because it is cemented with calcium carbonate derived from the underlying Bossardville Limestone. It is in a cut bank on the outside curve of Flat Brook. Concrete barriers are positioned to prevent rocks from falling onto the road. Some large blocks of Coeymans Limestone, several feet long, are mixed in with the debris.

Figure 24. Concrete barriers confining falling rock from a debris slide near a cut bank of Flat Brook.

Figure 25. Coarse gravels in the head scar of debris slide slide, about 60 feet high.

BEDDING-PLANE FAULTS IN THE BLOOMSBURG RED BEDS AND THEIR POTENTIAL FOR FAILURE

Faults along bedding in the Bloomsburg Red Beds were recognized by Epstein and Epstein (1967, 1969) throughout eastern Pennsylvania and northwestern New Jersey. They consist of polished and slickensided bedding surfaces, occasionally in a zone as much as one foot thick. Steps on the slickensides invariably indicate that overriding beds moved to the west, regardless of the rock's position on a given fold. Wedges (small ramp thrusts) and dragged cleavage also corroborate this general sense of movement (fig. 24). These faults are commonly zones of weathering and ground water flow.

During engineering evaluation for the proposed Tocks Island dam after the Delaware Water Gap National Recreation Area was authorized by Congress in 1965, the U.S. Army Corps of Engineers (ACE) encountered 360 feet of laminated and massive, partly desiccation-cracked, red siltstone and shale with some greenish-gray mottling and minor gray quartzitic sandstone in a 600-foot long exploratory adit in the Bloomsburg at the base of Kittatinny Mountain in New Jersey near Tocks Island (figs. 2 and 25). The maximum stress level parallel to one of these faults was determined as 1,000 psi during excavation for the proposed spillway (x in figure 25), and was attributed to the weight of the entire rock mass between the fault and the surface (Dan Parillo, ACE, written communication, 1970). The maximum stress level below the fault was 525 psi, indicating that there is little or no strength across the fault and if the toe were daylighted the entire uphill mass would move downhill, according to Parillo. Many of the bedding faults shown in figure 25 are zones of abundant water flow.

Even though the Tocks Island dam was never constructed, there should be concern about future road building or other construction along the Old Mine Road in New Jersey, or anywhere else where these conditions exist in the Bloomsburg. Should any of the bedding faults be intersected during road construction, there is the potential for massive failure along these zones of weakness. There are many of these zones in the Bloomsburg and individual potential construction sites need to be analyzed for their presence. One such fault zone, which may have contributed to failure, is the rockfall in Worthington State Park, discussed above.

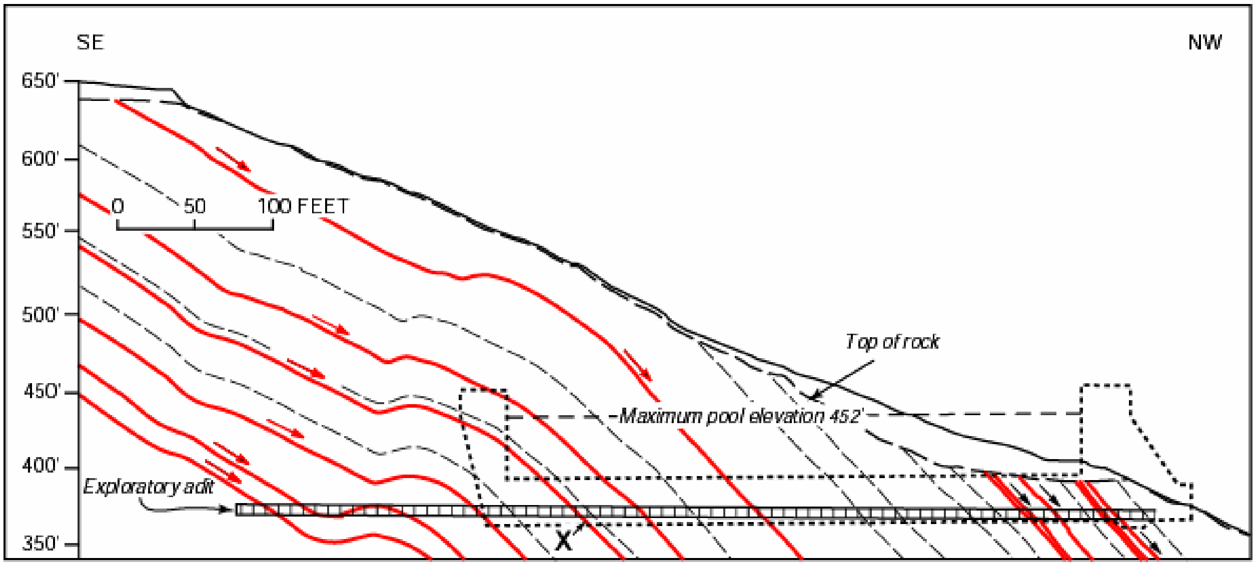

Figure 25. Cross section through the Bloomsburg Red Beds in the lower part of the northwest slope of Kittatinny Mountain in new Jersey near Tocks island (modified from Depman and Parrillo, 1969). Structure determined from exposures in 600-foot-long exploratory adit and several drill holes. Solid heavy lines are bedding-plane faults, arrows indicate direction of movement of overriding beds; short-dashed lines are normal bedding surfaces. "X" is bedding-slip fault along which stress measurements are discussed in text. Dotted lines show proposed spillway for the now-deauthorized Tocks Island dam.

ROCKFALL HAZARD AT LEHIGH GAP

Well-developed longitudinal and cross joints in competent Paleozoic rocks are a consequence of folding in the Appalachian Mountains of Pennsylvania. The orientation, concentration, and character of these joints, in relation to slope aspect, can lead to instability. PA 248 is a major four-lane highway through Blue Mountain in Lehigh Gap south of Palmerton, Pa. The northbound lane of the road is cantilevered above the southbound lane, their location is constrained by a railroad along the Lehigh River to the left and a railroad, now abandoned, above. The roadway is also constricted by the resistant coarse clastic rocks of the Shawangunk Formation in the gap. Slope failure at the site is due to conspicuous north-northwest-trending cross joints in the sandstones and conglomerates in the Shawangunk that parallel the highway. These generally irregular and roughly planar open joints have created overhanging blocks of rock many tens of feet long and wide (fig. 26), creating the potential for rocks toppling onto cars in the road below. A more detailed description of the geology of the site and of the rockfall hazard are presented in Stop 4 of this Symposium (Epstein, 2007).

Figure 26. Open cross joints in conglomerates and sandstones of the Shawangunk Formation at Lehigh Gap. Views looking north (left) and south (right). Loose blocks of rock up to ten feet long have fallen to the road below. The unconformable contact between the Shawangunk (Ss) of Silurian age and shales, slates, and graywackes of the Martinsburg Formation of Ordovician age is seen along the abandoned railroad grade 60 feet above the highway.

The parallelism of the cross joints and the highway are not fortuitous. Cross joints form perpendicular to fold axes; erosion of the resistant rocks in these folds forms ridges parallel to the fold axes. Water gaps cut through the resistant ridges along planes of weakness controlled by cross fractures and other structural anomalies (Epstein, 1966). The gaps form a transportation corridor through the ridge. If the highway cut is steep, as it is at Lehigh Gap, overhanging rocks divided by through-going cross joints create loose blocks and the potential for slope failure. The orientation of joints at the gap site and in the surrounding area are shown in figures 5-15 and 5-16 in Epstein (2007).

The Martinsburg Formation is exposed south of the Shawangunk. It weathers into small fragments along cleavage, bedding, and joints, leading to spalling from the steep face above the railroad grade. The use of gabions, fences, and screens to prevent these products of erosion from falling on the road below, in both the Martinsburg and Shawangunk, is discussed at Stop 4

(Epstein, 2007).

In 1990, following an evaluation of the landslide potential, the potential for slope failure was mitigated by removal of large blocks of rocks from the fractured Shawangunk and by construction of a gabion along the outcrop of the Martinsburg Formation to prevent erosion and rock spalling. More recent mitigation efforts are discussed at Stop 5.

CONCLUSION

The complex geology, both bedrock and surficial, in northeastern Pennsylvania and northwestern New Jersey presents a variety of conditions that have resulted in slope instability. An understanding of these conditions would be useful in predicting the potential for slope failure in similar environments. Bedrock instability is favored by (1) fractures in bedrock, such as joints, bedding, and cleavage, (2) the relation of slope aspect to the orientation of these fractures, and (3) type of geologic material. Omnipresent glacial deposits are prone to failure on steep slopes either along cut-banks along streams or due to highway excavation. Glacial deposits and shallow soils developed on ice-scoured and polished bedrock, especially where they have not been anchored by tree roots, are prone to sliding on steep slopes. Appreciation of these factors and an adequate geologic database would be helpful in avoiding or mitigating the types of slope-failure hazards.

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Low Altitude Large Scale Reconnaissance

Or Is This A Great Job or What?

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ABSTRACT

Low Altitude Large Scale Reconnaissance

Or Is This A Great Job or What?

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The Colorado Department of Transportation (CDOT) has been using Low Altitude Large Scale Reconnaissance (L.A.L.S.R.) since 1994 for the identification of geologic hazards, environmental issues, and project documentation. CDOT performs LALSR by using what is called a U.A.S. or Unmanned Aerial System as defined by the Federal Aviation Administration (FAA)**.**

What this really means is CDOT uses Remote Controlled (RC) airplanes and helicopters to perform aerial photography of sites which are either too difficult or costly to fly using conventional aerial photography or satellite imagery applications.

This presentation will present the aerial systems CDOT employs, the applications it is used for, and case histories as well as lessons learned since the inception of this program.

Introduction

L.A.L.S.R. is an acronym for Low Altitude Large Scale Reconnaissance. What it provides is inexpensive, high resolution digital photographs at low altitudes.

L.A.L.S.R. photography fills a gap in aerial reconnaissance. In remote locations it is less expensive, more flexible, and gives better resolution than satellite photography and is lower in cost and easier to access difficult or restricted areas than with manned aircraft. It is another tool in our bag that we can use to help us gather information.


UAV Airplane **UAV Helicopter**

The Colorado Department of Transportation's (CDOT) "air force" consists of Unmanned Aerial Vehicles (UAV's as the FAA refers to them) of 2 Remote Controlled (RC) airplanes and 2 RC helicopters. Our airplanes are self designed and are used to fly areas where detail is important and areas are greater than ¼ mile in length and altitudes above 500 feet are required. The airplanes are capable of flying to altitudes of 1500 feet above ground level and to distances of up to 1 mile from the radio control point.

Our helicopters are used in areas where greater detail and smaller areas of operation are required. These generally being in the mountain canyons of Colorado where space is limited and aerial photography is difficult to provide using other aerial photography systems. They are capable of flying to altitudes of up to 200 feet above ground level and have a radius of operation of up to 1000 feet. Our unmanned helicopters are used in our rockfall program to identify rockfall hazards at mitigation sites.

We use a small digital camera mounted in our UAV aircraft and pictures are taken remotely from the ground via sophisticated radio transceivers. Two cameras are mounted on the UAV aircraft, one giving a fixed "pilots" view and the second "ball mount" providing multiple angles of view which the camera operator can adjust in flight to capture pictures from vertical to almost any oblique angle. The flight crew on the ground always has visual confirmation of the shots via these video cameras mounted on the aircraft. The UAV's can actually be flown using the video downlink and monitors.

The CDOT UAV's are extremely versatile. This lightweight aircraft and supporting equipment is transported in a van that allows the crew to access almost any site, no matter how remote. The plane requires approximately 500 feet of "runway" and "take-off" from bridge decks, frontage roads, and even by hand launching have all been successfully accomplished. Landings also require little in the way of space. Our helicopter needs only a 12 foot by 12 foot open space to take off and land.

CDOT Operations Van

This is especially useful in restrictive or narrow areas such as canyons and construction projects. The UAV is flown only visually and during daylight hours when weather conditions allow. An operations van is used to capture the video downlink and uses VHS recorders and TV monitors to fly in "real" time.

The Aerial Reconnaissance Programs' use of UAV's for aerial photography is significant because it provides an inexpensive way to acquire quality, large scale photographs. The use and cost savings have successfully allowed a wide array of applications to include: geologic hazards such as landslides and earth flows, rockfall are design, mitigation and emergency response; and as historical references. Other used include support for environmental restoration with color and infrared photographs, documenting flood damage to structures and monthly and/or yearly recording of work progress for ongoing construction projects. Site characterization activities such as drilling, hazardous waste sampling can be documented as well. Pre- and post-closure conditions at a site can be documented, supplementing standard ground photography. Right-of-way properties can be confirmed and portions of aerial coverage no longer reflecting current conditions can be updated. CDOT, the Colorado Geologic Survey (CGS) and local agencies have used the program to identify natural hazards and Military Affairs has used the program to inventory of all the Colorado Armories.

These systems are not toys so the Federal Aviation Administration (FAA) has developed guidelines for their use. Both pilot and ground crew must either be certified pilots or have completed an accredited flight ground school. Then your aircraft must be certified as "airworthy" by your agency. A Certificate of Authorization (COA) must be filed with the FAA and approved. This contains information such as location, duration, and date of scheduled flights in addition to information about the pilot, ground crew, and UAV system. Cost and time for this is approximately 40 hours of training and $500 per person.

History

The Colorado Department of Transportation has been using the L.A.L.S.R. system for the past 13 years to identify geological hazards, environmental concerns, archeological and paleontological sites and for project documentation.

CDOT's 1st UAV

The concept of using RC airplanes for low level photography was first conceived by Ken Wood, Research Engineer, of the CDOT Research Branch. In 1994 a pilot program was implemented by Brandi Gilmore, Supervising Geologist, and Robert Florez, Senior Geologist, of the Staff Geotechnical Unit. It was a research project designed to determine the usefulness of aerial photography for a variety of CDOT environmental and engineering projects. The photography had to be low cost, large scale, high resolution, and available on short notice.

This was all accomplished in the first few years using homebuilt airplanes and 35 mm cameras. These airplanes were flown visually and only took vertical photographs, but yielded photograph scales between 1:100 to 1:1500 with high resolution and no image blur.

Since 1994, there have been 14 airplanes that have been built and used by the Aerial Reconnaissance Program and each new UAV has had significant upgrades. The latest version now uses digital photography, a ball camera mount, fiberglass body, two video downlink cameras, and molded carbon fiber re-enforced styrofoam wings.

The use of our helicopter in the Aerial Reconnaissance Program started in 2002. They have been upgraded and now are used almost exclusively in areas that are restricted from airplane use due to flight constrictions. They also use digital photography and use a swivel mount capable of providing multiple angles for photography.

These upgrades have allowed the current UAV's to be smaller, lighter, and more durable while providing better quality photography along with greater flexibility. The use of the digital camera allows for the transfer of photographs to the requestor that same day. Using any number of photographic computer programs allows the project engineer to digitally annotate the photographs and more easily place them into project logs and presentations.

Costs

Our annual budget to operate is $25,000 per year with this covering construction flights as well as maintenance and repair to the UAV's. The cost is inexpensive, with flights in the Denver area usually costing less than $300 per flight. This compares to manned flights in the Denver area costing around $1,000 per hour. Start up costs for this program is around $2,000 which includes the fabrication of the airplane and the electronic surveillance equipment.

Airplanes are usually replaced about every one to two years. This is typically due to the unforeseen risks of flying. Radio interference, structural defects, mechanical failure, pilot error, turbulence from large commercial or military aircraft in the area or even large birds can create crashes.

Airplane Crash Site

Case Histories

Project Design and Development

Before major construction projects begin, designers like to know what possible impacts new construction may have on the public. Aerial photography has allowed the designers a "bird's eye" view of what they might encounter during construction and to plan for it.

Project Engineers use aerial photography for a variety of reasons. Usually as project documentation to see and have a record of progress made on the construction site and to help in estimating payments to the contractor. This also becomes a valuable tool when public forums are held. The project engineer has a visual aide to show what construction is planned and how it may affect the traveling public.

Many of these aerial photographs have been used in publications, during awards presentations, and as historical reference for future projects.

I 70 & SH 58 Interchange looking North

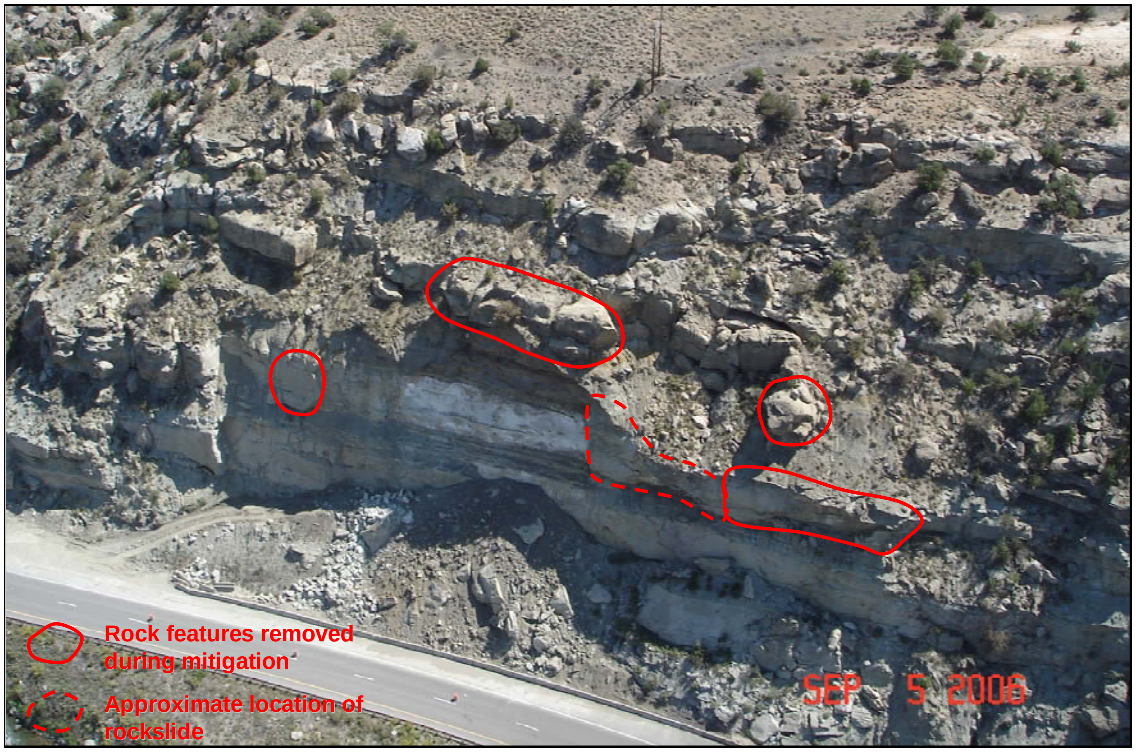
SH 58 & I 70 Interchange looking East

DeBeque Canyon

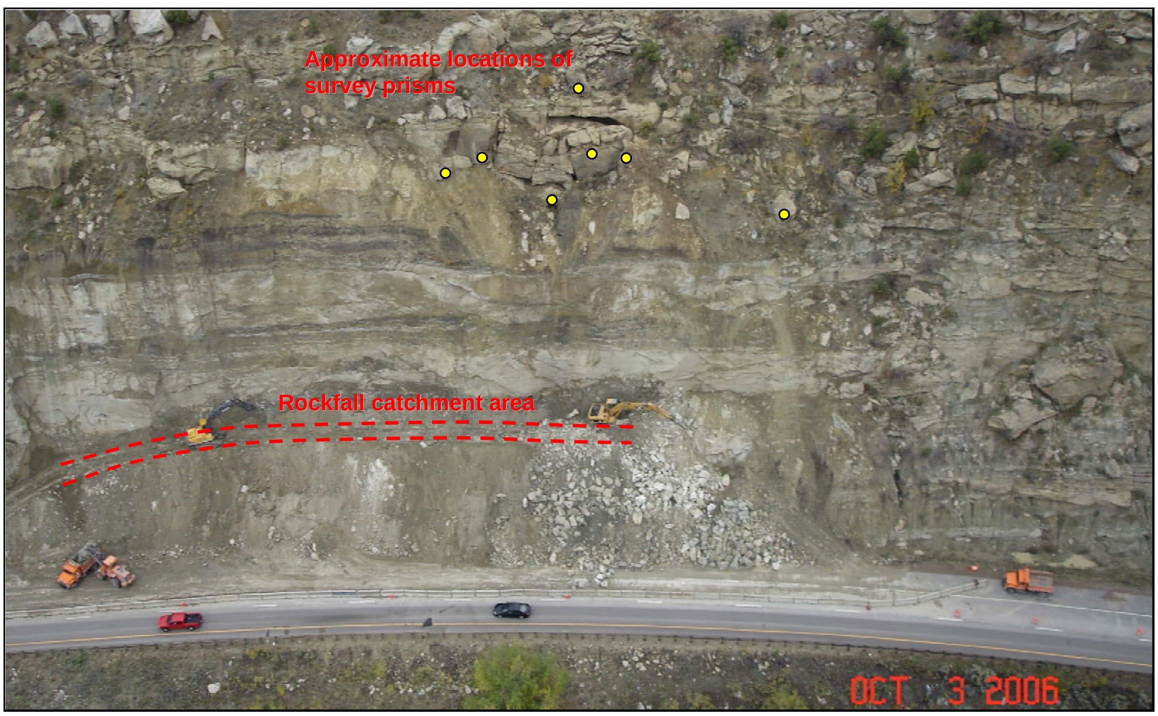
In September and October of 2006, the Aerial Reconnaissance Program was asked to fly the site of a large rockfall event.

Ground views of the rockfall site

The Aerial Reconnaissance Program was able to acquire the necessary photographs which significantly enhanced the mitigation efforts.



Slope failure and rock features to be removed were identified.



Monitoring devises were installed and rockfall catchment area was determined.

SH 67 Cripple Creek

In July of 2004, the Aerial Reconnaissance Program was contacted by the Regional Engineering and Maintenance sections and asked to investigate possible subsidence of a road due to mines in the area. The roadway was scheduled to be improved because of increased traffic volume produced by gambling in the nearby town of Cripple Creek.

SH 67 near Cripple Creek, Colorado

The Aerial Reconnaissance Program arrived and walked the site to determine where the subsidence was and where possible mine portals and adits might be located. After flying the site the program was able to determine the location of the subsidence and helped determine which shafts needed to be mitigated to stabilize the roadway to allow for the additional improvements to the roadway.

What Lessons Have We Learned

Plan ahead and do a site investigation prior to flying. Not all projects are good candidates for L.A.L.S.R. Take off and landing zones are crucial as well as safe zones to abort the flight if necessary. Be aware of traffic and overhead obstructions.

Communicate with local airports to advise them that you are in the area so they can notify other airmen in the area of a potential hazard to them.

Communicate with your pilot. If they are not comfortable with flying an area, then you shouldn’t be either. Consider the risks and the down time if you have a crash.

Be able to adapt. Take enough equipment with you so that if a failure occurs, it can be fixed in the field. One UAV may not be able to do all that you want.

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Design and Construction of a Highway Interchange over Abandoned Lead and Zinc Mines – Route 249 – Joplin, Missouri.

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ABSTRACT

The Missouri Department of Transportation is completing the Route 249 – US 171 interchange on an abandoned underground lead-zinc mining site near Joplin, Missouri. The interchange required the construction of 5 major bridges connecting massive earth embankments and involved the construction of 23 bents, 3 culverts and 3 MSE walls.

The geotechnical characteristics of the project site are the product of the original limestone and chert bedrock geology, the tectonic processes leading to brecciation, folding and paleo karst, and the anthropogenic (manmade) activities associated with mining. These processes resulted in a site that exhibited critical geotechnical attributes:

- Highly variable rock parameters (strength, deformability and hydraulic conductivity)
- Very low level of predictability or correlation between subsurface explorations
- Locally modified hydrologic regime due to the mining disturbance
- Presence of residual mining voids in both the upper “confused” zone and the lower “sheet” zone
- Potential for ground loss beneath foundation elements
- Presence of heterogeneous mine fillings including compressible clay
- Presence of vertical shafts with random and partial filling

These attributes led the design team to the selection of a foundation design concept that required bent specific geologic characterization based on subsurface exploration and geotechnical testing; pre design utilizing stress analyses and other analytical approaches to predict the interaction of ground quality, voids and imposed loading; systematic confirmatory exploration during construction; and ground treatment and foundation design modifications as required by the confirmatory construction site engineering and inspection work. The basic premise of this approach was that there were no foreseeable ground conditions at the project site that could not be adequately improved to provide foundation support for the proposed structures. Discussions during the pre bid phase emphasized ground conditions would be evaluated on a continuous basis in the field, reducing or eliminating any delays.

This paper describes the historical research, site investigation, design methodologies and construction of spread footings at 7 of the bents and the 220 micropiles used to provide deep foundations for 16 of the bents. Pregrouting of the rock mass involved 8698 yds^3 (6,650 m^3) of LMG and 504 yds^3 (385 m^3) HMG and was conducted to confirm design assumptions at all bents and as a site preparation for those bents to be micropiled. A separate program of mine shaft location and treatment was also required.

INTRODUCTION

The Missouri Department of Transportation (MoDOT) is constructing a five bridge interchange in Jasper County, Missouri connecting Route 249 and US 171. The project is a segment in the future I-49 Shreveport, La. to Kansas City, Mo corridor. HNTB with project team sub consultants Geosystems, L.P., Monir Precision Monitoring Inc. and Wyllie & Norrish Rock Engineers Inc. was retained by MoDOT to provide geotechnical exploration assistance, design of bridges, retaining walls and box culvert structures for the entire interchange as well as resident construction engineering and inspection.

Figure 1 shows the bridges, retaining walls and box culverts locations for the interchange project. The bridges range from two spans to eight spans with lengths ranging from 220 to 1457 ft. (67 to 444.2 m). Approach fill heights at the bridge locations range from 23 to 70 ft. (7 to 21.5m). The three Mechanically Stabilized Earth (MSE) walls range from 21 to 26 ft (6.5 to 8m) high. The three box culverts will carry the existing Mine Branch Creek through embankments at the south end of the project. Embankment heights over the boxes range from 43 to 75 ft. (13 to 23m).

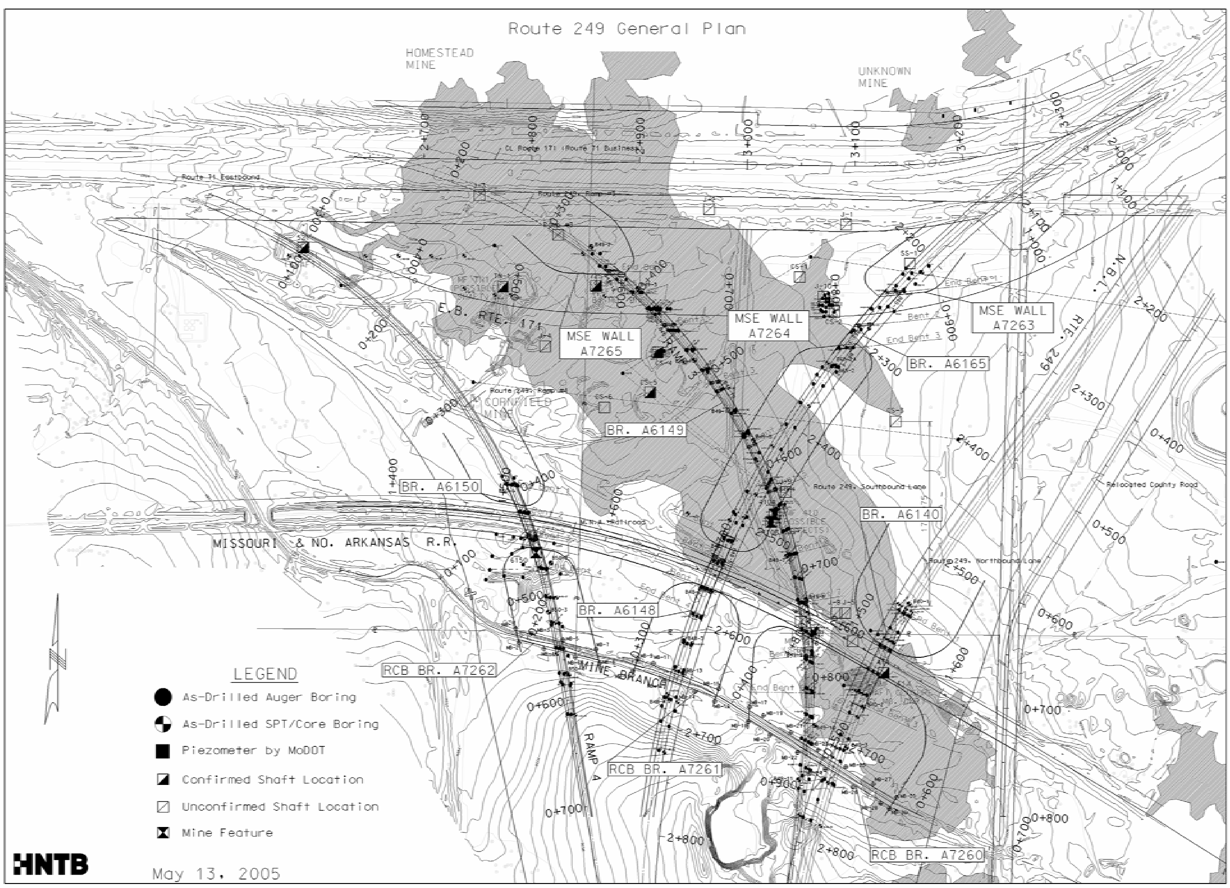


Figure 1 – Bridge Layout with Shaded Abandoned Mine and Suspected Mine Shafts

SITE HISTORY

The Tri-State mining district, so named for its location at the junction of Missouri, Kansas and Oklahoma, was formerly one of the largest lead and zinc producing districts in the world. Major minerals mined were sphalerite (zinc sulfide) and galena (lead sulfide). Mining began in the project area in the 1850's with excavation of the shallow "upper ground" deposits of insoluble residual minerals in the upper clay or in incompetent zones of broken, unconsolidated chert. These deposits were dug by hand in typically small claims of 100 to 645 ft² (30 to 60 m²). Mines were dug as deep as practical by hand; typically the limiting depth was the groundwater table, usually 50 ft. (15 m) or less below the surface. Many shafts were sunk at close intervals due to the unstable nature of the upper ground and the broken rock that prohibited drifting or horizontal mining. The gangue material was randomly placed as backfill in the mined areas. Ore was crudely smelted on site with wood fires.

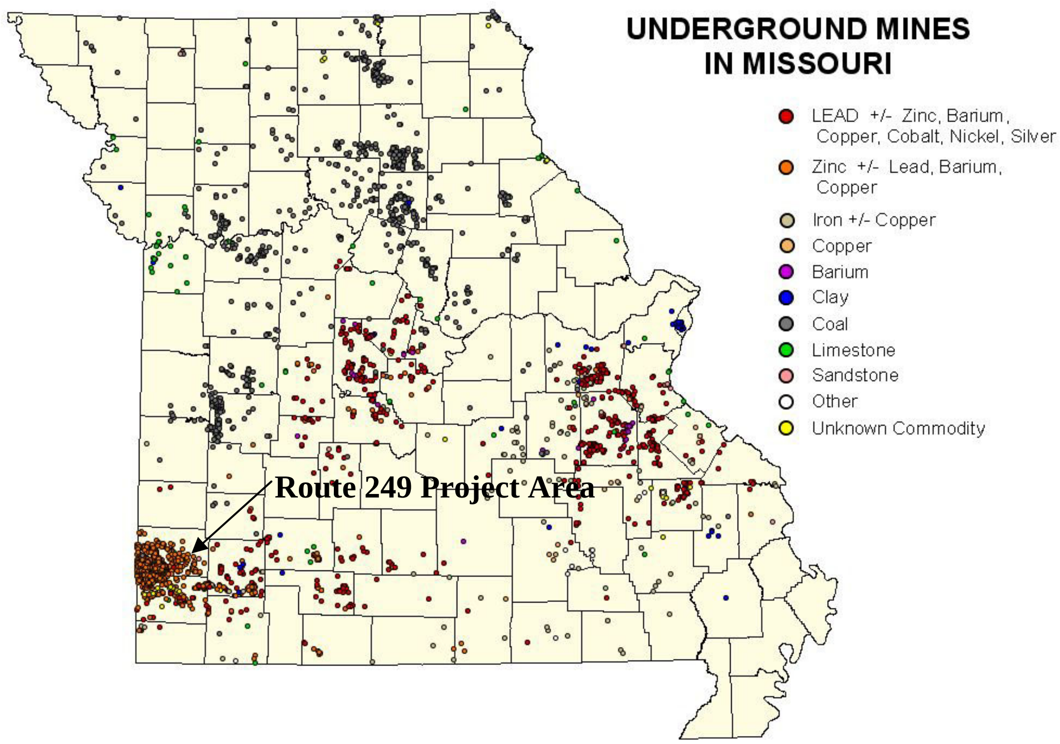


Figure by Missouri Department of Natural Resources

In the period near 1900, advances in mining along with capitalization brought mechanization, milling, hoisting, explosives and pumps with the ability to dewater the mines and lower the groundwater. Shafts were deepened and horizontal drifts began in the lower more competent rock layers. This began the era of large scale company mining on larger tracts of ground. The deeper shafts began mining veins or runs of the larger, extensive, lower grade "sheet-ground" ore bodies. Horizontal mining, or drifts, were highly irregular in size, shape and elevation, as the excavation followed the irregular shaped ore deposits. Only the economically feasible ground was mined. Mining references suggest a few of the openings may have been as large as 60 to 70 ft. (18 to 21m). More common openings found in the project explorations size the openings in the sheet ground at 3 to 6 ft. (1 to 2m) or less. The sheet ground deposits were mined by irregular pattern room and pillar methods.

Mining flourished during the World War I period in the Webb City Carterville Area. The Cornfield Mine associated with the project was thought to have been worked during the period 1910 – 1920. At the end of World War I mineral prices declined and operations ceased in this area with much of the equipment moved to the richer, more profitable Pitcher Field of Oklahoma. Lead and zinc were again in demand during World War II and, while not documented at this location, it was typical practice for a small mining operation to reopen some of the mines and scavenge any readily available ore. Another common practice was to rob the pillars supporting the mine. It was also during this period the vast mine tailings piles "chat" were reworked for additional recovery of minerals.

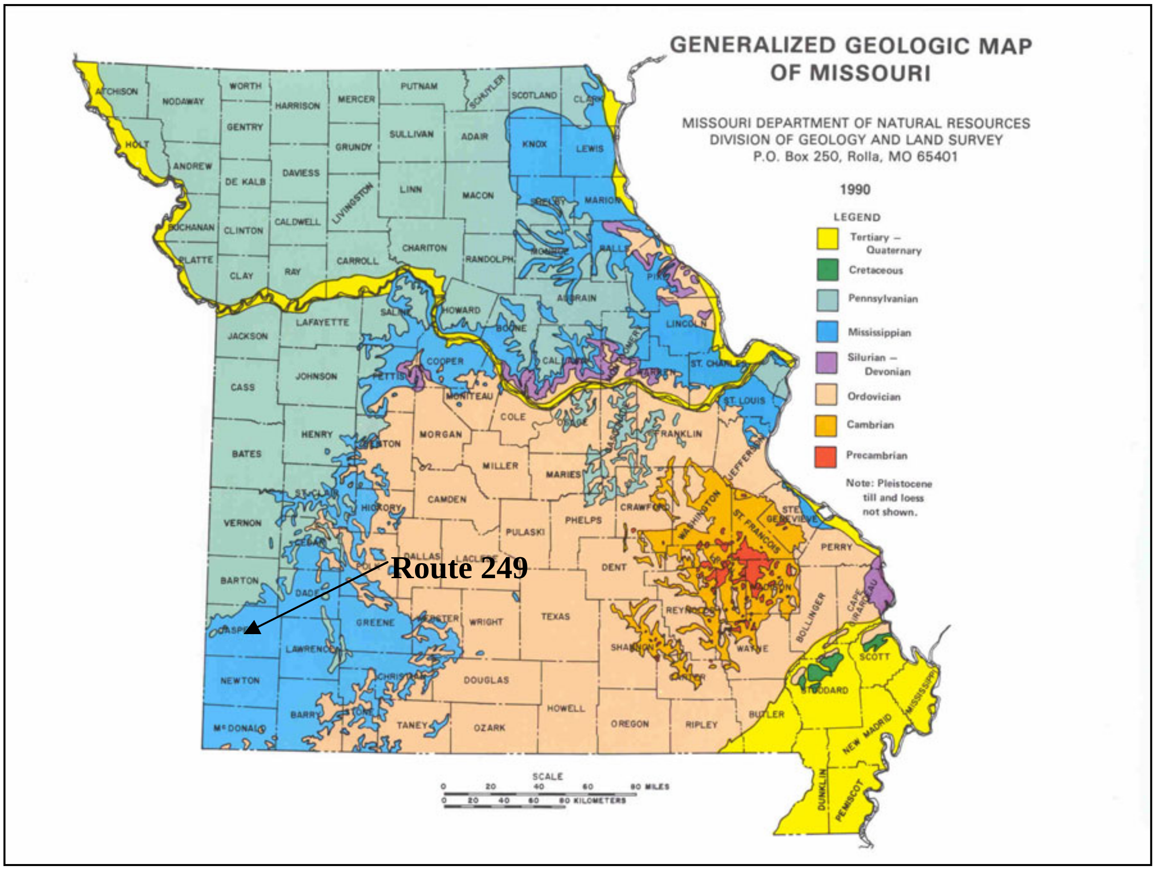
After the mines stopped operating, they filled with groundwater and are believed to be filled with water today. Also, since the closure of the mines, the enormous tailings piles have been transported off site for use as aggregate and mineral fillers. The Environmental Protection Agency has designated the tailings as a hazard due to the presence of heavy metal particles. Special material handling and encapsulation will be required for the proposed highway project. Present day evidence of mining on the right of way for the proposed project include, chat piles, mine shafts closed by Missouri Department of Natural Resources, and occasional surface depressions.

GEOLOGIC SETTING

The project area is situated within the Ozark Plateau physiographic province, a gently uplifted plateau of nearly horizontal sedimentary rocks. As the area is on the far west flank of the Ozark Dome, the dip is gently to the west – northwest at about 10 feet per mile. The plateau has been eroded to form a topography of rolling hills.

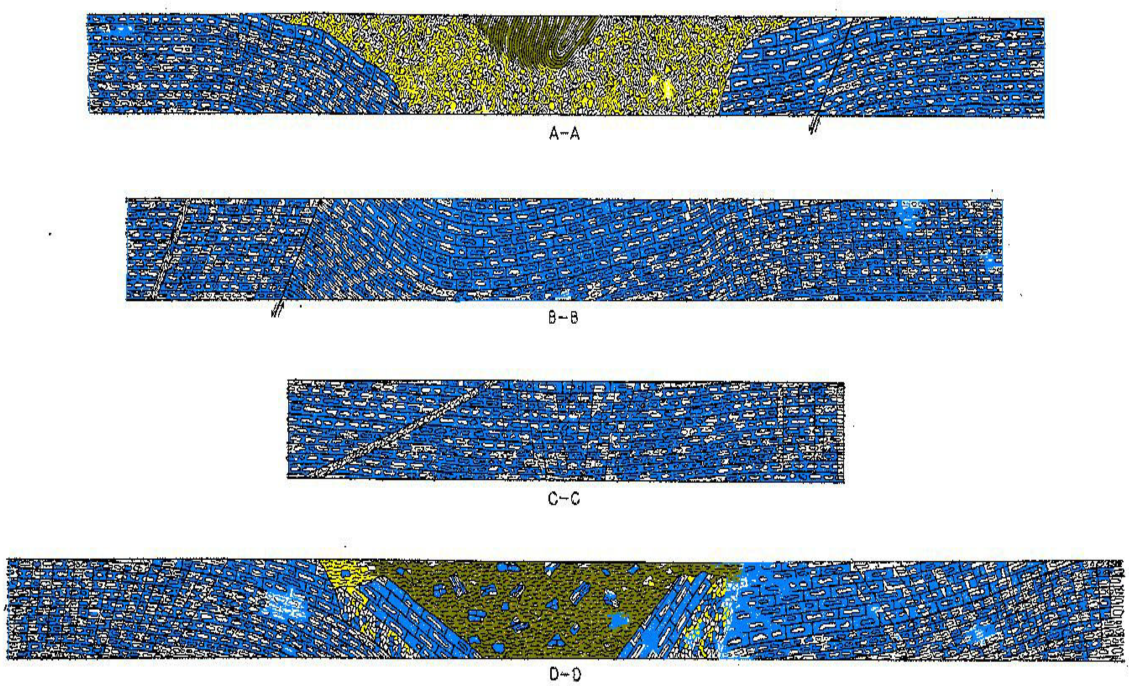
Structurally, the area is controlled by northwest – southeast trending Joplin Anticline and parallel east adjacent Webb City Syncline. References indicate the mineralization of the area appears to be confined to the synclinal areas.

Bedrock is of the Lower Pennsylvanian and Mississippian Age (Figure 2). The lowermost rock is the Reeds Springs Formation composed of nearly equal parts of chert and limestone. The chert is bluish to tan, nodular and irregularly bedded. Chert can make up one third to two thirds of the formation. The formation averages 100 to 150 ft. (30 to 45 m) thick in the project area. While not included in the modern nomenclature, the Grand Falls Chert Member, or Elsey Formation is usually included in the Reeds Springs formation, but this rock layer is prominent in the study area and is composed wholly or heavily predominant beds of chert. Above the Reeds Springs is the Burlington – Keokuk Formation, 65 to 100 ft. (20 to 30 m) of coarsely crystalline limestone with layers and nodules of chert common. The Short Creek Member is found near the top of the Burlington Keokuk Formation. The Short Creek is a persistent 3 to 10 ft. (1 to 3m) thick layer of oolitic limestone. The undifferentiated Pennsylvanian Carterville and Cherokee Member Shale found in the study area is the result of paleo karst activity. The shale "confused" or "broken" ground and consist of broken, angular chert lying on the slopes and bottom of the formerly solutioned, collapsed valleys. The chert is the residual component of the solutioned cherty limestone. It is within this porous, confused ground zone that most of the mineralization of the area has occurred. Areas of confused ground can extend nearly throughout the rock column of the project area, from the bedrock surface to over 115 ft. (35m) deep near the top of the Grand Falls Chert Member.

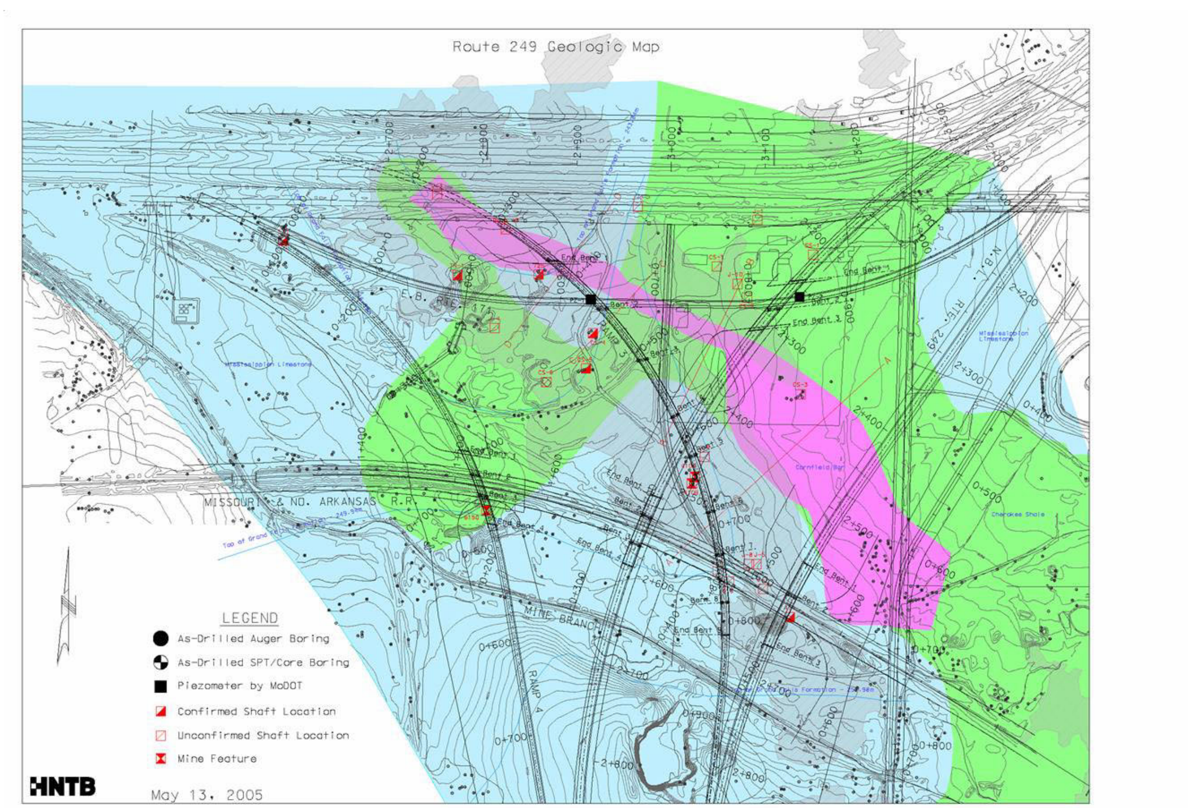


In addition, many areas of brecciated, intact rock were noted in the borings. These areas are known as "sheet" breccias and are characterized as chert crushed in place and recemented by chert. The brittle chert layers were broken in place by horizontal stresses while the more elastic limestone escaped brecciation. The stresses occurred as a result of minor faulting, solution adjustment, warping, and horizontal thrust. During mineralization, the chert was re-cemented.

The "Cornfield Bar" feature has a large influence on the project, controlling the location of the broken and confused ground as well as location of the shale bedrock. The confused ground reaches nearly down to the sheet ground in the area of the Bar, so called because it is barren of mineralization. The width of the bar varies from 50 to 300 ft (15 to 90m), with the location of the bar being in direct relationship to the location of the Cherokee Shale.



Geologic Sections Across Paleosink (Joplin District Folio, Smith and Siebenthal 1907)



Site Geology

The synclinal bar is characterized by flanking bedrock dipping into the trough. The trough filled with a confused mixture of limestone and chert mixed with weathered and broken shale. All this lies unconformably on the broken sheet ground. The bar is believed to have been formed by overlying strata slowly sinking into a solution cavity after Mississippian Time.

SITE INVESTIGATIONS

- **371 Borings were drilled for the project**

Between 1996 and 1998 during the initial mine study phase, MoDOT drilled borings near selected known or suspected mine features. Subsequently in 2000 and 2001 MoDOT forces drilled 152 sample and core borings plus additional pattern auger borings during the bridge preliminary design phase.

During the bridge final design phase in 2004, MoDOT forces drilled 134 sample and core borings plus additional pattern auger borings. Due to changes in the bridge configurations, additional borings were required at new substructure locations, box culverts and retaining walls. MoDOT Drill Crews drilled 8 additional borings for the bridges, 30 borings for the addition of 3 box culverts, and 24 borings for the addition of 3 retaining walls to the project in 2005. MoDOT forces utilized truck mounted Failing 1500, Mobil B - 31, CME – 45, and track mounted CME – 850 drill rigs to accomplish the drilling.

The HNTB exploration program consisted of 23 borings drilled during the period November 9, 2004 and January 17, 2005. . The borings were drilled under a subcontract with Boart Longyear of Wytheville, Virginia. Depths of borings ranged from 92.5 to 217.2 ft. (28.22 to 66.20m).

Drill Rig

Coring of the bedrock was accomplished with triple tube core barrels (double swivel tube with a set of split inner tubes). Three diameters of cores (PQ-3, HQ-3, NQ-3) were taken, with larger diameters starting at the surface, switching to smaller diameters if drilling difficulties were encountered. Many different types of impregnated bits with varying diamonds and matrices were used. Much of the coring was accomplished with little or no water return to the surface.

The drilling was observed and rock core logged in the field with items such as percent recovery, RQD, lithology, physical characteristics, drilling action, and drilling fluid loss noted and recorded. Additional structural geologic logs were also recorded and field point load testing accomplished at the core storage facility. The geologic structural logs further described items such as core loss, areas of RQD calculation, breaks per foot, rock type, color, weathering, grain size in general accordance with ISRM(1981). Description of and graphic log of discontinuities such as bedding planes, fractures and filling were also recorded. Point load test, both axial and diametral were performed generally at 10 ft. (3m) intervals throughout all testable core. All core was photographed.

Borehole videos, acoustic televiewer (ATV), and borehole caliper diameters, were also taken at selected locations. The work was performed under subcontract to Boart Longyear by Geological Logging Systems of Bluefield, Virginia. The borehole video produced movies of drill hole sidewalls in digital format available on DVD. The Acoustic Televiewer provided an orientated, full 360 degree view of the borehole sidewall, detecting not only the presence of fractures, but also the dip angles and direction of the fractures and bedding planes. The ATV produced a hard copy log of the borehole. The caliper tool measured the diameter of the borehole and produced a graphical record.

GEOTECHNICAL CHARACTERIZATION

Overview

The following general material types were present at the project site:

Overburden:

Chat: Loose gravel and sand, crushed limestone and chert fragments
Tailings: Loose silt and sand, localized deposits
Soil: Primarily medium stiff to stiff clay developed from bedrock weathering

Bedrock (with typical ISRM strength classification and typical fracture spacing)

Limestone: Strong to very strong, close to moderate spacing
Chert: Very strong to extremely strong, close to moderate spacing
Breccia: Medium strong to very strong, very close to close spacing
Shale: Extremely weak to very weak, very close to extremely close spacing
Sandstone: Weak to strong, very close to close spacing

The above tabulation underscores the inherent variability of the bedrock units.

Mining voids were known to be present in both the upper “confused” zone and the deeper “sheet” zone. Based on published information the upper zone voids were reported to be irregular and variable in shape and extent. Typical sizes were 50 to 165 ft. (15 to 50m) in horizontal dimension and 6 to 26 ft. (2 to 8m) in vertical dimension. The lower voids exhibit lateral continuity with horizontal dimensions greater than 325 ft. (100m) but with vertical dimension limited to less than about 10 ft. (3m).

Interpretation of Void Size

Core drilling intersected multiple voids in both the upper “confused” mining zone and the lower “sheet” mining zone. These void intersections included core loss zones, broken rock and very soft clay infilling. Previous studies showed that remote sensing methods (geophysics) were inadequate to positively determine the size and location of voids. Consequently a trio of indirect approaches was employed:

Literature: 50 to 100 ft. (15 to 30 m) (physical dimensions)

Sinkholes: 65 to 115 ft. (35 m) (surface expression)

Boreholes: 20 to 235 ft. (6 to 72 m) (inferred correlations)

Therefore, based on the above indirect evidence, it was concluded that a **horizontal** void continuity of 20 to 60 m should be used for foundation design purposes.

The drilling programs and borehole logging provided direct information on the vertical continuity of voids and mine features An analysis of this data indicated the upper mining voids had vertical continuity typically less than 16 ft (5m) although two features were intersected with apparent vertical continuities greater than 65 ft. (20m). For the lower mining horizon (sheet ground) the drilling indicated vertical void continuity in the range of 1.5 to 10 ft (0.5 to 3m). Two features were intersected with vertical continuities of about 20 ft (6m). For design purposes the respective **vertical** continuities for voids were:

- Upper “confused” zone: 15 ft (5m)
- Lower “sheet” zone: 10 ft (3m)

Mechanisms

It was recognized that both short and long term processes could affect the performance of bridge foundation units:

Short Term:

- Bearing failure due to imposed loading

Long Term:

- Ground loss into shafts or mine voids
  - erosion of shaft plugs

    - dissolution of limestone
    - roof cave
- Increased loading related to regional groundwater lowering

The long term processes were discounted based on either direct mitigation (e.g. permanent shaft closures), institutional controls (e.g. regional groundwater effects) or perceived low risk (dissolution and roof cave). Foundation design was therefore based on short term bearing failure for which two mechanisms were considered for analysis:

- A "punching" failure of a spread footing or micro pile group situated on a rigid stratum (e.g. limestone) overlying a compressible unit (void or mine infilling).

- A flexural "beam" bending of a rigid stratum (e.g. limestone) overlying a compressible unit (void or mine infilling).

The objective of the analyses was to determine the thickness and quality of rock mass required to support the foundation loads assuming the structures were located over voids with the horizontal continuity as developed above. The punching failure was analyzed using an equilibrium formula while the "beam" behavior was modeled using both an analytical solution and finite element stress analysis.

SELECTION OF FOUNDATION CONCEPTS AND TREATMENT

General

The highly variable nature of the geology required a systematic approach be implemented to reduce the risk associated with variability between assumed and actual ground conditions at any specific location. To reduce this risk, the project team developed a foundation design concept that incorporated the following basic principles:

1. The *exact nature* of the rock mass under each foundation would be identified and verified during production drilling.
2. At each foundation location productions holes would be drilled to verify the ground conditions and also to treat the ground to limit subsequent micropile grout takes and/or to improve the mechanical properties of the rock mass.
3. The actual foundation built at each location would be responsive to the information obtained from the production drilling.
4. No bent (or wall) would be built over the location of a mineshaft that had not been remediated in some definite fashion in advance.

Bridge Foundation Types

Multiple foundation types were evaluated including drilled shafts, H piles, spread footings, and micropiles. Drilled shaft foundations were not recommended due to the variability of rock conditions, presence of underground mining and anticipated difficulty and costs of advancing to significant depths within the chert layers. H piles were not recommended for support of heavy loads due to the variability of rock conditions, presence of underground mining, and the possible necessity of significant high cost predrilling. Thus either spread footings or micropiles were the preferred foundations for the support of each bent.

In evaluating the 23 bent foundation locations, two geotechnical/geostructural categories of foundation conditions were defined to simplify foundation design:

- **Ground Type 1 consisted of competent, non mined limestone extending to at least 130 ft (40m) below ground surface.**

- **Ground Type 2 consisted of all other conditions, included voided, collapsed, solutioned or highly fractured ground.**

Spread footings on rock were recommended at bent locations where competent limestone was shallow and geologic conditions were interpreted to be in an area of Ground Type 1.

Micropiles were recommended at bent locations where geologic conditions were interpreted to be of Ground Type 2.

Footings at each bent location would therefore either be cast on a spread footing keyed 6 inches minimum into competent limestone or be supported on micropiles. Micropile design would be in general accordance with FHWA “Micropile Design and Construction Guidelines”, Publication No. FHWA-SA-97-070.

The micropile permanent steel casing was designed to extend approximately 10 ft (3m) into rock based on the lowest interpreted elevation or to a minimum elevation as required for lateral stability with the bond zone designed below the bottom of casing. Bond zones below the cased length of 16 to 26 ft (5 to 8m) were foreseen in competent, strong limestone, and of 33 to 51 ft. (10 to 15.5m) in more chaotic horizons. Micropiles were designed to support axial compression loads in side friction along the bond zone length and lateral loads through a combination of battered piles and bending.

Four preconstruction “performance piles” were installed and tested to geotechnical bond failure or to at least twice the anticipated average bond stress to verify overall design assumptions prior to production piles being installed. A minimum of one production pile from each bent was selected for a proof testing to at least 120% of the design working load.

Production Exploration and Treatment Principles

As noted above, the exact nature of the rock mass under each foundation was verified during production drilling. These exploratory holes were also used to treat the ground to limit subsequent micropile grout takes and/or to improve the mechanical properties of the rock mass.

The general approach to exploring each bent location was uniform and consistent, but the foreseen amount and type of drilling and grouting at each bent was to variable based on the preproduction understanding of local foundation conditions (i.e. Ground Type 1 or Ground Type

2 as a base) and the footing geometry. This approach featured the concept of “intensity” of the treatment conducted at each bent prior to construction of the spread footings or micropiles; namely low, medium and high. Each exploratory hole was drilled vertically to the target depth. Core drilling was not specified but each hole had to be logged during drilling in accordance with automated Monitoring While Drilling (MWD) principles. Minimum hole diameter was set at 4 in. (100) mm in rock. The grout type was varied with the severity of the conditions.

Low Mobility Grout (LMG) was used in voided conditions (apertures greater than 4 in. (100 mm). High Mobility Grout (HMG) with or without sand was used in tighter ground conditions.

Based on the ground water chemistry, Type II cement was recommended for use in both the HMG and LMG grouts. LMG was specified to have a low slump (less than 5 in.), high internal friction and 28 day strength in excess of 600 psi (4 MPa). The HMG was specified to have a Marsh Cone Viscosity of 40-50 seconds, be stable and have 28 day strength in excess of 600 psi (4 MPa). In the event of excessive take in any one location, it was foreseen that sand would be added to the HMG or the viscosity would be modified.

Depending on the actual conditions found in the field at each bent the level of treatment could escalate i.e. additional holes might be required to explore and treat the bent. Conversely, it was not anticipated that there would be cause to reduce the intensity of treatment at any bent under this program.

It was anticipated that ascending stage grouting principles could be used for both the LMG and HMG operations. However, particularly severe ground conditions could require downstage grouting. For bidding purposes it was estimated that up to 25% of the drilling would require downstage grouting due to variable rock quality.

The plans and specifications included the foreseen ground treatment program and quantities for each bent location. Modifications to the treatment program and foundation construction were made primarily in the field based on actual field conditions encountered and on the judgment of the monitoring and design personnel.

MSE RETAINING WALL FOUNDATION EXPLORATION and TREATMENT

At three locations, Mechanically Stabilized Earth (MSE) walls were selected to reduce the overall bridge lengths and associated costs. MSE walls were selected as they are considered the most economical and can accommodate a variety of subsurface conditions. It was recommended that maximum wall heights be kept to about 30 ft. (9m).

The major foundation concern for these structures was the potential for loss of ground into, and loss of support from, underlying voids or mine shafts. As for the bents, all known or suspected shafts were to be pretreated.

It was recommended that the top of leveling pad and the bottom of wall be exposed and inspected first to verify the actual foundation material conditions. As Type 2 ground conditions were assumed, a line of primary treatment holes at 6 meter centers was drilled to a depth of 30 meters and treated with LMG if Type 2 conditions were found or inferred.

It was noted that depending on the local conditions, additional “closure”, or secondary treatment holes may be needed. With respect to grouting, this phase of exploration and treatment was intended to locate and fill shafts or other major voids. Systematic fissure grouting was not deemed necessary, and so only LMG was specified.

Additionally, one treatment hole approximately every 118 ft^2 (36 m^2) was planned to be drilled to a maximum depth of 65 ft (20m) below ground surface in a regular pattern under the footprint of the reinforced mass for the MSE wall if Type 2 conditions were found or inferred. These treatment holes would be treated with LMG. It was recommended that should significant features be encountered, additional “closure”, or secondary treatment holes may be needed.

BOX CULVERT FOUNDATION EXPLORATION and TREATMENT

Three box culverts were planned at the southern portion of the project. The box culverts will carry the Mine Branch Creek below the approach roadway embankments of Rte. 249 NBL, Rte. 249 SBL, Ramp 3 and Ramp 4.

The major foundation concern for these structures is the potential for loss of ground into, and loss of support from, underlying voids or mine shafts. As for the bents and walls, all known or suspected shafts were pretreated.

Even though Type 1 ground conditions were foreseen, it was recommended that one primary treatment hole approximately every 65 ft^2 (36 m^2) be drilled to a maximum depth of 65 ft. (20m) in a regular pattern under the footprint of the box foundation slab. These holes were treated with LMG. It was noted that depending on the local conditions, additional “closure”, or secondary treatment holes may be needed. With respect to grouting, this phase of exploration and treatment was intended to locate and fill shafts or other major voids. Systematic fissure grouting was not deemed necessary, and so only LMG was foreseen.

MINE SHAFT/OPEN FEATURE CLOSURE PHILOSOPHY

The general philosophy was that each known or suspected mine shaft or open feature location on the site should be explored and treated in addition to the specific actions to be conducted at individual bent, MSE wall, and box culvert locations. The locations of the major structural elements of the project including the bridge foundations, MSE walls, and box culverts had already been adjusted during preliminary design iterations to avoid known mine shafts or open features. It was noted that several suspected features were located on MNA Railroad property.

Two categories of mine shafts/open features were defined based on their proximity to the proposed structures and grading limits. Each had a different treatment method and intensity:

"Type 1 Shaft" - Mine shafts/open features located within approximately 50 ft (15m) of a major structural element of the project or within approximately 15 ft (5m) of the footprint of an embankment or cut footprint.

"Type 2 Shaft" - Mine shafts/open features located beyond approximately 50 ft (15m) of a major structural element of the project and beyond approximately 15 ft (5m) of an embankment or cut footprint.

Based on review of mining maps, literature, site drilling and reconnaissance, some 26 potential mine shaft/open feature locations were identified within the proposed interchange. A number of locations were investigated further by performing test pit excavations. As a result, seven potential feature locations showed no indication of a shaft; therefore, no further investigation or treatment was recommended at these locations. One mine feature location, J-7, was located beneath the MNA Railroad tracks and no investigation or treatment by MoDOT was recommended. Of the remaining 18 feature locations, six locations were confirmed as mine shafts either open or previously plugged by the Missouri Department of Natural Resources (MDNR). These six shafts were recommended to be closed either as Type 1 or 2 closures.

Twelve unconfirmed shaft locations were identified for investigation and possible closure. Exploratory inspection excavations were recommended at these locations to determine if mine features are present. If so, closure would be required either by Type 1 or 2 procedures. The actual number of Type 1 and 2 closures would therefore be determined during the exploration phase.

It was noted that the excavations for the exploration and closure of mine features might encounter groundwater and that excavations on or near the railroad right-of-way may necessitate the use of temporary shoring to control the excavation and maintain the railroad tracks.

For Type 1 shafts, the recommended treatment involved full penetration by drilling with an initial treatment hole to confirm the shaft base elevation, filling it with LMG and verification of thoroughness of treatment by a minimum of 2 additional treatment holes. These additional treatment holes would be within the mine shaft limits.

Therefore, each Type 1 shaft would require a minimum of three treatment holes drilled to the bottom of the feature. It was noted that the drilling might encounter obstructions and/or other complexities in the backfill such as timber, metal, concrete, reinforcing steel, etc.

Regarding the LMG volume, it was anticipated that not all the shaft space was void, but that there would be workings leading off the shaft which may still be open, filled, or collapsed.

In addition, wherever Type 1 shafts were encountered, it was recommended that at least three vertical treatment holes shall be drilled at 10 ft (3 m) centers to a depth of 100 ft (30m) in a line running transverse to the direction of the shaft and any structure that was within the critical distance. These treatment holes were intended to verify that no open shallow lateral workings still existed between the shafts and the interchange structures.

It was recommended that these activities should be conducted under the utmost safety standards. Drilling equipment should operate from either frames/platforms or be vertically suspended from remotely located leads. Prior excavation to top of rock would provide visual evidence of the in situ geometry of the feature, indication of the required of safety measures, anticipated quantities of LMG, and the precise location of the treatment holes.

Type 2 shaft closure involved partial excavation to top of rock, temporarily plugging the throat with polyurethane foam, and then casting a reinforced concrete plug over the top.

CONSTRUCTION

PHYLOSOPHY

One of the conclusions of the geologic/geotech investigation was the chaotic and "confused" nature of the subsurface at the site. During the course of several years, many borings were taken in an attempt to characterize the site. The characteristics of the subsurface were known to change drastically between boreholes located less than a meter apart. Drilling additional holes during the design phase, might not provide further useful design information. Therefore, during design, the subsurface was classified into zones of ground type. The subsurface characterization, as well as the design of ground improvements would be continued during the construction phase by drilling and treating the encountered mine voids and highly disturbed ground. The subsurface would be logged at each drill hole and treatment recommendations made in real time.

The production drilling equipment would include the use of monitor while drilling (MWD) as well as the real time observation of drilling and logging of the hole by a geologist or geotechnical engineer employed by the engineer. Grouting, both low mobility and high mobility would also be electronically recorded and monitored by the field personnel.

The selection of ground treatment type was based on actual subsurface conditions encountered at hundreds of production holes rather than a few exploration holes taken during design. Low mobility grout (LMG) was used in voided conditions, all areas of mass ground treatment, and for closure of mine shafts. High mobility grout (HMG) was used in the fractured rock and foundation treatment to limit the use of grout during micropile installation. The use of real time observation was used successfully to modify the ground treatment and micropile installation in a seamless effort.

CONTRACTING

The project originally consisted of two contracts to construct the interchange. The first was to include ground investigation, treatment, and installation of micropile and spread footing foundations prior to construction of the five bridges.

The second contract would construct the embankments, bridge structures above the footings, retaining walls, and box structures. The second contract was expected to commence within a few months of the first, with the work then proceeding simultaneously with the cooperation of both contractors.

Unfortunately, MoDOT received only one bid for the original foundation contract. The lone bid exceeded the engineers estimate and was recommended for rejection. Subsequently, contract and plan adjustments were made to attract additional bidders. In order to maintain schedule and spread risk to a general contractor, the contract now would be bid a second time as a combined bid.

A couple of months later, the contract was let as a combination project - foundation activity along with the embankment and structures. The job special provisions maintained the requirement the ground treatment contractors be pre-approved. This pre-approval process provided prospective prime-contractor bidders with a list of three ground treatment contractors. APAC Missouri was the successful prime contractor, constructing the structures and providing excavation at the mine shafts and footings. Layne Geo-Construction served as a subcontractor to APAC performing the ground treatment and micropile activity.

HNTB was responsible for the construction engineering and inspection of the project with respect to the ground treatment project and micropile installation only. HNTB also provided documentation to MoDOT for daily pay quantities and inspector diary information. The contract specified all contractor correspondence to be provided as RFI's. The RFI's were submitted through the prime contractor and then to MoDOT. MoDOT entered the information into a website database that HNTB accessed and responded to any formal request.

PREQUALIFICATION

Early in the project, the design team noted that a highly experienced specialty foundation contractor was required for the project, mainly due to:

- the site being extremely complex with highly variable geology;
- the methods being proposed to investigate and treat the foundation rock at each major structural location, although well-defined in principle, required interactive implementation during construction;
- the ground required a high level of construction expertise, flexibility and responsiveness. The contractor would need to able to react to variations in drilling and grouting quantities from those foreseen, and for large variability between adjacent locations;
- the extensive scale of the site and time constraints on the project.

The mechanism for selecting the most suitable and qualified specialty foundation contractor merited special attention. It was understood that restrictions imposed by State Law made it not feasible to use “Best Value” selection means. Therefore, a pre-bid prequalification process was selected.

The pre-qualification process required potential bidders to review provisional contract documents (approximately 80%) and submit a Statement of Qualification. The Statement of Qualifications was used by the Commission to determine prospective bidders that were capable of performing the specialized work that was necessary for this project.

The Commission would not accept bids from any prospective bidder that had not been pre-qualified through this process. Attendance by prospective bidders at the pre-bid conference held on site was mandatory for a contractor to be pre-qualified

QA - INSPECTION

The QA/QC requirements from the specifications are summarized in Table 1.

ITEM/ACTIVITY	QA/QC	PURPOSE
Drilling	• Verticality, location and depth.	Holes in intended plan location.
	• MWD (i.e., real time monitoring and recording of major drilling information, e.g., penetration rate, strata changes, drill actions, flush characteristics, hole stability.	Given the variability of the site, each hole will act as an investigation to reduce geological uncertainty and demonstrate improvement of the ground at each location during treatment. This will also confirm the suitable bearing horizon for the micropiles.
Grout Materials	• LMG – slump, cube strength • HMG – bleed, s.g., Marsh Cone, cube strength.	Ensure accurate and consistent batching and proportioning.
Grouting Process	• For all grout injections the following parameters will be recorded: pressure, rate of injection, volume (per stage per unit length). Also recorded will be observations on interhole connections, breakouts, pressure irregularities, delays.	Data will permit the team to analyze incremental performance of each phase of grouting (i.e., via use of Reduction Ratios, etc.) leading to logical decisions as to intensity of treatment (e.g., more or fewer holes, mix type selection).
Micropile Capacity	• Minimum three "Performance Tests" prior to construction in the different ground types. • Minimum one "Proof Test" per bent on productions piles.	Performance tests will confirm basic design assumptions. Proof tests will demonstrate consistency of installation quality of each main structural location.

Analysis of these data in real-time was particularly important on this project to assure that a responsive treatment was provided at each structure location, notwithstanding the provisions of the Specifications.

The successful implementation of this concept required: The full engineering cooperation between the owner's representative (HNTB) and the specialty foundation contractor (Layne); and the on-site presence, guidance and participation of the owner's representative (HNTB).

HNTB provided a team for the construction engineering and inspection which consisted of a resident engineer and an inspector (geologist or engineer) for each drilling or grouting operation. The scale of the operations required a staff of a resident and four to five inspectors. Layne normally ran two or three rigs drilling and two rigs grouting.

Prior to the drilling of any hole, the field inspector responsible for logging the hole reviewed the Geotechnical Baseline Report (GBR) information to determine the expected elevations of rock-head and features within the hole. Part of the initial site set-up involved ensuring that all the relevant information was available onsite in an easy to search format to allow the inspectors to easily find this information.

The holes were logged during drilling by an HNTB inspector independently of Layne. Once the hole was drilled, logs completed by HNTB and Layne were reviewed and compared to the design intent of the plans and specifications as well as the GBR and Geotechnical Design Report. The automatic parameter recorder data submitted by Layne was checked to ensure that the automatic parameter recorder data and the manual log were consistent.

The holes were drilled with a down-hole-hammer, thus, the ground types logged were limited to match the sensitivity of the drilling system. The comments section contained additional information. The person logging the hole was trying to determine if the drill was penetrating material from one of the following categories:

- overburden;
- shale;
- chaotic, poor quality limestone, chert breccia;
- hard competent limestone;
- void/filled feature.

The log contained space for instantaneous penetration rates, flush comments and general comments.

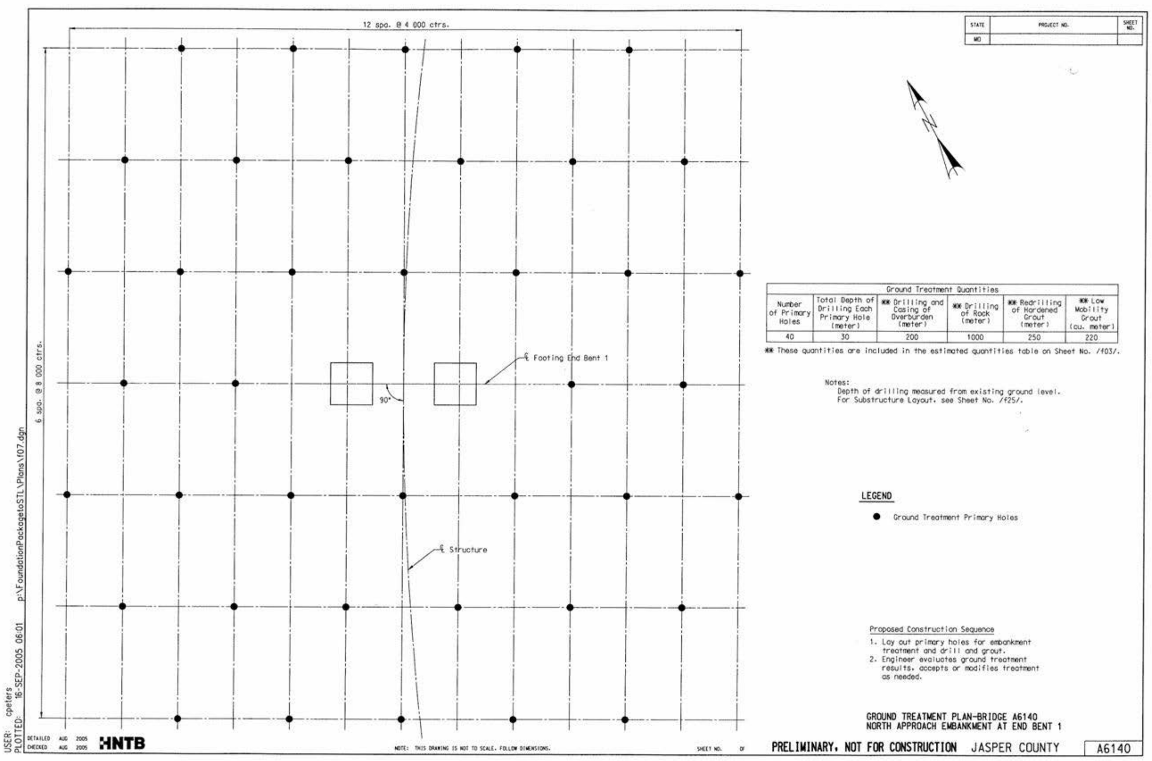
CONSTRUCTION PROCEDURES

Construction began with the installation of the four design verification piles. The pile locations were placed in the three previously identified types of ground plus treated ground. The verification piles were placed in good limestone, broken and confused ground, treated broken and confused ground and shale.

GROUND TREATMENT

Mass ground treatment was undertaken in the areas where formerly mined (both shallow and deep) ground were thought to exist. The purpose of the mass treatment was to reduce the risk of ground loss under bridge approach embankments and in previously identified poor ground in the vicinity of bridge foundations.

The treatment consisted of a pattern of holes generally 13 by 13 ft.(4 by 4m) to a designated depth. The holes were drilled with down the hole hammers. Holes were logged in real time by Jean Lutz monitor while drilling (MWD) electronic apparatus. The MWD system provided advance rate, thrust pressure and rod torque. The holes were also logged real time in the field by the inspectors.



Typical Foundation Treatment Grid

The contractor's MWD data and the inspectors field logs were then compared and a grouting treatment specified. The mass ground treatment holes were all grouted with LMG. The intent of the mass ground treatment was to explore and fill mine voids.

The LMG consisted of a contractor designed mixture of sand, cement, fly ash, additives and water. LMG grout strength was specified as 28 day strength of 600 psi (4 MPa) with a slump of 5 in. (127mm) or less. Several modifications in mix design were necessary at the beginning of the project to achieve the project strength and slump criteria as well as the pumpability and set time required. Type C instead of type F fly ash was allowed due to material availability.

The LMG was furnished by a local concrete batch plant and brought to the site in transit trucks normally carrying between 6 to 8 yd³ (4.5 to 6.1 m³). The grout was pumped with standard concreted pumps in lifts of 3.3 ft (1m) to a refusal criteria of 600 psi (4 MPa) at the drill rig.

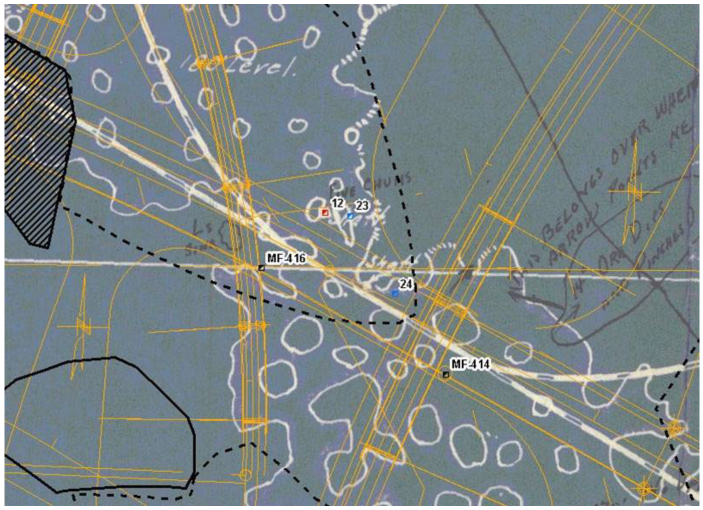
Typical Ground Treatment Process – One rig drilling, one grouting

The main purpose of the ground treatment program was to explore for mine voids and reduce the risk of collapse. The holes normally ranged in depth from 65 to 100 ft. (20 to 30m). LMG takes for holes without voids were normally less than a 1 to 2 cubic yards. When mine voids were encountered, grout takes ranged from 6 to 307 cubic yards (5 to 235 cubic meters). In holes with very large takes, the holes were allowed to rest for approximately two hours if 50 cubic yards were placed without achieving refusal criteria in a lift. The resting time and grout rate were varied and made at the discretion of the rig inspector. In almost all cases, the rest period resulted in achieving the refusal criteria with addition of smaller amounts of LMG.

MINE SHAFT REMEDIATION

Several vertical mine shafts were also on the project right of way. None of these shafts were open to the surface prior to construction. A few of the shafts were evident from observed surface expressions and were excavated in the exploration phase and included in the contract documents. Other shaft locations were taken from mine maps obtained from historic sources. The contract documents included multiple suspected shaft locations that were to be explored by backhoe during the construction phase. The shaft exploration cost was based on measured volumes of material excavated. Some of the listed shafts were located, some were not, and other shafts not anticipated by surface expression or mine maps were found during the site grading.

Due to the possibility of encountering open shafts and ground collapse, a crane was specified to be placed in the vicinity of each work area for worker safety.



Historic mine maps overlaid with roadway layout.

Open Mine Shaft

Grouting Mine Shaft

TYPE 1 MINE SHAFT CLOSURE

Once a shaft was located, the area was excavated generally to top of rock with backslopes for a safe temporary work area. The contractor then placed timber crane mats over the shafts and drill rigs placed to access the vertical shaft. Mine shaft drilling was a separate contract item due to the inherent possibility of encountering a variety of possible materials which over history may have been placed in the shafts. The shaft were typically used as a local solid waste disposal site and could be filled with nearly any type and size of material. The shafts were generally 5ft (1.5m) square.

Once the drill rig was over the shaft, the down the hole hammer was taken to elevations thought to be the previous mine floor or to penetration of several feet into material which appeared solid. Some of the shafts were necked off with several feet of miscellaneous fill and then water filled, while other shafts were filled with miscellaneous mostly soft fill to the bottom.

After drilling, the rods were withdrawn and grout casing placed into the hole. Low mobility grout was placed in 5 ft stages to the project refusal criteria. Again, the amount of grout placed and any periods of rest time were placed at the discretion of the inspector.

In addition to the first hole, two additional confirmation holes were placed one to two meters from the original location. The purpose of these secondary holes was to confirm the original grout placement and explore for stopes and adits which may have occurred off the vertical shaft.

In addition to the shaft grouting, a series of three additional confirmation holes were located approximately 33 ft (10m) from the shaft in the direction of any nearby adjacent structure. Again these additional holes were designed to explore for any possible stopes or adits emanating off the main shaft.

A total of 12 shafts were found and remeditated using this method. The amount of LMG need to close a shaft ranged from 4 to 409 cubic yards (3 to 313 cubic meters).

TYPE 2 MINE CLOSURE

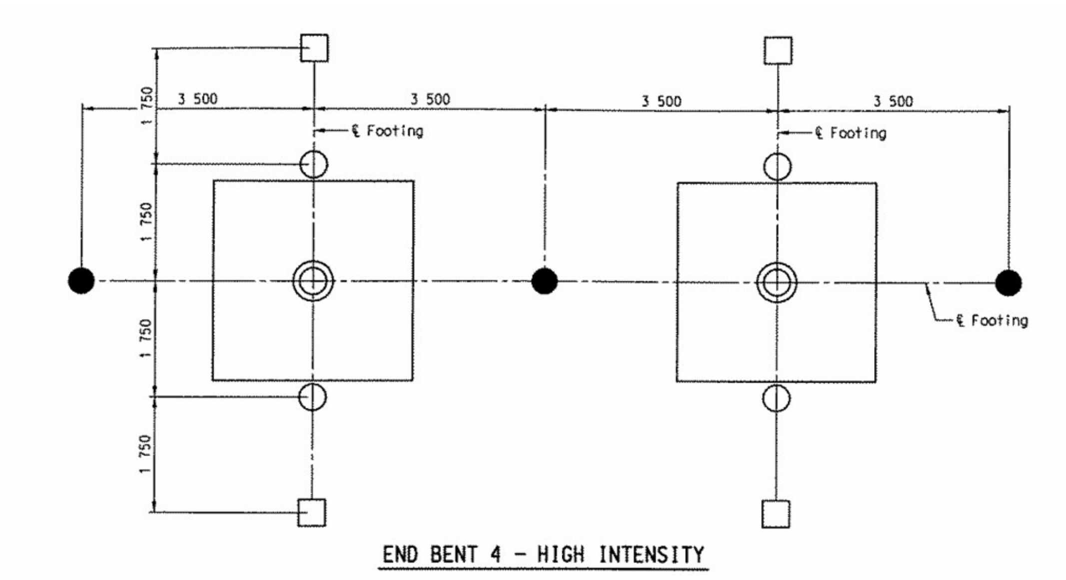
Another type of shaft closure was designed to be employed at shaft locations on the right of way but not near any bridge structure. The purpose of these shaft closures was long term site safety.

These closures were known as Type 2 closures and consisted of excavating the area of the shaft to top of rock and placing a plug of expanded polyurethane foam in the throat of the shaft and then placing an inverted cone of cast in place reinforced concrete to seal the opening.

FOUNDATION TREATMENT

The area surrounding each foundation unit was excavated to bottom of footing elevation and inspected for signs of any mining activity. Then a series of holes was laid out surrounding and covering the footing area. These holes were also drilled with a down the hole hammer to depths ranging from 43 to 180 ft (13 to 55m). Again, the holes were both logged by MWD and the rig inspectors.

Based on type of ground anticipated, three levels of foundation treatment intensity were specified in the plans, low, medium, and high. The low intensity averaged three primary and two secondary holes for a two footing bent. The medium intensity averaged three primary, two secondary, and four tertiary holes for a two footing bent. The high intensity treatment averaged three primary, two secondary, four tertiary, and four quaternary holes per two footing bent.



Typical High Intensity Foundation Treatment Layout

After reviewing the drilling logs, a treatment scheme was chosen based on the character of the rock and the number and size of any voids logged in the drill hole. The purpose of the foundation treatment was two fold, the first to look for mining voids and unstable ground, the second to reduce the amount of grout needed for installation of the micropiles. In general, voids larger than 6 inches (152mm) were desired to be treated with low mobility grout while broken and fractured rock was to be treated with high mobility grout.

The high mobility grout consisted of fluid grout designed by the contractor and composed of cement, fly ash, bentonite and additives. There were three different grout mixes based on viscosity, A, B, and C. The grout was mixed at a central automated grout plant (Tecniwell TM 30). The grout mixes ranged from a marsh cone of 40 to infinite. HMG was required to achieve a 28 day strength of 600 psi (4 MPa). The high mobility grout was installed using a packer in 10 ft. (3m) meter stages to project refusal criteria of flow verses time.

Prior to proceeding with micropile installation, the information from the entire group of foundation treatment holes was plotted, analyzed and compared to the depths, thicknesses and types rock materials assumed in the design of the micropiles. At this point the casing and micropile bond lengths were adjusted to match the actual conditions gathered during the foundation treatment. The casing and micropiles were generally adjusted as a group at a single bent footing rather than on a pile by pile basis.

Using HMG to Control Groundwater

MICROPILES

The micropiles consisted of a cased and bonded length. In the case of piles designed with lateral loading, the cased length consisted of a single piece of steel casing, 7.625 in (193.7mm) OD, N80 Mill Secondary steel pipe. The casing tensile strength was 80 ksi with a minimum wall thickness of 0.5 in (13mm). In the case of piles without lateral design load, threaded joints were allowed. The pile steel reinforcement consisted of an epoxy coated 2.5 in (65mm) OD, grade 150 KSI thread bar.

Installation of the micropiles generally enlisted two methods; (1) drill, install casing, drill, install reinforcing bar method and (2) drill and advance casing to bottom of hole, install reinforcing, grout and withdraw casing to plan depth.

In the first method, a 9.625 in (245mm) bit and down the hole hammer was used to drill to the planned bottom of casing. The hole was logged to obtain the desired penetration into rock specified by the design. The rig inspectors logged the drill hole and adjusted the depths accordingly. The casing was then capped and grout pumped down the casing and up the annulus until undiluted grout returned to surface.

After the casing was grouted and allowed to set for one day, the bond area was drilled through the grouted casing and into the bedrock. The bond zone was drilled with a 6.25 in (160mm) bit and down the hole hammer. Again, the rig inspector logged the hole for agreement with the bent specific characteristics intended in the micropile design. The micropile was either installed per plan or lengthened accordingly. The micropiles were never shortened due to better than expected conditions.

After drilling the bond zone, the threaded bar reinforcing was installed. The bar was made up of stock 10 and 20 ft (3 and 6 m) lengths and cut to final grade. The bars were joined with mechanical threaded couplers. PVC centralizers were placed on the bar at approximately 10 ft (3m).

A grout tremie tube was attached to the bar before it was placed in the hole drilled for the bond zone. Grout was then pumped until undiluted grout returned to surface. Nearly all micropiles installed were below the local groundwater table.

One micropile was selected for proof testing at each bent. Normally, the selection was based on a possible anomaly observed during the installation of the pile. Some of the factors may be high grout takes, low grout takes, hole instability, or difficulty inserting the bar.

Micropile Proof Test

CONCLUSIONS

The system of gathering and inspecting information from the drilling and grouting in real time reduced the risk associated with design of ground improvements and micropile installation in a very complex geologic and mined environment. The system helped control quantities on the project and allowed for adjustments to all aspects of the grouting and micropile construction without interrupting the work process.

Item	Plan Quantity	Actual Quantity	Over Under	Percent of Plan
Type 2 Mine Closure	3 ea	1 ea	-2 ea	33%
Mine Shaft Drilling	2503 ft 763 m	2408 ft 734 m	-95 ft -29 m	96%
Micropile Proof Load Test	16 ea	16 ea	0 ea	100%
Micropile Verification Test	4 ea	4 ea	0 ea	100%
Overburden Drilling	5630 ft 1716 m	7286 ft 2221 m	+1656 ft +505 m	129%
Rock Drilling	53855 ft 16415 m	48442 ft 14765 m	-5413 ft -1650 m	90%
Micropile Bond Length	6614 ft 2016 m	6844 ft 2086 m	+230 ft +70 m	103%
Micropile Cased Length	4072 ft 1241 m	4295 ft 1309 m	+223 ft +68 m	105%
Redrill	13474 ft 4107 m	2812 ft 857 m	-10662 ft -3250 m	21%
High Mobility Grout	752 yds^3 575 m^3	504 yd^3 385 m^3	-248 yd^3 - 190 m^3	67%
Low Mobility Grout	4756 yds^3 3636 m^3	8698 yd^3 6650 m^3	+3943 yd^3 +3015 m^3	183%

The planned contract quantities were originally estimated taking into consideration the conditions of the site as a whole, not each footing. In the end, the quantities varied greatly from hole to hole and bent to bent as the subsurface conditions were truly chaotic and confused. However, when the highly variable quantities were applied to unit costs, the total cost was within four percent of the original estimate.

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New Rockfall Drapery System for Rockfall Mitigation
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Disclaimer

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ABSTRACT

This paper illustrates the use of a new hybrid rockfall mitigation system for drapery. This new product was used by the Pennsylvania Department of Transportation on SR 79 near Pittsburgh, Pennsylvania as a rockfall mitigation system. The system consists of PVC coated double twisted mesh with steel cable integrated within the mesh during the manufacturing process. The incorporation of steel cable within the wire mesh provides multiple advantages in the installation and in the product characteristics. For a better integration with the rock, the wire mesh was coated with a black polymeric barrier. The design for the project was carried on using the mechanical characteristics of this new drapery system. In the project, the maximum height of the rock slope protection was 120 feet high with a total surface area of 40,000 square yards.

Intensive laboratory tests were done to demonstrate the performance of this new system: mesh tensile strength, junction strength between cable mesh, cable sleeve connection, and punch test. The improvement of this new system over the specified product was not only related to the strength of the product, but also on the cost savings during installation.

INTRODUCTION

With the improvement of an existing portion of the State Route 79 (SR 79) located in Collier and Robinson Townships, Allegheny County, Pennsylvania, a rockfall drape netting intervention was required in three distinct rock slope areas of the project: located along SR 79 between Exit 59 (i.e., State Route 279) and Exit 57 (i.e., Carnegie, Pennsylvania). See Figure 3.

The Project is about 10 miles Southwest of Pittsburgh, Pennsylvania (see Figure 2), and resides within the Appalachian Plateau, which is situated West of the Appalachian Mountain range/chain. In this region, the rock mass is generally made up of horizontal beds/layers of slightly weathered, laminated, fine-medium grain sandstone and shale of the Casselmen Formation and Conemaugh Group (e.g., Pennsylvania Period, or about 320 to 290 million years ago).

Figure 1 -Photo of I79 Southbound before construction.

The owner of the project is Pennsylvania Department of Transportation (PennDOT); the contractor was Trumbull Corporation; the engineer Golder Associates, Inc.; the material was manufactured and supplied by Maccaferri, Inc.

The installation of the rockfall netting began in late spring 2006 and was completed in early fall 2006.

Existing Condition

The rockfall mitigation works consisted of three sections:

Southbound Slope

This area is located between roadway stations 322+50.00 to Sta. 349+63.60 on the left side of the road and SR-8007-Ramp "C" Sta. 36+25.00 to Sta. 41+51.94, Right (RT). This section is about 3,241 linear feet (988 m) in length. Slope heights generally vary from about 40 to 160 feet (12.2 m to 48.8 m).

Northbound Slope

This area is located between northbound roadway stations 329+72.00 and 329+94.00 on the right side with a length of 22 linear feet (6.7m). Slope heights generally vary from about 30 to 90 feet (9.15 m to 27.4 m).

Ramp "B" Slope

This section is located on the southbound of SR-0079 between Sta. 413+10.00 to Sta. 419+19.77 on the left and SR-8009-Ramp "B" Sta. 51+25.00 to Sta. 53+96.98 on the right for about 882 linear feet (269 m)in length. The slope heights generally vary from about 30 to 80 feet 9.15 m to 24.4).

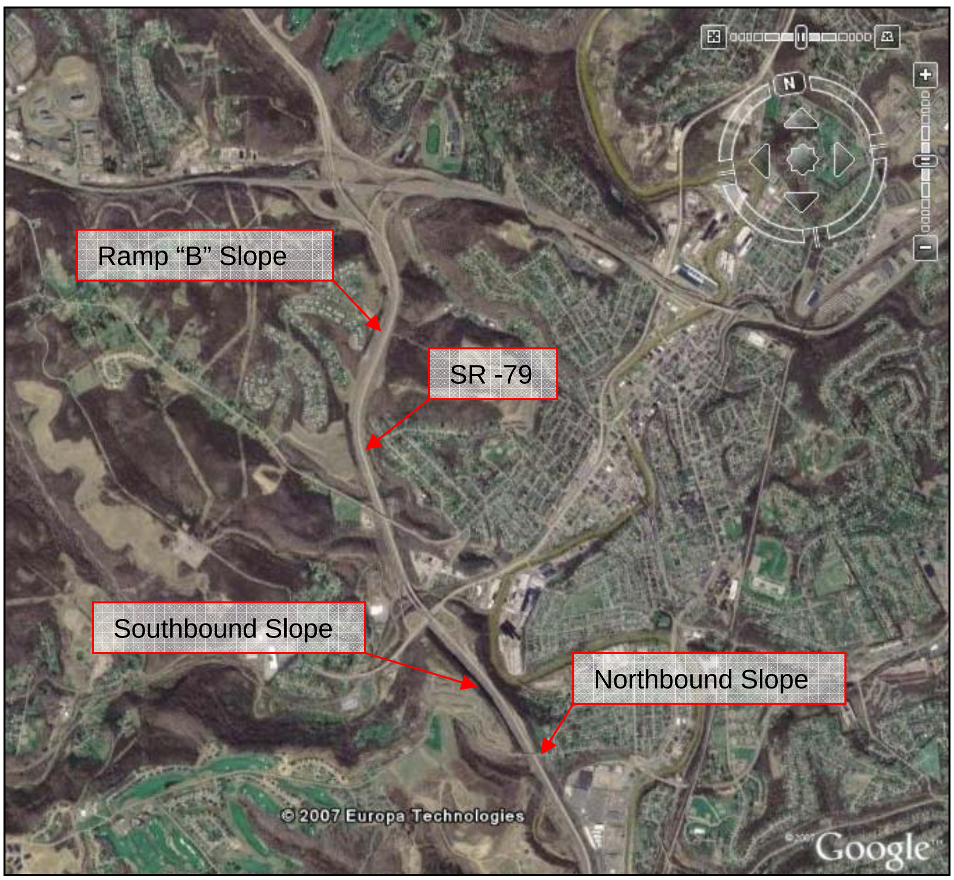

Figure 2 – Slope Treatment Location.

The total surface of rockfall drape was 40,000 square yards with a height varying from 30 to 120 feet (9.15 to 36.6 m).

During the site visit by (3) Golder Associates, Inc., the following conditions were observed on the rockfall treatment areas:

- The rock faces for all three areas are parallel to the roadway alignment, generally trend North-to-South, and are steeply dipping between 65 to 80 degrees from horizontal.

- A steeply dipping (75 to 85 degrees) systematic joint set intersects at nearly 90 degrees forming cubic rock blocks of varying size (upwards of about 3 cubic-feet). The joint spacing appears to vary between about 6 and 36 inches (150 to 914 mm) while some may occur closer or farther apart in some areas.

- Weathering of thinly laminated fine-grained rocks create raveling conditions, which results in rock fragments with particle sizes of about 1 to 2 inches (25 to 50 mm).

- Differential weathering between thinly laminated, fine-grained and medium grained rocks results in an undercutting condition of the bedding within the rock mass.

- Each of the three designated rock slopes has catchment areas at the toe-of slope, and these catchment areas appear to be offset about 10 to 20 feet (3 to 6 m) from the adjacent slope toes.

- During their site visit, rock blocks were observed within the catchment areas for each of the three designated rock slopes, as defined herein, which indicates these slopes are currently subject to active rockfall events.

- The designated rock slopes were partially covered with snow and ice.

Selection of Material

The initial bid document was asking for an alloyed high strength carbon steel wire with a minimum strength of 256,000 psi (1765 MPa) and coated with Galfan®. Mesh construction was in a single twist diamond form with a twisted loop at the end. The mesh was required to be colored to match the existing rock.

As an alternative product, Maccaferri contacted Trumbull Corporation to offer a functional equivalent product made of steel wire mesh and steel cables. The Rock Mesh B900 was proposed as an alternative for the SR 79 project. This product consists of a PVC coated double twist steel wire mesh with steel cable of 5/16 (8 mm) inserted within the mesh during the weaving process. The steel cables are inserted every 2 feet (0.6 m) in the mesh direction and 3 feet (0.9 m)in the cross direction. The transversal cables are secured at both ends with aluminum sleeves. The PVC coated wire is available in multiple colors. (See Figure 3)

Figure 3 –Rock Mesh B900 with Brown PVC under manufacturing process.

Product Testing Performance

For simple drapery, the extra strength of the wire mesh fabric is provided by the tensile strength of the longitudinal cables spaced at every 2 feet (0.6 m); each cable had a tensile strength of 9800 lb 43.6 kN. This hybrid product has cable spacing 2 by 3 feet (0.6 by 0.9 m) and with steel wire mesh within the cable mesh opening. In comparison, this hybrid system is stronger than standard double twist wire mesh.

The cables are connected together using the wire twist mesh during the weaving process. As per CTC *(1)* lab test in February 2005, the connection strength of the steel cable within the mesh was 54.3 kN/m for breakage. This is 2.5 times the minimum connection strength of 1400 lb/ft (20.4 kN/m) as required by ASTM A975 for connecting two double twist wire mesh panels together. In 2005, the Construction Technology Institute – National Research Council in Italy had tested a certain number of connection systems for cable panels. The test results published in Maccaferri Literature *(4)* had demonstrated that the strength of the connection with steel clips varied from 103 lb (4.6 kN) and 303 lb (13.5 kN) for a cable panel of 8 mm diameter. If we extrapolate for comparison, a cable netting panel with a mesh opening of 12 by 12 inches (30 by 30 cm), the panel will have a connection strength between 946 lb/ft (13.8 kN/m) and 2775 lb/ft (40.5 kN/m). This is lower than the strength achieved by the Rock Mesh B900 at 3720 lb/ft (54.3 kN/m).

Figure 4 – Rock Mesh test at Bathurst *(1),* Clarabut Geotechnical Testing, Inc.

Other tests were performed to demonstrate that the mesh would not break before the cable got in tension. The test was carried out by Bathurst *(1),* Clarabut Geotechnical Testing, Inc. in 2005. With 2 cables inserted at 2 feet (0.6 m) spacing, both vertically and horizontally, in all the tests the cable was breaking before the mesh.

Design consideration

The project specification required a stamped design by an Engineer registered in the state of Pennsylvania. Maccaferri had retained the service of Golder Associates, Inc. to perform the design. The design was done in accordance with industry standards and PennDOT's initial drawings, by considering mesh weight, rock size and existing site conditions. The design required a product that was stronger than the regular double twisted mesh; the engineer selected Rock Mesh B900 as the solution for the project.

Installation Procedure

The rockfall drape nets were installed across all exposed rock slope surfaces within the three designated slope areas defined on the drawings with the following procedures:

Removal of Vegetation

All vegetation and loose debris, if any, was removed from the rock slope face area. The vegetation located within ten feet of the rock slope crests was also removed.

Rock Scaling

All exposed rock slope surfaces within the three designated slope areas were scaled to remove all loose, unstable rock blocks/fragments, which could potentially move down-slope and represent significant future rockfall hazards. The scaling was done manually with a nacelle using hand tools.

Rockfall Netting Installation

- The Rock Mesh B900 rolls were made 12 feet (3.66 m) wide with customized lengths for each section. Each roll was numbered with an assigned location to increase the productivity during the installation. The length of the rolls were from 120 to 230 feet (36.6 to 70.1 m) long with multiple sections of panels per roll for a total of 220 rolls.

- The wire mesh color selected by PennDOT was black PVC.

- The Rock Mesh was installed with all adjacent panel sections connected together using lacing cables 5/16 inch (8 mm) and secured at the end.

- Mesh panels were anchored along the entire top of the slope; they were connected to the top anchors with a 5/8 inch (16 mm)cable. The wire mesh panels were lapped over/around the top anchor cable a minimum of 2 feet (0.6 m), and the connection was done by connecting two transversal cables together with the lacing cable.

- Installed wire mesh panels were secured to the rock slope using a combination of top and vertical anchorage cables, and rock bolt anchors. (See Figure 7).

- All rock bolts and anchors were drilled into competent bedrock to a minimum depth of 6 feet. The minimum pull out resistance required was 48 kips (214 kN), every 5 anchors were checked on the field for pullout resistance.

- All rock bolt anchors were 1 1/4 inch (32 mm) diameter, Grade 75, galvanized All-thread bars, as manufactured by Williams Form Engineering Corp. The rock bolts were anchored using resin grout cartridges.

- The anchors were installed every 6 feet (1.83 m); they were placed alternatively 2 feet (0,6 m) from the crest of the slope and at a minimum of 15 feet (4.6 m) back of the crest in an angle of 35 to 45 degrees from the vertical.

- Three rock bolt anchors were placed at an equal distance on both ends of each section/area to contain the rock within the protected area.

Figure 5 - Lacing cable at every mesh opening.

- Each panel was connected using 5/16 in (8 mm) steel cable in every mesh opening and secured at the end.

- No cables were installed at the toe to capture the rock; the debris is caught by the catchment area.

- The cable panels were designed to be 20 to 30 feet (6.1 to 9.15 m) shorter than the catchment area.

Figure 6 - Installed Rock Mesh B900 on the southbound area.

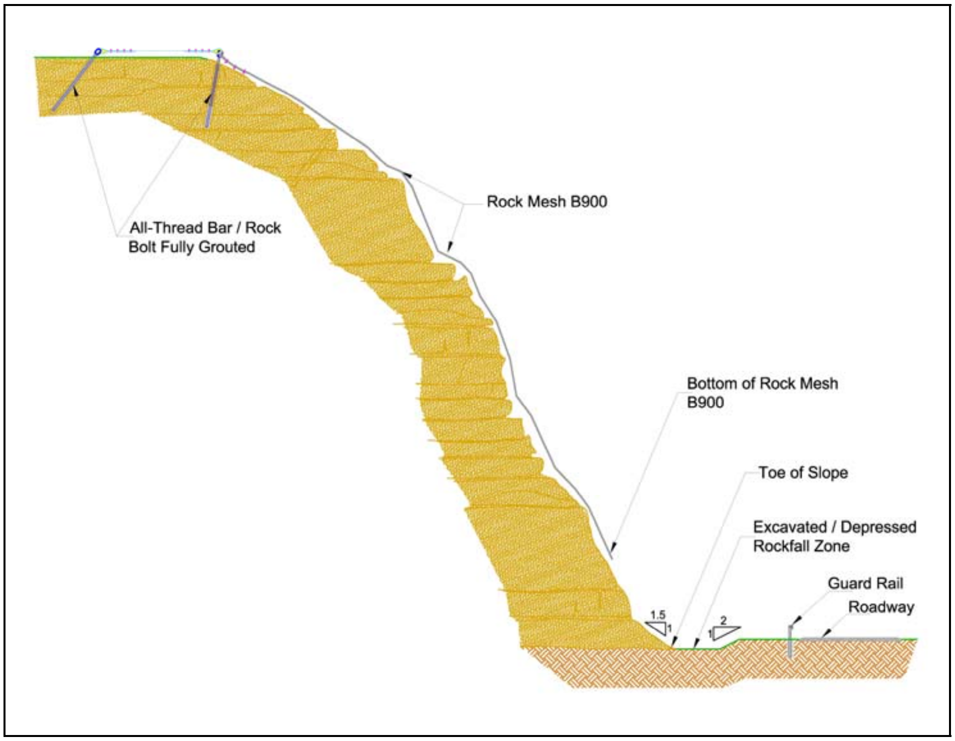

Figure 7 – Typical cross section of the drape netting.

Conclusion and Recommendation

The project went generally well considering that it was a relatively new product on the market and the contractor (Trumbull) had limited experience in installing a rockfall netting system.

- PennDOT had observed some problems related to the ice accumulation of the drapery on a similar site where the anchors pullout resistance was diminished after a few years. In order to prevent this from occurring on this project, the pullout resistance was set at 48 kips (214 kN) almost twice what is normally required. The number of anchors were also increased in comparison to other projects. Inspection of the site may be required periodically to monitor the behavior of the anchors.

- The length of the drapery system is 20 to 30 feet (6.1 to 9.15 m) shorter than the slope, creating some problems from rocks bouncing in the catchment areas. The shorter lengths allow for cleanout of the drop zone without worrying about catching the drape with equipment and damaging it

- The reduced experience of the contractor did require more onsite assistance.

- Although the rock on site was gray in color, the black PVC blended well with the existing rock.

- Manufacturing each panel at the required length did not really save time during the installation.

- This system required no overlap for the panel to panel connection, compared to the specified system, which reduced the quantity of material by 10%.

- The panel to panel connection did not require using clips and cabling as was required for the specified material.

- In certain areas, mostly under the power line, the drape did not go high enough on the slope. This resulted in several rock falls last spring and will likely create some problems in the future unless we can get this area covered. For recommendation, the designers should double check the heights of the slope to ensure they are covering the entire slope of concern.

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Slope Stabilization Techniques Applied in Mountainous Terrain

U.S. Route 2 Realignment Project, Gilead, Maine

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ABSTRACT

In the late summer of 2005 the Maine Department of Transportation (MaineDOT) encountered unanticipated unstable slope conditions during the final stages of a roadway widening and realignment construction project in steep mountainous terrain. While excavating into the base of a 200 meter (m) high steep slope for the realignment construction, unexpected undulating high bedrock conditions were encountered that would not allow placement of an intended 0.6 m thick riprap section on the 1.5H:1V design slope angle. Unfortunately, the upper half of the slope cut had already been excavated and surfaced with riprap using a top-down construction method when the high bedrock condition was encountered at the lower portion of the slope cut. The slope design was modified for the high bedrock condition and included a presplit rock cut for the lower slope and a rockery wall to support the in-place riprap layer. During blasting the general contractor encountered unstable rock and unexpected steep soil slope conditions that could not be stabilized with the planned rockery wall. Due to the onset of winter conditions, only a two month window was available to develop a new design and complete the slope stabilization construction.

Subsequent to the discovery of unstable conditions, MaineDOT retained Golder Associates, Inc. (Golder) to provide an assessment of slope conditions and develop stabilization and roadway redesign alternatives. Golder worked with MaineDOT, the prime general contractor (H.E. Sargent, Inc.), and Janod Contractors, Inc. (Janod), a specialty slope stabilization contractor, to develop, design and construct the following combination of slope treatment and stabilization systems:

- Partial removal of soil and riprap
- Removal of displaced rock blocks
- Stabilization of talus blocks in-place with rock dowels.
- Removal of highly fractured rock.
- Stabilization of remaining steep slopes with a wire mesh system tensioned to rock dowels.
- Stabilization of base riprap layers with anchored steel ring nets.

Stabilization construction proceeded in a five-phased sequence to accommodate design development of stabilization components and the time needed for delivery of materials to the site. The first phase addressed partial removal of soil and riprap on some of the oversteepened soil slopes using mechanical slusher scaling equipment. The second phase included scaling dilated rock blocks and removing a very large displaced rock block (estimated to weigh 280 metric tones) using air bags and a hoe ram. The third stabilization phase required drilling rock dowels to stabilize large talus blocks and soils in-place on 45 degree slopes using a variety of specialty drilling equipment suspended by steel cables attached to trees above the slope work area. Rock dowel bars and wire rope anchors needed for the ring nets and wire mesh system were also drilled and installed during this phase. Phase four included placement of ring nets over the lower segments of remaining riprap. For the fifth phase of work the wire mesh system was installed and tensioned on steep soil slopes varying from 45 to 65 degrees.

Golder provided full-time geologic, geotechnical and engineering oversight during the construction to assess the conditions encountered and aid Janod with the adaptation of stabilization methods. All work was completed in November 2005.

INTRODUCTION

The segment of U.S. Route 2 located in Gilead, Maine is a scenic highway passing through the White Mountain National Forest in western Maine near the New Hampshire border. In 2004 the MaineDOT began a multi-phase reconstruction project for a 12 kilometer (km) section of Route 2 winding through steep terrain from about the Maine/New Hampshire State line to the Pleasant River. The overall project will be constructed in three separate contracts over several years and includes full depth roadway reconstruction, realignment and widening improvements and a bridge replacement. The first construction contract included the western 2.3 km highway section of the project and was completed between January 2004 and November 2005.

In August 2005, during the latter stages of construction for the first contract, unexpected and unstable mixed-face slope conditions were encountered during excavation along roughly a 200 m segment where the road alignment bends around the north face of a steep slope. In early September 2005 MaineDOT engaged Golder and Janod to assess, design and construct slope stabilization measures. The stabilization systems were constructed between September 28 and November 30, 2005. This paper describes the nature of the slope instability, the basis for the stabilization design, and the stabilization construction.

Project Description

Figure 1 shows the location of the subject 2.3 km section of the U.S. Route 2 reconstruction project.

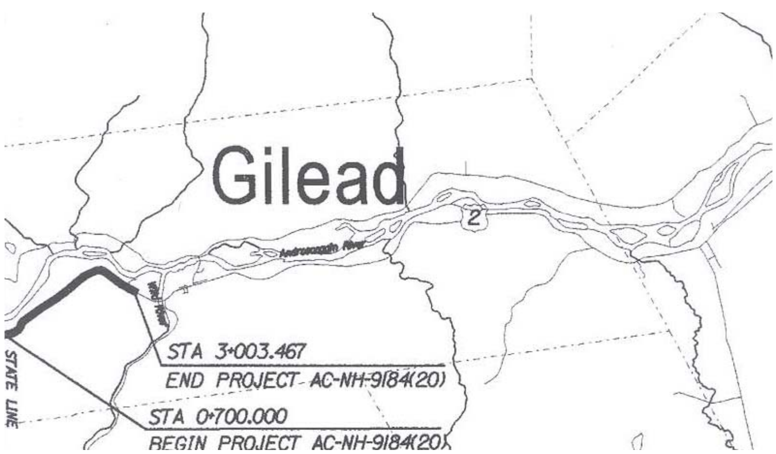


Figure 1 – Project Location

At roughly the midpoint of the project section the road alignment bends around the north-facing slope of Twin Mountain, located within White Mountain National Forest property. Realignment grades in this section required excavations extending up to about 19 m horizontally and up to about 7 m vertically into the pre-existing steep slope face. Wooded steep slope conditions extend approximately 150 m in elevation above the required limit of excavation. The final cut slope design for this section was based on the assumed presence of soil materials along the full slope length and required a 1.5H:1V (horizontal to vertical) slope surfaced with a 0.6 m thick layer of riprap.

Figure 2 – Project View from East

Figure 3 – Project View from West

The construction sequence for this segment of the project included a top-down process whereby the slope was excavated to final grade and covered with riprap in segments starting from the top of the slope and working to the bottom. During the summer of 2005 when the slope excavation and grading work had progressed about half way down the slope length, the bedrock surface was encountered at a higher elevation than anticipated. To account for the rock structure, blasting operations, and the desire to maintain previously place riprap, the slope design was modified by MaineDOT to include blasting the bedrock at an approximately 4V:1H rock slope using presplit smooth wall blasting techniques. The modified slope design included a rock cut with a maximum height of about 8 m. The modified design also included the placement of a rockery wall along the face of the rock cut intended to support the riprap layer present at the top of the rock cut.

During blasting operations in August 2005 for the 4V:1H rock cut, unstable rock and soil slope conditions were encountered along the cut slope section from Sta 2+060 to 2+120 as shown on Figures 2 through 5. Adversely oriented joints and fractures in the bedrock mass were dilated from explosive gas travel, and allowed some large bedrock blocks to be displaced. These joints and fractures dip towards the roadway, i.e., to the north. Near vertical soil slopes were

Figure 4 – Vertical Soil Slope Section 8/26/05

Figure 5 – Unstable Rock Mass Section 8/26/05

also exposed on eastern portions of the cut face extending up to about the full height of the proposed rockery wall. MaineDOT concluded that the post-blasting slope conditions could not be properly stabilized and retained with the planned rockery wall construction. Concurrent with this decision Maine DOT requested Golder and Janod to make an initial site visit to discuss possible slope stabilization alternatives to minimize rockfall and soil slope failure for short and long term conditions.

GEOLOGIC SETTING

The site is located within the White Mountain Range within west central Maine. The bedrock consists of the coarse-grained schist and gneisses of the Devonian-aged Littleton Formation[1]. The bedrock contains several sets of orthogonal joints, including a stress-relief joint set roughly parallel to the ground surface, formed during glacial unloading. This joint system, prevalent throughout crystalline bedrock in New England, plagues rock excavation projects involving blasting and slope stabilization. Overburden soils at the site consist of a mixture of thin, discontinuous saprolite, glacial till, colluvium and landslide deposits[2]. Large talus blocks, up to 5 m in length, exist as embedded and nested boulders within the colluvium at the site. Nearby talus blocks from remnant periglacial landslide events have been measured up to 30 m in length. The presence of these boulders within overburden materials in cut slopes of western Maine are problematic as stability treatment needs to address the finer grained materials as well as the large blocks.

STABILIZATION EVALUATION

Conditions Requiring Stabilization

Pursuant to Golder's initial site visit in late August, the project contractor (H.E. Sargent, Inc.) scaled loose surficial rock from the slope area and cleaned up the soil and rock materials at the base of the slope to allow two lanes of travel on Route 2. Conditions present in early September 2005 are shown on Figures 6 and 7 and included the following conditions requiring stabilization:

- Nearly vertical glacial till soil and completely weathered bedrock slope, with overlying riprap armor

- Dilated bedrock blocks on adverse joints and fractures dipping towards the roadway

- Soil and riprap overlying bedrock surfaces dipping toward the roadway

- The presence of large rock blocks on the slope, generally above the blasted rock zone, that were suspected to be talus blocks and not bedrock.

- Persistent groundwater seepage at the base of the overburden soils exposed above the rock cut.

Figure 6 – Eastern Slope Section 9/2/05

Figure 7 – Dilated Rock Block 9/2/05

Based upon these initial observations a listing of slope areas and attendant stability concerns were identified as summarized in Table 1.

Table 1 – Slope Conditions Requiring Stabilization

Approximate Station Limits	Conditions of Concern
2+120 to 2+100 (eastern end)	Soil slope steeper than 1.5H:1V, with riprap at top of cut
2+087 to 2+100	Near vertical soil slope, riprap at top of exposed soil, suspected talus blocks at top of slope
2+076 to 2+087	High rock face with thin layer of soil and riprap at top of rock cut, suspected talus blocks above rock cut
2+066 to 2+076	Dilated rock blocks, adverse fracture planes, soil/riprap at top of rock cut
2+059 to 2+066 (western end)	Highly fractured rock exposed on slope face, soil and riprap at top of rock cut

After the initial site reconnaissance, updated slope survey data was obtained that indicated the slope section from 2+120 to 2+100 could be regraded to a 1.5H:1V slope with an overlying layer of riprap in accordance with the original design. This section was removed from further consideration leaving four slope areas of concern requiring stabilization.

Consideration of Alternatives

During the initial assessment of stabilization alternatives MaineDOT considered redesigning the road alignment so the final cut slope angle could be flattened along the sections with stability concerns. However, this alternative was rejected due to schedule, cost, and anticipated permitting difficulties.

Several possible remedial alternatives were reviewed to stabilize different portions of the slope including measures to reduce the driving forces causing instability and measures to increase resisting forces in the slope. All alternatives were hampered by the presence of the riprap layer and the unknown thickness and characteristics of soil materials overlying the bedrock in the area immediately upslope of the rock cut. Moreover, schedule constraints required the development of a construction approach that could be adapted to conditions encountered during stabilization construction. Stabilization alternatives initially considered included the following:

- Remove soil and riprap, where possible, to expose a stable bedrock surface.
- Retain soil, riprap and possibly rock blocks in-place with retaining structures including soil nail walls, gabion walls, cast-in-place concrete retaining walls, or gravity retaining walls.
- Stabilize steep soil slopes with anchored wire mesh and geotextile (Tecco system).
- Remove displaced rock blocks or anchor displaced blocks in-place with rock dowels.
- Remove talus blocks or anchor talus blocks in-place with rock dowels.
- Remove highly fractured rock or protect in-place with shotcrete and dowels.
- Anchor the base portion of the riprap layer on the slope face with steel wire ring nets.

Selected Stabilization Treatments

Based on schedule constraints with the pending onset of the winter season, cost considerations, long-term stability, reliability, and practicality, a phased combination of slope treatment and stabilization systems were selected including: partial soil and riprap removal; removal of displaced rock blocks and highly fractured rock; anchoring talus blocks with rock dowels; stabilization of steep soil slopes with Tecco wire mesh; and, securing the toe of the riprap layer with anchored steel ring nets.

Forces causing instability were planned to be reduced where possible by removing soil, riprap, rock blocks and highly fractured rock. Soil removal was planned in the oversteepened glacial till excavation slopes in eastern areas in hopes that a stable 1.5H:1V slope could be formed or the bedrock surface could be exposed. The prospect of soil removal in areas extending from the base of the slope to roughly 60 m up the slope was difficult and made more challenging by the limited room for equipment to operate in the narrow shoulder area at the edge of the roadway. The use of Janod's "slusher", a mini-dragline for mechanically scaling slopes, made this work possible. Riprap that posed a safety threat during stabilization work were also planned to be removed with the slusher. Rock blocks that could be removed without compromising the stability of upslope materials were identified and planned to be removed via scaling with air bags and bars.

For soils, riprap and rock blocks that needed to remain on the slope, three general methods were selected to improve resistance and stability: Tecco wire mesh for steep soil slopes; steel ring nets to support riprap; and steel dowels to anchor large talus blocks. Figure 8 shows a typical cross-section of the application of these stabilization methods.

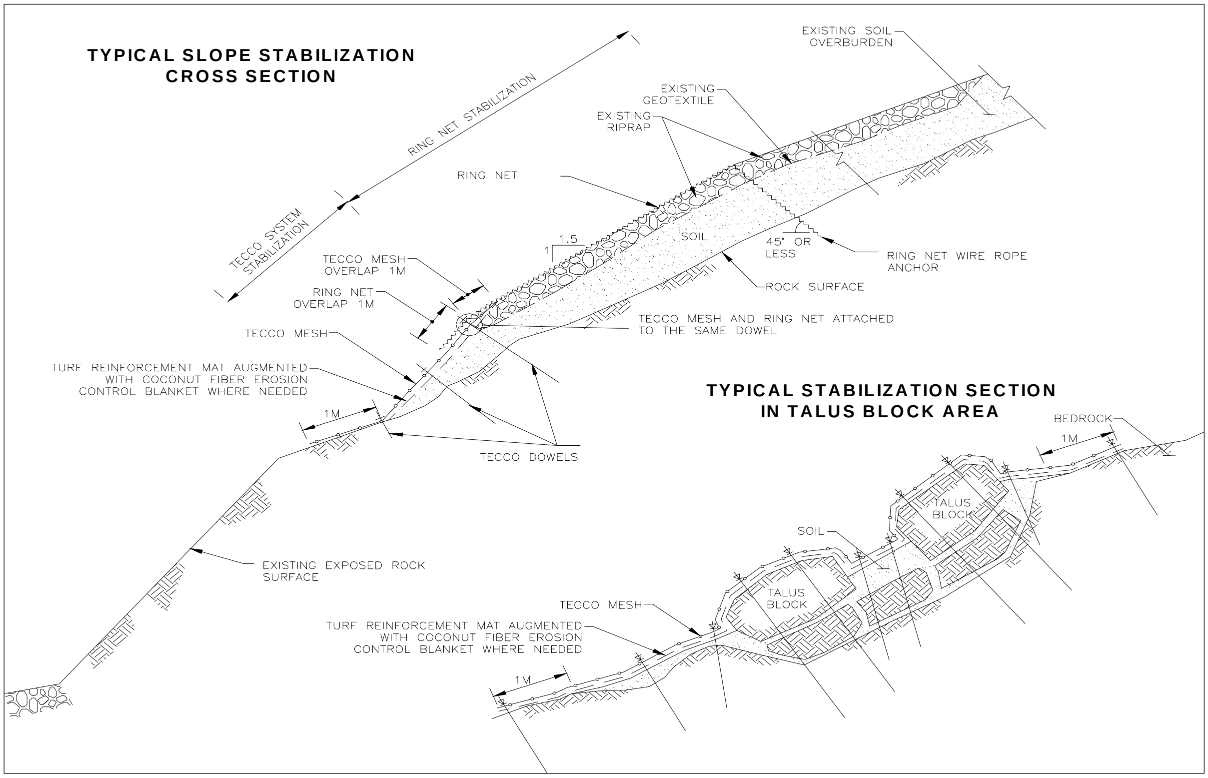


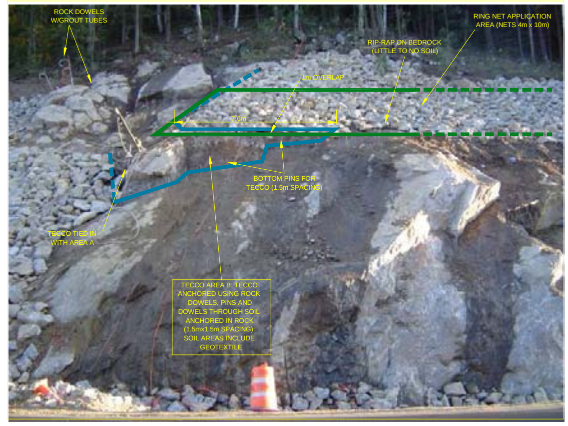
Figure 8 – Typical Stabilization Cross-Section

Combining the planned material removal and in-place stabilization methods, four general slope areas were identified with similar treatment approaches as summarized in Table 2 and shown on Figure 9.

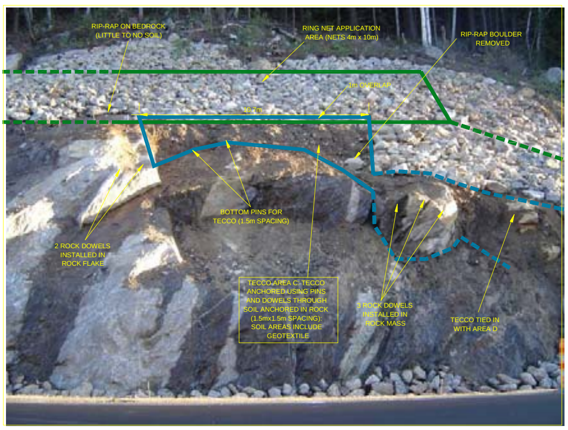
Table 2 – Slope Treatments Selected

Area	Approximate Station Limits	Slope Treatment and Stabilization Remedy
A	2+087 to 2+100	Remove soil and riprap; stabilize talus blocks in-place with rock dowels; place Tecco wire mesh on soil slope areas between talus blocks
B	2+076 to 2+087	Stabilize talus blocks with rock dowels, stabilize soil with Tecco mesh, stabilize riprap with ring nets
C	2+066 to 2+076	Remove rock blocks, stabilize soil with Tecco mesh, stabilize riprap with ring nets
D	2+059 to 2+066	Remove highly fractured rock, stabilize soil with Tecco mesh, stabilize riprap with ring nets

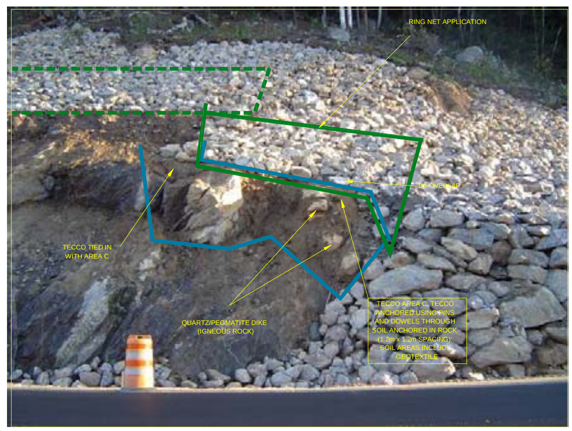
Area A



Area B



Area C



Area D

Figure 9 – Stabilization Areas A, B, C and D

The components and design basis for the selected stabilization methods are summarized as follows:

Tecco Mesh System for Steep Soil Slopes

A tensioned wire mesh system was selected to stabilize steep soil slopes in two types of areas on the slope face: zones between bedrock and previously placed riprap/rockery wall materials; and, soil located between and around talus blocks. The soil slope angles where wire mesh was planned ranged from about 45 degrees (1H:1V) to 65 degrees (0.5H:1V), slope lengths were typically less than 4 m, and the soil thickness was typically less than 2 m. Soil materials were a very dense, partially cemented, glacial till.

The wire mesh system selected was the Tecco system manufactured by Geobrugg Protection Systems. Tecco consists of a diamond-shaped, high-tensile steel wire mesh with an aluminum-zinc anticorrosion coating, anchored to soil and rock slopes using spike plates tensioned onto high strength steel dowels anchored into the slope. For this project all dowels were grouted into bedrock. The long term surficial stability of soils retained by the Tecco system relies on the establishment of a vegetative cover to prevent erosion of fine soil particles between the openings in the wire mesh. Accordingly, the system also includes a geosynthetic filter material beneath the wire mesh to prevent soil erosion while the slope is seeded and mulched and a vegetative mat develops as a natural filter.

The Tecco system design consisted of an evaluation of the surficial stability of soils overlying bedrock that are reinforced with a protective wire mesh layer anchored with dowels. The analysis accounts for slope angles, soil and rock conditions, soil and rock strength properties, pretensioned mesh force, variable anchor forces, and an evaluation of localized instability between individual anchors. The analysis was conducted using the Ruvolum software[3] as described in Geobrugg product documentation[4], and yields a design dowel spacing and tension force for the slope conditions considered. The results of the analysis indicated dowels securing a mesh tension force of 50 kiloNewtons (kN) and spaced on a 1.5 m grid would provide satisfactory resistance.

Talus Block Anchoring with Rock Dowels

For talus blocks considered too large to remove without compromising the stability of upslope conditions (i.e., soils, riprap, other talus blocks, and/or slope conditions beyond the limits of construction), the blocks were anchored in place with rock dowels. Passive rock dowels were chosen for the talus blocks as eventual freeze-thaw conditions will mobilize the blocks as the till shifts. Only small displacements on the shear plane are needed to mobilize shear resistance from the dowel, and as the dowel bends it quickly mobilizes axial tensile strength.

The rock dowels were designed using a limit equilibrium analysis using free body diagrams of a worst-case scenario of a talus block on a sloping bedrock surface.[5,6] The analysis considered forces acting along the talus block/bedrock interface, the weight of the talus block, and the rock strength properties. Seepage and seismic loads were also considered. The analyses were used to determine whether additional resisting forces from dowels were required to provide adequate stability for each talus block considered. Where dowels were required, the analyses indicated the minimum tensile forces required, and a dowel pattern was developed for a uniform dowel capacity.

The results of the analysis indicated an additional dowel tensile force of about 133 kN was required for the largest talus block identified at the site to provide a minimum factor of safety of 1.5 against sliding. The design included use of 25 mm diameter dowel bars with a minimum tensile capacity of 75 kN. a minimum embedment in sound rock of 1.2 m. Dowel spacing was varied depending on the size of the talus block. A minimum dowel embedment in bedrock of 1.2 m was required for tensile strength, but all talus dowels were drilled at least 1.8 m into sound rock to account for the possibility of nested blocks.

Ring Nets for Riprap Toe Support

Steel ring nets were selected to retain the lower portions of the pre-existing riprap layer left in-place on the upper half of the slope. The ring nets consisted of 300 mm diameter interwoven rings of 7-wire galvanized steel draped over the lower 5 m section of the riprap layer, and secured to the slope with rock dowels and wire rope anchors embedded in bedrock. The ring net design used Federal Highway Administration guidelines for the analysis and design of cable net slope protection systems[7], and included an evaluation of the potential rockfall event size, riprap dimensions, slope conditions, interface friction angles, debris load factors, anchor loads, impact loads and snow loads. The results of the analysis were checked against requirements for a short retaining wall to resist lateral earth pressures imposed by a 0.6 m thick riprap layer placed on a slope at the slope angles existing at the site.

The results of the analysis indicated the upper boundary of the ring nets could be supported by 170 kN capacity wire rope anchors spaced about 8 m apart, and the lower boundary could be secured to the rock dowels installed for the adjacent section of Tecco wire mesh at a 1.2 m spacing. The wire rope anchors consisted of galvanized 16 mm diameter, grade 36 steel with a minimum grouted embedment length of 2.0 m in sound bedrock. The rock dowels were 25 mm diameter, grade 150 steel bars, with a minimum grouted embedment length of 1.8 m in sound bedrock.

CONSTRUCTION

Sequence of Work

The stabilization construction procedure involved the following sequence of work (refer to the treatment Area designations described in Table 2 and shown on Figure 9):

1. Regrade oversteepened soil slopes in Area A to 1.5H:1V.
2. Scale loose bedrock blocks and fractured rock in Areas B, C and D.
3. Test probe drilling to determine depths to bedrock in Areas B and C.
4. Drill and install rock dowels in all areas.
5. Drill and install ring net anchors in Areas B, C and D.
6. Install ring nets in Areas B, C and D.
7. Install Tecco mesh system elements in all areas.

Regrade Oversteepened Soil Slopes

Figure 10 – Slusher in Operation 9/29/05

Janod completed soil excavation to layback oversteepened soil slopes covered with riprap to 1.5H:1V in Area A using a slope scaling "slusher" machine. The slusher consists of a compressed air powered, three-drum winch motor that uses a specialized excavation bucket to scrape loose material from soil and rock slopes. The drums contain steel wire rope, and are operated using separate clutches to engage the winches. The two outer cables are thread through pulleys mounted on trees or other anchors at the upper right and left sides of the slope to be excavated, and are attached to the upper side of the bucket. The center cable is attached directly to the lower side of the bucket. The slusher operator can move the bucket up and down the slope, as well as left and right, to access the area requiring excavation. Excavated and scraped materials were piled at the base of the slope and removed with an excavator. Figure 10 shows the slusher in operation. Slushing operations were halted when the excavation encountered talus rock blocks too large to be removed (on the order of 3+ m diameter).

Scale Loose Rock Blocks and Fractured Bedrock

Janod and Sargent removed several rock blocks containing dilated joints and fractures exposed on the bedrock face in Areas B, C and D. Janod removed the smaller rock blocks by scaling with hand scaling bars and compressed air bags used as jacks to open and lift rock blocks, and Sargent used a hoe-ram hammer attached to an excavator to remove and break-up larger blocks. The largest rock block was estimated to weigh 280 metric tons (about 6 m long, 8.5 m wide and 2 m thick) and was removed using both air bags and the hoe-ram.

Test Probe Drilling

Janod drilled four test probes across the riprap layer covering upper slopes in Areas B and C to assess soil thickness, depth to sound rock and the feasibility of removing riprap and overburden soils in these areas. The probes were drilled using a hand-operated "plugger" percussion drill. Below the riprap layer the plugger drill typically encountered about 1.2 m of glacial till and 1.0 m to 1.7 m of weathered rock overlying sound bedrock. Based on these findings it was decided to leave the riprap in place in these areas and stabilize in-place with ring nets.

Rock Dowel Installations

Golder and Janod personnel laid out rock dowel locations based on the design dowel spacing, and additional rock dowels were placed as required to maintain Tecco system requirements for intimate contact between the wire mesh and the slope surface conditions. A total of 177 rock dowels were installed for the project over a five week period using three types of drilling equipment:

- Hand-controlled “plugger” drills
- Air-percussion “plugger”-type top hammer drill mounted on bencher mounted to the rock surface or on a wagon drill frame
- Down-the-hole (DTH) hammer drill mounted on a wagon drill frame

In general, the hand-operated “plugger” drills were used to install dowels in exposed bedrock and for shallow (3 m or less) rock dowels. The bencher-mounted and wagon frame-mounted drills were used to drill dowels for the talus blocks and the Tecco system dowels extending through soil materials (see Figure 11). The DTH drill (Figure 12) was used to drill holes for the wire rope anchors.

The drills mounted on the wagon frames were suspended by steel cables attached to trees above the slope. The drills were maneuvered on the riprap surfaced slope using the self-propelled air winches at the front of the wagon drills. Drill hole diameters were 51, 64 and 89 mm. Drilling for the Tecco and ring net anchors placed within overburden materials required use of casing to prevent collapsing conditions. As the minimum diameter of the DTH wagon drills is about 89 mm, this allowed for drilling a pilot hole within the overburden, followed by drilling in bedrock.

Figure 11 – Top hammer drills mounted on bencher (left) and wagon frame (right) 10/12/05

Upon completion of drilling the dowel bars were inserted into the drill holes and then grouted with a either neat-cement or sand-cement grout. The rock dowel lengths ranged from 0.6 to 6.7 m. Dowels longer than 2 m were grouted by pumping neat-cement via a grout pump through a 25 mm plastic tremie tube mounted to the bar that was open within the bond zone. For rock dowels 2 m or less in length, grouting was performed by placing sand-cement grout via gravity into the drill hole, then inserting the bar. For bars 2 m or longer, polyvinyl chloride (PVC) centralizers were used.

The two types of grout were used including Sika© 212 dry premix grout (sand-cement) and Sika© 300 dry premix grout (neat-cement). Due to cold weather conditions, the grout water was heated prior to mixing. Grout takes were measured and compared to required grout volumes to assure that satisfactory grout quantities were used in each hole. Water testing could not be performed within most of the drill holes, as saturated conditions already existing in many of the holes. However, grout levels were observed for any dropping levels immediately following grouting, and if drop did occur, the holes were topped off the next day.

Each rock dowel was completed with a galvanized steel nut and face plate. The nuts for rock dowels that were not part of the Tecco mesh system were tensioned using an air-powered impact wrench to a maximum tension of about 27 kilonewtons (kN). Dowels in areas requiring Tecco mesh were later completed with Tecco spike plates and tensioned after wire mesh placement to 50 kN with a hand-operated torque wrench.

Ring Net Wire Rope Anchor Installation

Figure 12 – DTH hammer drill on wagon frame used to drill holes for wire rope anchors

The ring net anchor holes were drilled using the DTH wagon drill (Figure 12) with a 89 mm diameter bit, and were inclined at about 45 degrees from horizontal. Anchor drilling depths were terminated after penetrating at least 1.8 m of bedrock. Drilled anchor lengths ranged from 1.8 to 4.3 m.

Upon completion of drilling, the wire rope anchors were inserted into the drill holes. The anchors consisted of 16 mm diameter galvanized wire rope (6x25), frayed at the bottom end. Centralizers were used for anchors longer than 2 m. After anchors were inserted in the hole they were grouted with Sika 212 sand-cement grout.

Dowel Testing

Two test anchors were installed at the base of the bedrock slope below to verify the suitability of the design bond strength values. One test anchor was a 25 mm diameter steel bar used for rock dowels and one test anchor was a 16 mm wire rope used for ring nets. Both anchors were installed in 2 m deep holes with neat cement grouted bond zones limited to the lower 0.6 m (for the dowel bar) and 1.0 m (for the wire rope). Janod then created a jackstand pad at the anchor hole collar and setup a calibrated jack, jackstand and pressure gauge for testing. Four days after grout placement each anchor was loaded in increments to about 80 percent of the guaranteed ultimate strength of the steel in accordance with Post Tensioning Institute guidelines[8]. The rock dowel bar was loaded to a maximum load of 423 kN and the wire rope anchor was loaded to a maximum of 167 kN. Both tests were held at maximum load for 2 minutes with no indication of yielding. The testing confirmed actual rock/grout bond strengths exceeded the design value of 1,033 kPa.

Ring Net Installation

Sargent used a 45-ton crane to move four ring nets (10 m x 4 m size each) onto the slope. Janod then unrolled the ring nets and placed them on the slope riprap for later tie-in. Janod tied in the ring nets using a 16 mm steel wire rope as a seaming rope placed around the perimeter of the ring nets. The upper portions of the ring net perimeter cable were connected to the wire rope anchors. The side and lower portions of the ring nets were tied into rock dowels using galvanized shackles placed beneath the face plates and nuts. The four ring net panels were seamed together using galvanized shackles. Once the nets were connected, the seaming rope was tensioned using a come-along and the rock dowel nuts were tightened.

Figure 14 – Placement of turf reinforcement mat on soil prior to wire mesh

Figure 15 – Tecco mesh / ring net connection prior to tightening seaming cable

Tecco Mesh Installation

The Tecco mesh system materials were constructed according to the following sequence:

1. Excavate soils around rock anchor collars to about 0.3 m depth to accommodate tensioning of the spike plates.

2. Spread seed mix over soil areas and cover with hay.

3. Place a coconut fiber erosion control blanket (ECC-2) over two soil "gully" areas where groundwater seepage was observed during the stabilization construction.

4. Place Landlock TRM 1060 turf reinforcement mat over all soil areas (Figure 14).

5. Place Tecco mesh in vertical strips. The mesh was hung vertically by unfurling the rolls (3.4 m wide), and placing them over the rock anchors/dowels. The mesh was overlapped by at least 0.4 m, and adjacent panels were tied together using Tiger-Tight clips.

6. Place and tension a 16 mm wire rope seaming cable around the perimeter of the Tecco mesh panels. The seaming rope was also integrated with the seaming rope of the ring nets (Figures 15 and 16).

7. Tension the galvanized spike plates over the mesh using an air-powered impact wrench and hand-operated torque wrench to 50 kN (Figure 17).

Figure 16 – Close-up view of Tecco anchor (prior to tightening), with in order from top to bottom: nut, spike plate, shackle, 16 mm wire seaming rope (right), ring net cable (left), Tecco wire mesh

Figure 17 – Using air-operated impact wrench to tighten nuts onto Tecco anchor spike

POST CONSTRUCTION CONDITONS AND CONCLUSIONS

Near completion conditions at the onset of winter precipitation in November 2005 are shown in Figure 18, and recent conditions in the summer of 2007 are shown in Figure 19. While the stabilized slope faces the north and receives little direct sunlight, the vegetative species planted as part of the stabilization, as well as native wild-growing species have already taken root and are supplementing the mechanical stabilization construction. No slippage, sloughing or other evidence of slope movement in the stabilization area were noted in the summer of 2007, about 1½ years after construction was completed.

The stabilization approach used for the Route 2 project was designed to be flexible, as subsurface conditions were not known prior to the design, and would only be known during the anchor/dowel installation phase. This allowed no time for redesign of the stabilization systems. Changes to the design, such as extending drilling depths to obtain minimum anchor bond lengths, were made in the field using full-time geologic and geotechnical observation. This method was required in order to prevent project delays, and reopen the roadway to the full extend prior to the winter season. The project succeeded due to the close teamwork and cooperative effort between Maine DOT, the geotechnical design engineer, the slope stabilization contractor and general contractor.

Figure 18 – Stabilization work nearing completion 11/22/05

Figure 19 – Slope conditions 1½ years post construction – 7/17/07

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A Nightmare On Elm Street:

The December 26, 2005 Rockslide and Subsequent Rock Slope Stabilization and Repairs, Downtown Montpelier, Vermont

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ABSTRACT

On December 26, 2005, between 4:15 and 6 p.m., a rock slide occurred in downtown Montpelier, Vermont, starting near the crest of a 100 feet (ft) high rock slope below Cliff Street during a heavy warm rain event following a period of sub-freezing temperatures. About 2,700 cubic yards of rocks, soil and debris slid downslope and onto Elm Street, with several large rocks striking an apartment building. Above ground utility lines were cut by falling rock, and the area lost power and communication services. Damage to Cliff Street included loss of about 5 ft of pavement and 70 ft of guardrail. Both Elm and Cliff Streets were immediately closed to traffic.

Mapping, drilling and inspection via rope rappel indicated the failure occurred due to flexural toppling of strata near the crest dipping steeply into the slope, and chevron toppling mid-slope in a weathered bedrock zone about 10 ft deep. The strata consist of interbedded phyllites, calcareous metasiltstones and graphitic slates of the Devonian/Silurian Waits River Formation, complexly folded and foliated, and subjected to differential weathering. The rock mass strength near the crest was decreased by weathering, soil infilling and root jacking, which enhanced dilation of fractures and bedding planes.

Slope repairs were completed between mid-January and early May, 2006. The repairs consisted of several slope stabilization construction methods, including mechanical and hand scaling, construction of a shotcrete/rock nail retaining wall, pattern and spot rock bolts, Tecco stabilization systems on steep soil slopes, dental shotcrete, a rockfall drape, and installation of drain holes.

INTRODUCTION

On December 26, 2005 a rock slide occurred in Montpelier, Vermont (see Figure 1). The rock slide was located northwest of Elm Street, between School Street on the southwest end and Spring Street on the northeast end. Cliff Street is located adjacent to the crest of the slope, approximately 85 to 100 feet (ft) above and roughly parallel to Elm Street in the slide area. The rock slide occurred during a heavy warm rain which followed a period of intense freezing. Rocks, soil and debris toppled and slid to the southeast down the slope, across the pedestrian walkway located northwest of Elm Street. Several large rocks struck planters in front of the apartment building located on the southeast side of Elm Street. Two utility poles on Elm Street were broken by falling debris, and the lines attached to them were cut or damaged. Cliff Street lost about 5 ft of street width (pavement) and approximately 70 ft of guardrail. The City temporarily closed Elm Street and Cliff Street to traffic in the area of the slide[1].

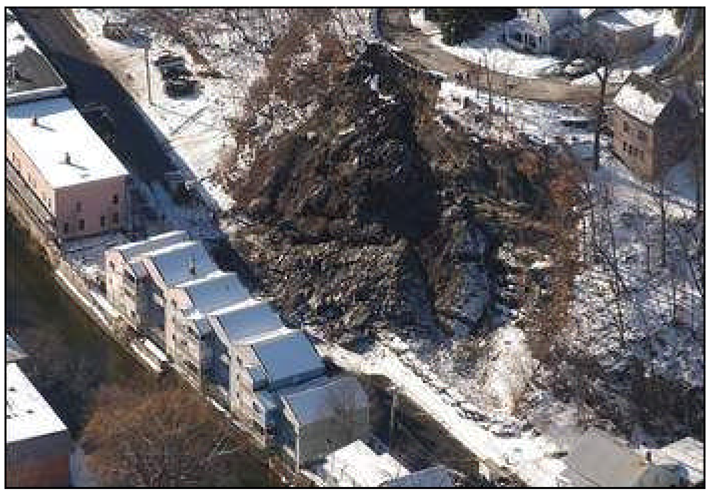

Figure 1 – Aerial view of rock slide on December 26, 2005, one day following the slide event

The rock strata consist of low-grade metamorphosed, gray to dark gray phyllite, calcareous metasandstone and graphitic slate of the Cambrian-aged Waits River Formation. The Waits River Formation consists of undifferentiated brown, gray, punky weathering crystalline limestone (often sandy), and calcareous phyllites interlayered with dark gray, graphitic phyllite[2]. The slide occurred due to a combination of flexural toppling of steeply dipping (56 to 76 degrees) competent rock strata that dipped into the slope in the upper slide area and chevron toppling of less competent rock strata down slope from the competent rock in an approximately 10 ft thick weathered bedrock zone. The upper rock zone at the crest of the hill was covered with 5 to 10 ft of soil and supported a growth of trees. Based on observations of the adjacent slope crest, it is likely the rock strata near the crest were fractured and dilated, had soil infilling and roots growing into the joints, all of which decreased the strength of the rock mass at the slope crest.

ROCK SLIDE EVENTS

The mechanisms that triggered the slide are likely long-term weathering, dilation of rock strata at the slope crest, and water/ice pressure in rock and soil-filled joints that caused the slope to dilate further and topple. Eye witnesses reported the landslide event started at about 4:15 p.m. on Sunday, December 26, 2005, and lasted about two hours. The rock slide occurred following a heavy rain event on December 24-25, 2005, which was preceded by a month of sub-freezing temperatures. No one was hurt or killed during the slide; however power, telephone and cable TV utilities, as well as street lighting, were knocked out during the slide, as telephone poles were snapped by falling debris. The utilities were knocked out on both Elm Street and Cliff Street. The slide debris damaged the sidewalks and roadway of Elm Street, wooden planters of the apartment complex on Elm Street, and caused a 50-foot long and up to 5 ft wide swath of Cliff Street to collapse (including a water service utility; see Figure 2). No buildings suffered structural damage, but the City evacuated residents from the apartment building as a precaution against further slides. The slide occurred about three blocks from the state capitol building, and received extensive media attention.

Figure 2 – Damage at the head of the slide on Cliff Street, December 27, 2005

Access to about 18 residences on Cliff Street was severely limited by the slide, precluding delivery of fuel oil, refuse pick-up and emergency services. A very steep gravel road, treacherous in winter conditions, was the only way to access the residences; therefore reopening of Cliff Street, even to one lane, would allow resumption of vital services.

The rock slope has had a history of rock slide events, the most recent occurring in 1996 when a smaller slide damaged a house below the steepest part of the slope. The damage was so extensive that the City of Montpelier condemned the property, removed the damaged house, and placed a deed restriction for future structures. The City isolated the slide area using a 6-ft high rock wall. Previous slides at the site occurred in the 1920's.

The following day, the City conducted remedial measures to keep the area safe and prevent further slides. These measures included: clearing some of the slide debris on Elm

Street, directing utility crews to shut off and removed live utilities, placing jersey barriers in front of the apartment building, shutting down traffic flow on Cliff and Elm Streets, placing sand bags on Cliff Street to divert meltwater from the slide face, and placing 24-hour guards at the slide site.

GEOLOGY

The slide occurred at the top of a 100-ft high cliff composed of interbedded phyllite, calcareous metasandstone and graphitic slate of the Devonian-Silurian Waits River Formation[2]. The overburden consists of about 5 to 15 ft of dense, dark brown till and fill materials used to construct Cliff Street. The metasandstone beds reach a maximum thickness of about 5 ft. The foliation of the interbedded phyllite, metasandstone and slate dips steeply into the slope (i.e., to the west) at about 70 degrees. Mapping by the Vermont Geological Survey (VGS) indicates the Waits River Formation has experienced up to five different deformational events, consisting of foliations, complex folding, and broad flexural folding. Differential weathering of the very weak slates and less weak phyllites between the more resistant metasandstone causes localized toppling to occur. This type of slope failure is common within the Waits River Formation in other locations of the City and in Vermont.

An eyewitness described the slide failure starting at midslope within one of the thicker metasandstone beds, and not at the apparent slope scarp at the top of the cliff. As the midslope metasandstone beds toppled, one large block about 20 ft long (nicknamed the “bulldozer block”) slid down the south side of the cliff and snapped a utility pole. The path of the block on the slope is clearly evident on the aerial photographs. As the blocks toppled, phyllite and overburden materials above the metasandstone layer progressively failed further until part of Cliff Street was undermined.

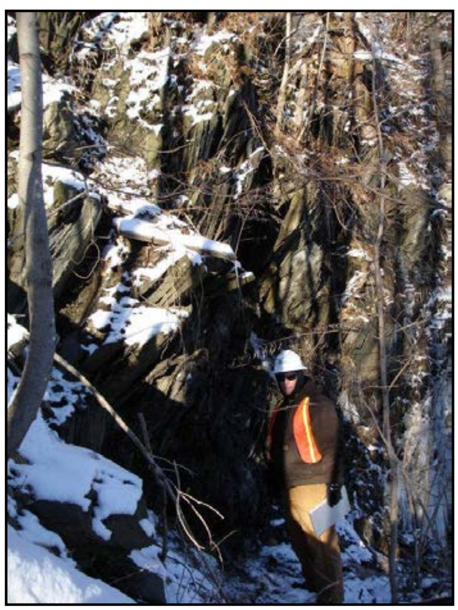

Figure 3 – Chevron toppling within phyllite dipping steeply into the slope

Subsequent field work conducted on rock outcrops located near the slide area, and consultation with the VGS indicate the failure mechanism consisted of chevron toppling[3] within the middle part of the slope, followed by rotational slumping within the overlying overburden materials (see Figure 3). We suspect that the extended sub-freezing weather in December 2005 caused ice jacking to occur behind the metasandstone layer, pushing the blocks out from the slope. When the sudden thaw occurred, accompanied by heavy rain, the ice that had held the blocks to the slope melted, eliminating the remaining support for the blocks, and the blocks toppled.

SLOPE REMEDIATION APPROACH

The City of Montpelier retained Janod to install the stabilization measures and Golder to prepare design drawings and specifications. The Elm Street stabilization project included temporary stabilization of the soil and weathered rock at the top of the slide with a three-inch shotcrete facing, installation of a temporary rock fall containment system, removal of slide debris, and construction of soil and rock retention systems, including a reinforced shotcrete wall, rock dowels, anchored wire mesh, and a rock drape. Janod installed these systems in an area roughly 200 feet long by 90 feet high.

Following the slide, the Federal Highway Administration (FHWA) determined that due to the impacts to traffic flow within the City, which include state and federal highways, the costs the City would incur to remediate the slide area and reopen the roadways would be 100% reimbursable from the FHWA if the work was completed within six months following the event. As the clock was ticking, and returning city life to normal was paramount, the City elected to conduct the remediation using a “design on the fly” approach, where geotechnical engineering and design would be conducted immediately ahead of the slope contractor’s work. This required a high level of collaboration between the City, Golder, Janod and VTrans such that work would not be delayed. As changed conditions are the norm in most slope remediation projects, the design approach was flexible, allowing for a variety of slope remediation techniques used in the design if conditions warranted. Due to the high profile nature of the slide event, the governor of Vermont directed VTrans to assist in anyway possible. VTrans provided geotechnical drilling, laboratory services and assistance and review of the remediation design.

The remediation approach consisted of placing a temporary rockfall barrier on Elm Street to protect the apartment building from future slides, geologic and geotechnical testing, temporary stabilization of the upper slide area to prevent further loss of Cliff Street, scaling of loose rock on the face, and permanent overburden and rock stabilization, including the reconstruction of Cliff Street, to reopen roadways and reconnect utilities.

Testing

VTrans provided a geotechnical drill rig to drill two vertical, 80-ft deep core borings collared at the top of Cliff Street to provide subsurface geologic and geotechnical data on the lithology behind the remaining cliff face. The samples of the rock core, as well as the rock outcrop observations, provided input on rock stability assessment (RQD, UCS, GSI, and Hoek-

Brown criterion). The VGS provided field mapping information of the area, and review of our geologic observations in context with regional metamorphic deformation.

Monitoring

Field observations on December 27, 2005 indicated the potential for tension cracks forming in the remaining pavement of Cliff Street, and the presence of an apparently unstable phyllitic rock mass remaining on the slope. Crack gauges, monitored at hourly intervals, indicated the cracks in the pavement were not growing, and movement at the head of the slide had ceased. Twice daily surveys using prisms placed in the apparently unstable rock mass, however, indicated the mass was moving out and downslope by as much as 0.12 ft per day (see Figure 4). Therefore future sliding events could be possible, and these potential slides had to be addressed immediately.

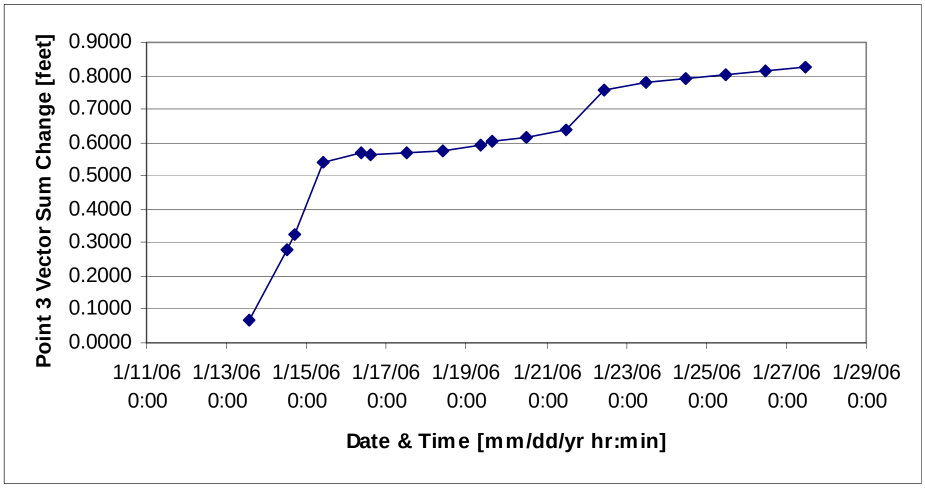

Figure 4 – Movement of rock debris from daily survey measurements

Temporary Rockfall Barrier

In mid-January 2006, Janod installed a 2,000 kilojoule rockfall barrier in the middle of Elm Street. The 260-ft long barrier employed interweaved ring nets, tieback cables with braking elements, and pivoting fence posts (see Figure 5). The temporary rockfall barrier served two purposes: to stop rockfalls and falling debris, and to catch rocks removed during scaling. Scaling of the slope was done using hand scaling bars, air bags, boulder busters and a mechanical slusher.

Figure 5 – Installation of temporary rockfall barrier on Elm Street

Temporary Stabilization

Temporary stabilization construction of the top of the slide area was completed concurrent with installation of the rockfall barrier on Elm Street. This stabilization consisted of installation of geotextile drainage strips and shotcrete facing installation, followed by relocation of utilities and drilling and installation of rock nails anchored within bedrock. As the nightly temperatures in January commonly dipped to below 0 degrees Fahrenheit (°F), hoarding, consisting of tarps and propane heaters, was needed to keep the ground and shotcrete above 40°F so it could properly cure (see Figure 6). The south facing slope, coupled with the heaters, kept the work area as warm as 70°F. One lane of Cliff Street was reopened for service vehicles following the temporary stabilization construction.

Figure 6 – Application of shotcrete under hoarding during winter conditions

Permanent Stabilization

As the slide area consisted of a variety of geologic elements (three lithologies and overburden), the permanent stabilization consisted of several elements (the "toolbox" approach). These stabilization measures consisted of a permanent rock nail/shotcrete wall, pattern and spot rock dowels, drainage elements, dental shotcrete, installation of Tecco mesh reinforcement, installation of a wire mesh rockfall drape, and construction of a moment slab and bridge rail to reopen Cliff Street to two lanes.

The thickened permanent rock nail/shotcrete wall installed below Cliff Street was designed with three layers of steel reinforcement (rebar and welded wire fabric), installed between layers of fill shotcrete (see Figure 7). The shotcrete included steel fibers for further reinforcement. Due to the cold weather installation, hoarding was used to keep the curing temperatures above 40°F. The rock nails were extended into the shotcrete as the wall was built out. The wall was founded on a hand-excavated bedrock bench, reinforced with rock dowels. The permanent shotcrete wall was built out to a maximum thickness of 5 ft.

Figure 7 – Construction of permanent shotcrete wall

Pattern rock dowels were installed in a section of the rock slope that experienced previous toppling, which created an overhanging condition. Spot rock dowels were installed to retain blocks of metasandstone that could be prone to toppling in the future. Areas containing voids behind scaled blocks were cleaned and filled with dental shotcrete for additional support.

While only one drillhole exhibited wet conditions, five slightly inclined drainage holes (about 30 ft long) were drilled at the base of the slope to drain the slope crest. PVC stubout pipes were installed in the collars of the drains. Additional drains were installed at the base of the permanent shotcrete wall.

Oversteepened soils existed on the sides of the head of the slide just below Cliff Street. To stabilize these soils, Tecco mesh attached to pattern rock nails anchored in bedrock was installed. The Tecco mesh included coconut netting and geotextile placed on top of a seed mix, followed by the heavy wire mesh.

Traffic on Elm Street is not limited to just vehicles. Pedestrians use both sides of Elm Street, and permanent closing of the sidewalk adjacent to the slope was not possible. To protect pedestrians from future minor rockfalls on the scaled rock slope, a double-twist wire mesh rockfall drape was installed. The design employed an open throat at the top, propped on vertical bars, to retain rocks falling from an upper, less steep portion of the slope.

The final remediation element consisted design and construction of a concrete moment slab on Cliff Street above the shotcrete wall. The moment slab design was adapted from typical VTrans bridge designs, as the moment slab also included anchorages and posts for a bridge rail. To provide drainage of shallow water under the slab, drainage stone and piping were designed to divert water away from the shotcrete wall and cliff face.

CONCLUSIONS

All of the rock slope remediation construction was completed by mid-May 2006, well within the six-month FHWA funding window, and therefore the City was fully reimbursed for these costs. Elm Street was opened to traffic in mid-May as well. Following moment slab construction and final paving, Cliff Street was opened to two lanes of traffic by July (see Figure 8).

Figure 8 – Final repairs of slide area

Success of the project was possible due to teamwork between city, state and federal governmental agencies, the geotechnical engineering consultant, and the rock slope stability contractor. The flexible design approach, using several stabilization methods adapted to changing conditions that all parties were familiar with, also allowed for minimal delays to the construction schedule. Finally, those most affected by the event, the residents of Montpelier, provided much support and encouragement for the designers and construction crews.

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Geotechnical Investigations for Realignment of I-76 Across The New Baltimore Landslide, SW Pennsylvania

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ABSTRACT

The Pennsylvania Turnpike Commission (PTC) plans to widen I-76 due to increased traffic in Somerset County. The New Baltimore landslide, which has caused serious problems for the turnpike since its construction in 1939, is located near mile marker 128 in Somerset County. The landslide extends 2000 feet (610 m) upslope and 1000 feet (305 m) laterally, and moves 5 to10 inches (13 to 25 cm) per year. The design plan for widening the turnpike may require cutting into the slope between mile markers 128 and 129 on the east-bound side of the turnpike, thereby passing through the New Baltimore landslide. A matter of significant concern is reactivation of the landslide upon realignment, adding to the existing instability.

This research project is being conducted to determine the engineering properties of rocks and stability of slopes located between the mile markers mentioned above. Field work is currently being conducted to find the upper limit of the landslide. Preliminary discontinuity data are being analyzed to evaluate various modes of slope failure. A subsurface investigation, consisting of 12 borings and installation of 6 slope inclinometers and 1 piezometer, was conducted by American Geotechnical and Environmental Services, Inc. of Pennsylvania. Laboratory tests were conducted on the core samples to determine slake durability and unconfined compressive strength.

The results of the study completed so far show that the bedrock geology of the New Baltimore landslide site consists of the Upper Devonian Catskill Formation which contains inter-bedded sequences of claystone, siltstone, and sandstone units. The slake durability index of claystone ranges from 72% to 99% whereas the unconfined compressive strength of the sandstone/siltstone units ranges from 8900 psi (61 MPa) to 25600 (177 MPa) psi. The inclinometer data show that there is small movement along the weaker claystone/clayey siltstone units (where gouge was encountered during drilling) and this is being closely monitored. This study is still in progress. The results of the study will be used to develop remedial measures for the New Baltimore landslide so that the interstate highway can be realigned safely.

INTRODUCTION

Background

The New Baltimore Landslide is located between mile markers 128 and 129 along the Pennsylvania Turnpike, I-76 (Figure 1), in Summerset County. The landslide has caused serious problems for the turnpike since its construction in 1939. The landslide has been shown to extend 2,000 feet (610 m) upslope and 1,000 feet (305 m) laterally and it moves 5 to 10 inches (13 to 25 cm) per year (Tinsley, 2004; Tinsley and Shakoor, 2004). The PTC completed the construction of the turnpike through the area of the New Baltimore Landslide in 1939. When the turnpike was built in this area, it cut through an ancient landslide that most likely reactivated the landslide due to removal of lateral support. The PTC plans to widen I-76 due to increased traffic in Somerset County. This may require cutting into the slope between mile markers 128 and 129 on the south side of the turnpike, thereby passing through the New Baltimore landslide. This construction through the New Baltimore Landslide is likely to cause an even greater decrease in lateral support in the toe area of the landslide. This could result in increased movement along the existing failure plane and dangerous conditions to traffic along the turnpike. The stability of the slope comprising the New Baltimore landslide on the turnpike needs to be determined to evaluate the potential for further sliding.

Geologic Setting

The stratigraphy in the New Baltimore Landslide area ranges from the Upper Devonian Jennings Formation through the Upper Devonian Catskill Formation. The total thickness of this stratigraphic sequence is approximately 8000 feet (2,438 meters). The stratigraphy consists of interbedded sequences of harder and softer strata including siltstone, shale, and grayish red sandstone units. The Catskill Formation is a deltaic formation that was deposited during the Acadian Orogeny (380-400 Ma) (Tinsley, 2004).

Structurally, tectonic forces created the Valley and Ridge Province resulting in thrust faults, folding, erosion, and incising of streams (Harper, 1999). The area contains northeast-trending anticlines and synclines which are strongly folded in the eastern part of Somerset County. There are no major faults shown on the surface, but drilling shows faulting at depth within these formations. Many discontinuities occur in the rock units in the study area, which provide potential for a variety of slope movements.

In southwestern Pennsylvania, earthflows represent the primary problem in slope movement. Earthflows occur in areas where shallow soils are developed on steep slopes that have clay-rich bedrock (Turner and Schuster, 1996). Water is another main triggering agent of landslides. Water can reduce friction along potential sliding surfaces. Landslides usually begin to move during or after heaving rainfalls. The presence of a weak rock layer on top of an impermeable layer, such as clay or claystone, can create a situation where the weak layer is easily saturated with water. This is common in Pennsylvania where much of the stratigraphy includes relatively thick, fractured sandstone and shale overlying carbonaceous shale and claystone.

Previous Investigations

The New Baltimore Landslide has been studied since the Pennsylvania turnpike was constructed in 1939. The construction of the Pennsylvania turnpike reactivated an ancient landslide and in the 1940's a new slide occurred which blocked an area of the turnpike. This slide prompted immediate attention and studies have been ongoing since then. The PTC has sketched and mapped the landslide to show the extent of the movement along the slide. A 40-foot (12 m) wide bench with drainage was constructed on the landslide by the PTC in the 1950's to try to stabilize the slide. This attempt at stabilization has not worked very well because the landslide is still creeping a rate of 5 to 10 inches (13 to 25 cm) per year. In 1972, PTC hired Geomechanics, Inc. to perform a subsurface investigation to find the failure plane of the landslide and they drilled three borings. In 1999, the PTC was concerned about the stability of the New Baltimore Landslide and hired American Geotechnical and Environmental Services (AGES) to perform a more detailed investigation. The purpose of their investigation was to find the rate of movement, location of the failure plane, and remedial measure need to stabilize the slope. They drilled 18 borings, installed 15 vibrating wire piezometers, 11 Time Domain Reflectometry cables, and three slope inclinometers (Henderson, 2000). Ryan Tinsley, a previous graduate student at Kent State University, worked along with AGES to research the slide. He mapped the landslide and reported his results in his master thesis which was completed in 2004. In 2006, the PTC hired AGES again to perform geotechnical investigations and to determine the stability of the slopes along the stretch of the Turnpike between mile markers 128 and 129. This company has drilled 10 borings on the south side of the turnpike (Figure 2). The area of these 10 borings is also shown as the "upper section" of the landslide in Figure 3. Additionally, two borings were also drilled on the north side of the turnpike to investigate the toe area of the New Baltimore Landslide (Figure 2). Slope inclinometers have been installed in 5 of the 10 borings on the southern side as well as in the 2 borings on the northern side of the turnpike. A nested vibrating wire piezometer has also been installed to measure hydrostatic pressure. The slope inclinometers have been monitored by AGES and the senior author of this research. Figures 4, 5, and 6 show examples of unstable rock at the toe of the New Baltimore Landslide.

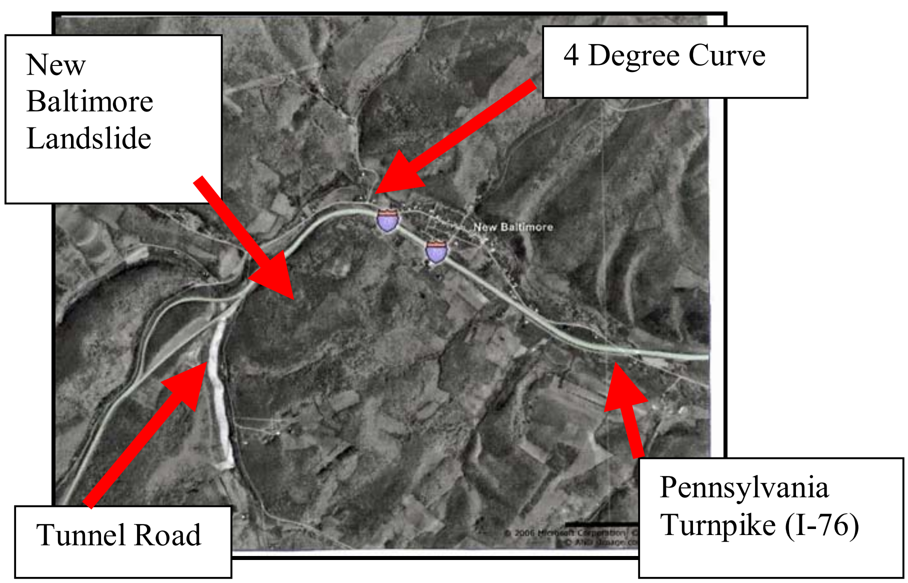

Figure 1 – The study area showing the locations of the New Baltimore landslide.

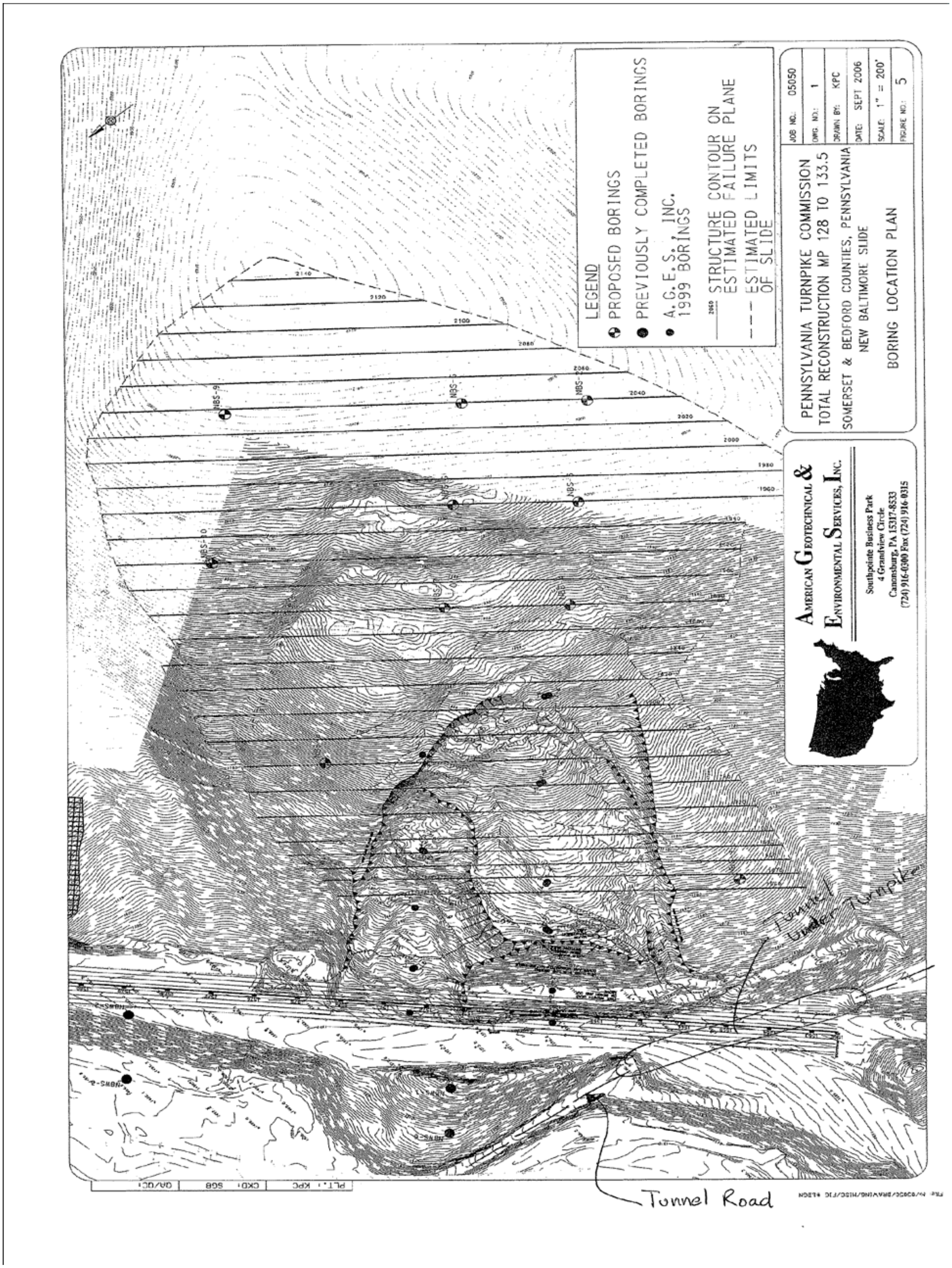

Figure 2 – Borehole location plan.

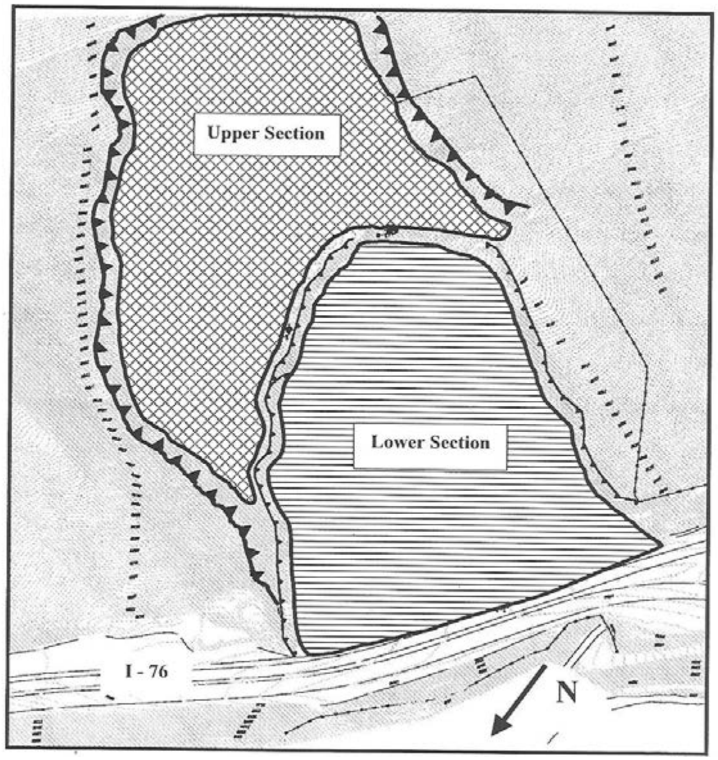

Figure 3 – Landslide divided into upper and lower sections (Tinsley 2004).

Figure 4 – Toe area of the New Baltimore Landslide along I-76.

Figure 5 – Unstable rock mass at the toe of the landslide along I-76.

Figure 6 – Toe of the New Baltimore Landslide.

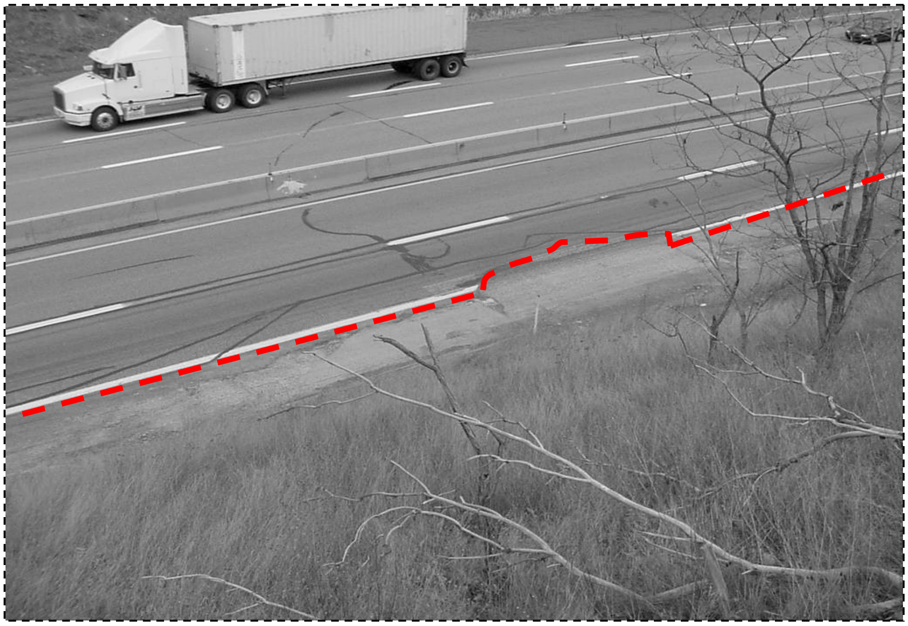

Figure 7 – Pavement heaving of the eastbound lane of I-76.

STUDY OBJECTIVES

The objectives of this research project are as follows:

1. Map rock outcrops within the cut areas along the turnpike and the limited exposures along the tunnel road slope (Figure 1).
2. Log new borings to be drilled in the fall of 2006. These include the borings above the existing slide in order to establish the limits of the prehistoric slide plane and the side of the slide area. Borings need to be logged also in the proposed cut at the 4-degree curve area to the east of the New Baltimore landslide (Figure 1).
3. Investigate the limit of the prehistoric landslide and verify if the stratigraphic interval containing the failure plane is present in the 4-degree curve cut and look at how the underlying geologic structure varies locally. Minor faulting may be present due to variations in the dip angle, strike and offset of the bedding.
4. Test the engineering properties of the material from pre-historic slide plane and compare these with Ryan Tinsley's results (Tinsley, 2004).
5. Test the engineering properties of the rocks collected from the 4-degree curve area.
6. Test the engineering properties of rock samples collected from the existing cut exposures at the New Baltimore landslide.
7. Perform slope stability analyses to investigate the measures required to prevent remobilization of the pre-historic landslide material if excavated.

FIELD AND LABORATORY INVESTIGATIONS

The subsurface investigation on the New Baltimore Landslide was performed by American Geotechnical and Environmental Services (AGES) and the senior author of this paper. The Pennsylvanian Turnpike Commission (PTC) hired AGES to conduct this investigation due potential slope instability. AGES contracted different private drillers to drill the borings in the investigated area. As stated previously, 10 borings were drilled on the slope of the landslide south of the turnpike and 2 borings were drilled on the north side of the turnpike. Borings were also drilled on the 4-degree curve area to compare the stratigraphy (Figure 2). The purpose of the borings was to record the stratigraphy, find the location of the failure plane, and monitor the ground water pressure. The borehole locations were carefully selected in order for cross-sections to be drawn in a line down the slope. The boreholes for this research were placed upslope from the previous boreholes drilled in 2000 (Figure 2). Figure 8 shows a close up view of the boring locations, and their designations, in the upper slope. The boreholes in the landslide area ranged in depth from 56.5 feet (17.2 meters) to 120 feet (36.6 meters). The core was placed in wooden core boxes and logged in detail by geologists from AGES. The logs included information on rock quality designation (RQD), percent recovery, sample description, and water content.

Slope inclinometers and piezometers were installed in the upper area of the slope, above the existing inclinometers from Ryan Tinsley's study (2000), and also on the slope north of the turnpike. Figure 2 shows the boring location plan for monitoring slope movement and Figure 8 shows a close up view of the borings in the upper slope. The borings were placed within the suspected area to define the area of the landslide.

The laboratory tests performed on the core samples included determination of slake durability, unconfined compressive strength, and dry density. The direct shear test will be performed on the weaker clay layers in the fall of 2007. All the laboratory tests were performed according to American Society for Testing and Materials (ASTM) standards (ASTM, 1996).

RESULTS

The results of this study show that the New Baltimore Landslide is a translational slide according to the modes of failure described in Hoek and Bray (1981). The mass is moving along a clayey siltstone layer located at a depth of 43 to 63 feet (13 to 19 m) on the upper slope (upper section in Figure 3). Slope inclinometer data were plotted to show depth and amount of movement. An example of such a plot for boring NBWS-1 is shown in Figure 9. Table 1 summaries the depth to the failure plane as indicated by 7 inclinometers that were placed on the north and south side of the turnpike. The date from each inclinometer was analyzed to determine the depth of the failure plane and also the rate of movement (Figure 9). On the lower slope moves at a rate of 5 to 10 inches (13 to 25 cm) per year and the upper slope, the slide moves at a rate of 1.5 inches per year (4 cm per year) as indicated by the slope inclinometers. Table 2 shows the results from the laboratory testing. The data in Table 2 are provided to characterize the materials involved in landslide mass. The density values in Table 2 along with the shear strength data (to be collected later) will be used in stability analysis.

Table 1 – Depth of failure plane below the ground surface.

Inclinometer	Depth of Failure Plane Below the Ground Surface
NBWS-1	90 ft
NBWS - 6	28 ft
NBS - 4	43 ft
NBS - 5	50 ft
NBS - 6	57 ft
NBS - 7	58 ft
NBS - 8	60 ft

Table 2 – Range and average results from lab testing.

Lithology	Dry Density (pcf) *		Slake Durability Index, Id^2 (%)		Unconfined Compression Strength, q_u (psi) **	
	Range (pcf)	Average (pcf)	Range (%)	Average (%)	Range	Average
Sandstone	157.8 - 168.6	164	NT ***	NT	5229.8 - 30627.6	19470.0
Siltstone	166.7 - 169.6	167.6	83.8-98.7	91.7	11408.4 - 26529.6	17386.1
Clayey Siltstone	NT	NT	72.4-96.8	87.7	NT	NT

* Pounds per cubic foot (1 pcf = 47.88 Pa)
** Pounds per square inch (1 psi = 6, 895 Pa)
*** NT = not taken

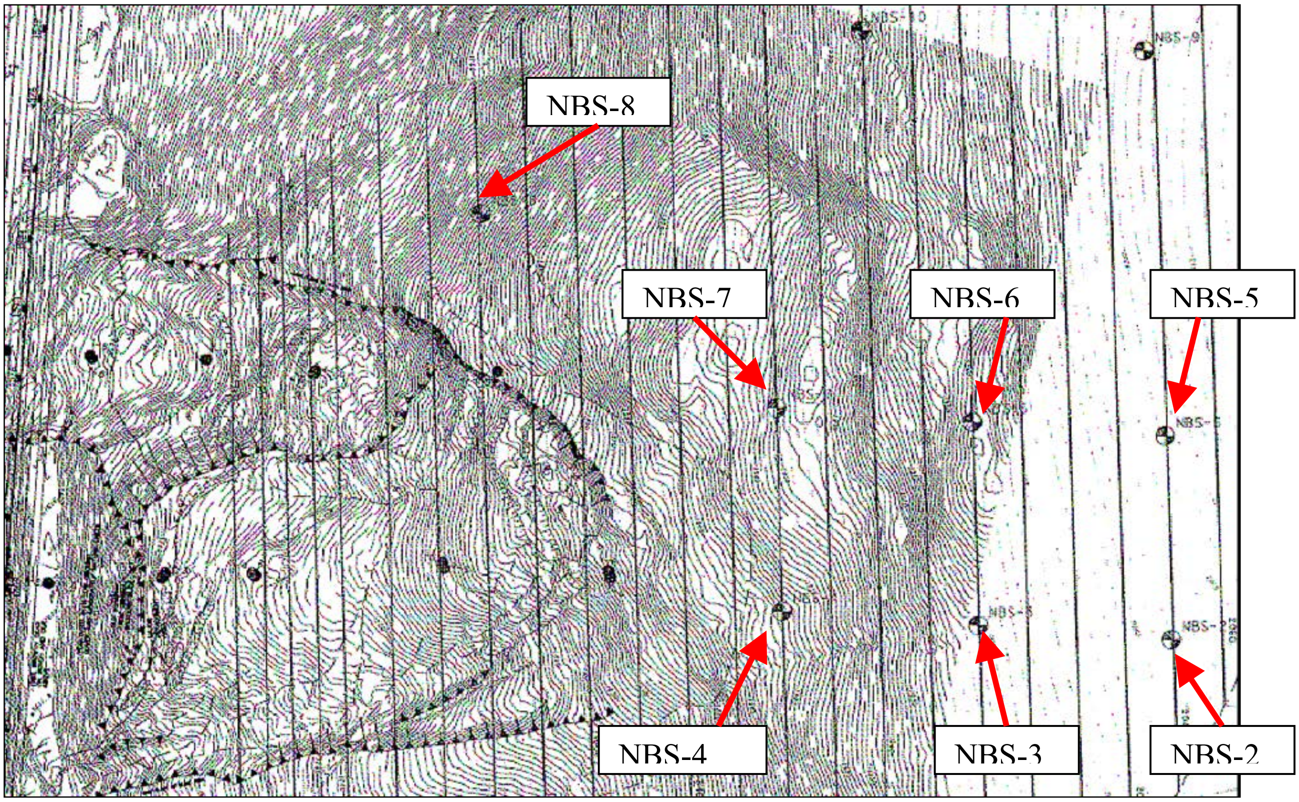

Figure 8 – Close-up view of boring and inclinometer locations.

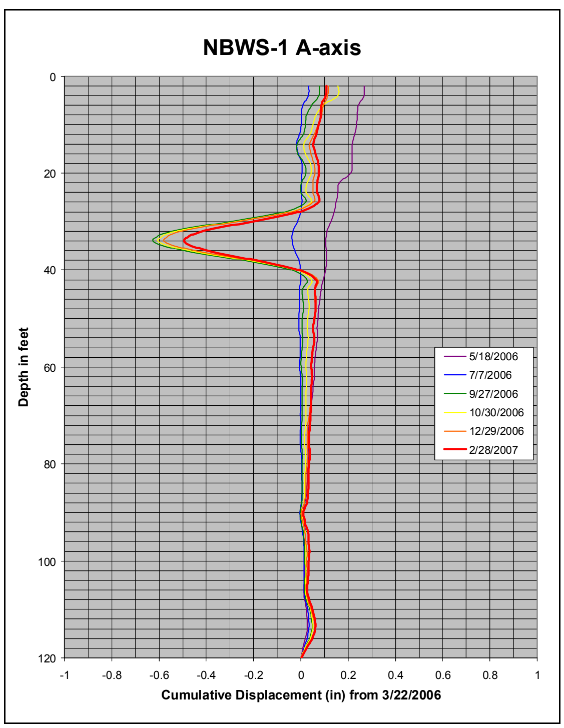

Figure 9 – Graph of inclinometer data indicating depth and amount of movement.

CONCLUSIONS

Based on the research performed so far, the following conclusions are:

1. The New Baltimore Landslide is a translational slide can be divided into an upper and lower slope. The lower slope moves at a rate of 5 to 10 (13 to 25 cm) inches per year according to previous studies, and the upper slope moves at a rate of about 1.5 inches per year.

2. Subsurface investigations and inclinometer data show that the material is sliding on a weak clayey siltstone layer located at about 63 feet (19 m) in the upper slope. The smaller area north of the turnpike has two failure planes. One located at a depth of about 30 feet (9 m) and one located at a depth of about 90 feet (27 m).

3. The New Baltimore Landslide is an active slide that poses a threat to the Pennsylvania Turnpike. When the turnpike is widened through this area, the toe needs to be cut further back which will cause a decrease in the lateral support of the slide. Remedial measures will be investigated once the stability analysis is performed.

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Crossing the Lehigh and Pohopoco Rivers: It's Not Just About the Bridges.

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ABSTRACT

The existing Pennsylvania Turnpike bridges over the Lehigh River and Pohopoco Creek near their confluence are scheduled for replacement with new structures that will offer increased traffic capacity. Alignment studies were performed to provide increased capacity for the approach roadways as well. As a result of the studies and identified impacts, the alignment will shift to the west from its present configuration.

The roadway was originally designed and constructed in the mid 1950's, with side-hill cut and fill conditions on both the north and south approaches to the rivers. The north approach is dominated by a 130-foot high rock embankment, with only a minor cut slope. The south approach is dominated by a 115-foot high rock cut slope, with a 60-foot high embankment. All of these features show signs of only marginal stability, and the new alignment adversely impacted them.

The roadway widening for the north approach originally was to have been a 47-foot high retaining wall constructed on the rock fill. When alternatives were analyzed, the projected cost of this wall was on the order of $2.5 million and measures to ensure global stability could double the cost. Subsequently, it was decided not to build the entire template width north of the bridges, and shift the alignment of the wall to minimize its height and impact on global stability.

The roadway widening for the south approach includes construction of a new rock fill with a 1.5:1 slope and a tiered retaining wall through the rock cut slope. The existing fill slope has incidental walls, slope paving, and horizontal drain outlets that will have to be dealt with during construction. The rock cut slope has structural discontinuities that need to be considered in design. It is proposed to construct the cut slope with rock anchors and soil nailing. The final product is to include a BoulderScape decorative finish.

While the three major roadway approach issues were all for one project, each merited individual attention which resulted in a different solution for each.

INTRODUCTION

Just north of the Lehigh Tunnel, the Northeast Extension (I-476) of the Pennsylvania Turnpike crosses the Lehigh River and the Pohopoco Creek between Milepost A-73.61, approximately 16.4 miles north of Lehigh Valley Interchange, No. 56, and Milepost A-75.29, approximately 0.4 south of the Mahoning Valley Interchange, No.74, in Carbon County, Pennsylvania.. Originally built in the late 1950's, the existing bridges crossing the river and the creek are near the end of their life. Replacement structures have been designed, with construction anticipated in the near future.

The new bridge construction provides for the replacement of Turnpike Bridge NB-525 over the Lehigh River and Turnpike Bridge NB-526 over the Pohopoco Creek on a new location offset from the existing structures. Work will also include the reconstruction of the bridge approaches, with associated walls and drainage structures. The bridges over the Lehigh River are proposed to be dual six-span continuous steel multi-girder structures with reinforced concrete circular shaft hammerhead piers founded on drilled shafts with 12-foot diameter rock sockets. The bridges over the Pohopoco Creek are proposed to be similar four-span continuous structures on 12-foot diameter caissons.

Early in design, constraints and desired features were identified. The design was to minimize new right-of-way needs, particularly minimizing impact to environmentally sensitive areas, a potentially contaminated parcel, and a local historic district. Existing traffic flow on the Turnpike was to be accommodated with a minimum of disruption. The project was to anticipate an ultimate six-lane configuration.

The roadways south of the Lehigh River crossing and north of the Pohopoco Creek crossing are both sidehill cut and fill sections. These roadways were constructed to mid-1950's standards and have performed well to date, although the cuts have weathered and the embankments appear to be only marginally stable. However, relocation and widening of the roadway resulted in major concerns at three areas: the embankment south of the Lehigh River, the cut south of the Lehigh River, and the embankment north of Pohopoco Creek. This paper addresses how those concerns were met.

THE EMBANKMENT SOUTH OF LEHIGH RIVER

The existing roadway embankment is constructed of random earth materials, with slopes locally steeper than 1.5H:1V. Portions of the embankment have seepage drains and surface treatments to increase stability. Widening of the roadway at the south end of the project extends the template beyond the existing embankment shoulder. Therefore; a wall, an embankment, or some combination thereof is needed in this area.

Walls were investigated at the top and at the base of the slope. A 1.5:1 rock fill also was proposed, however this required relocation of a hiking trail and additional drainage structures. Comparison of the alternatives indicated that the rock fill was preferred in spite of these impacts.

A 60-foot high rock fill was designed using standard benching, but it required additional right-of-way and coordination with effected parties. Other incidental construction was required as the hiking trail was relocated to accommodate the fill.

THE CUT SOUTH OF LEHIGH RIVER

The existing cut slope consists of sandstone, siltstone and shale from the Walcksville Member of the Catskill Formation. Slope stability is primarily structurally controlled, with periodic ratings since the mid 1980's typically placing it in the moderate hazard range. With construction of the new Lehigh River Bridges west of the existing structures, the roadway template was pushed further into the hillside.

The rock slope was field mapped, with prominent joint sets identified. Borings were taken to fully define the stratigraphy. A new cut slope was briefly considered, but would have required significant additional right of way. Various walls were considered, with a soil-nail wall being the selected alternative. Plans were developed for this alternative, including a Boulderscape finish.

THE EMBANKMENT NORTH OF POHOPOCO CREEK

The existing roadway embankment is a rock fill in excess of 100 feet high, with slopes as steep as 1:1. There are no plans or construction records to indicate what, if any, embankment foundation preparation was performed to ensure stability. The existing stability was assumed to have a safety factor slightly greater than 1.

With the six-lane template, a wall nearly 50 feet high was necessary at the top of this slope. Embedment, width and slope improvements required to achieve a 1.5 safety factor increased the cost of the wall to more than $5,000,000. It was recognized that reducing the roadway template at the north end of the project to accommodate just four lanes would allow significant reduction in wall height.

Alternatives for this four-lane template were developed, including a cast-in-place reinforced concrete cantilever wall, an MSE wall and a T-Wall. Construction of a 32-foot high "T-Wall" retaining wall is now planned. The estimated cost of this wall is less than $2,500,000.

CONCLUSIONS

Evaluation of the specific major geotechnical concerns on the project resulted in selection of a different solution in each instance. This case history demonstrates the importance of addressing geotechnical concerns on an individual basis even within a project of limited length.

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US-89 Kanab to Kanab Creek, Utah
A Geotechnical Engineering Case Study

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ABSTRACT

US-89 is the main access road from the Salt Lake Valley and I-15 into Kanab, Utah. The highway passes through some of the most scenic rock formations in Utah on its way to the North Rim of the Grand Canyon. North of Kanab, the highway follows the tight meanders of Kanab Creek creating blind corners and narrow shoulders along the highway. Because of the potential safety hazards to motorists, Utah Department of Transportation has recommended straightening and realignment of about three miles of the highway just north of Kanab. These improvements will require the excavation of soil and rock slopes and soil embankments above Kanab Creek.

Along this short reach, US-89 cuts into rock units of the Triassic Lamb Point Tongue Member of the Navajo Sandstone, and siltstones and sandstones of the Kayenta Formation and the Moenave Formation. In general, the bedrock units consist of sub-horizontal beds of sandstone, siltstone and mudstone with sub-vertical joints. Toppling failures of the sandstone have occurred as a result of the differential weathering of the weaker siltstone and mudstone beds underlying the more resistant sandstone. Depending on the geologic structure, planar and wedge failures may also be possible. In some areas, rockfall containment appears inadequate (UDOT recommends 99 percent catchment for design slopes).

During November 2006, a team of Kleinfelder geologists/engineers conducted horizontal window mapping and vertical window mapping using mountaineering techniques (climbing and rappelling) at existing rock cut slopes along the roadway alignment. The authors also performed a visual evaluation of the existing slopes on the Kanab Creek side of the highway to establish if the slopes were underlain with rock and looked for geomorphic signs and conditions for slope failure. The team completed slope design recommendations based on the rock and soil information collected, kinematic slope stability, limit equilibrium slope stability, and rock fall catchment area design.

The biggest challenge during the project was designing stable cut slopes and rockfall catchment areas while considering right-of-way, environmental, archeological and historical issues.

INTRODUCTION

Project Description

Figure 1: US89 North of Kanab, Utah

US-89 is the main access road from the Salt Lake Valley and I-15 into Kanab, Utah. The highway passes through some of the most scenic rock formations in Utah on its way to the North Rim of the Grand Canyon. North of Kanab, the highway follows the tight meanders of Kanab Creek creating blind corners and narrow shoulders along the highway. Because of the potential safety hazards to motorists, Utah Department of Transportation elected to straighten and realign about three miles of the highway just north of Kanab. The section stretches from Mile Post (MP) 64.8 just inside the city limits of Kanab to MP 68.4 by the bridge over Kanab Creek. These eventual improvements will require excavation of soil and rock slopes.

Local Geology

Along this short stretch, US-89 cuts into rock units of the Triassic Lamb Point Tongue Member of the Navajo Sandstone, the Kayenta Formation and the Moenave Formation. Sargent and Philpott, 1987 (1), characterize the main body of the Navajo Sandstone as a white to reddish-orange, cross-bedded, medium to fine-grained sandstone. The Lamb Point Tongue Member of the Navajo Sandstone is characterized as a grayish white to grayish orange, fine-grained, cross-bedded sandstone. This unit interfingers with the Kayenta Formation. The Kayenta Formation is characterized as a reddish-brown to pale-red siltstone and mudstone with very thinly bedded to laminated sandstone (1).

In general, the bedrock units consist of sub-horizontal beds of sandstone, siltstone and mudstone with sub-vertical joints. Toppling failures of the sandstone have occurred as a result of the differential weathering of the weaker siltstone and mudstone beds underlying the more resistant sandstone. Depending on the geologic structure, planar and wedge failures may also be possible. In addition, bearing failures may occur in the weaker units from the overlying sandstone. Raveling is also a problem with the relatively long 1:1 siltstone and mudstone road cuts with interspersed fractured sandstone layers.

FIELD INVESTIGATION

Rock Slopes

During November 2006, a team of Kleinfelder geologists/engineers conducted field mapping at existing rock cut slopes along the roadway alignment of US-89 north of Kanab, UT; in accordance with *The FHWA Rock Slopes Reference Manual* (2). Much of the information collected during the outcrop mapping activities dealt with the condition of discontinuities within the exposed rock masses. Key mapping windows were established for horizontal and vertical mapping traverses. This was facilitated by employing mountaineering techniques such as rappelling to complete the vertical mapping traverses (Figure 2). Since the bedding and lithologic contacts within the rock mass are sub-horizontal, mapping by simply a horizontal scan line will miss critical geological elements of the overlying strata. By rappelling along the vertical scan lines; the team obtained the other geological and geomechanical information that is critical to assessment and design of the rock slopes.

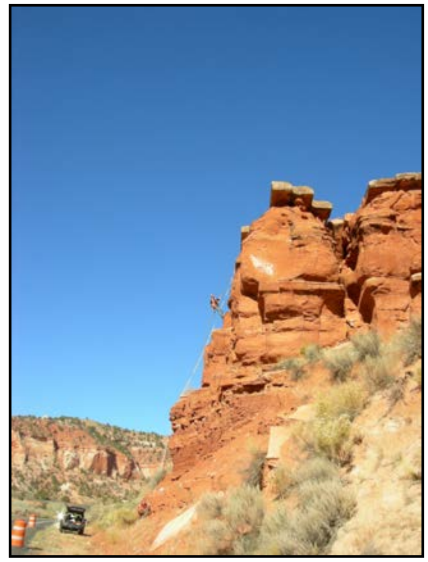


Figure 2: Mountaineering Techniques for mapping near MP 68.1

By review, discontinuity information that typically is collected for an investigation such as this includes the following:

- Location of the discontinuity in question
- Type of discontinuity
- Discontinuity orientation (dip and dip direction; Figure 3)
- Discontinuity persistence
- Discontinuity termination
- Discontinuity aperture width
- Discontinuity filling (or lack of)
- Discontinuity wall strength
- Discontinuity surface roughness and shape
- Discontinuity waviness, wavelength, and amplitude
- Barton's JRC value
- Presence or lack of water
- Discontinuity spacing

Figure 3: Kami collecting discontinuity orientations

In addition, the design engineer requires information on the rock mass including:

- Locality type
- Slope length
- Slope height
- Rock mass color
- Rock mass grain size
- Intact rock uniaxial compressive strength
- Rock mass fabric
- Rock mass block size
- Rock mass state of weathering
- Number of discontinuity sets

Figure 4: Kanab Creek cutbank evaluation

Cutbanks

Headward erosion of the slopes above Kanab Creek appears to impinge upon US-89. During the initial field investigation, the authors also performed a visual reconnaissance of the existing slopes above Kanab Creek on the west side of the highway (Figure 4). Kleinfelder's reconnaissance included evaluation of the slope geometry and geomorphic signs and conditions for slope instability. In addition, the team ascertained the composition of the slopes and the approximate location of bedrock.

The Kleinfelder team followed the initial reconnaissance by completing six exploration soil borings at the top of the cut bank slopes on the shoulder of the south bound lane of US-89 (Kanab Creek side) to evaluate the soil conditions underlying the roadway, to identify potential landslide shear planes, and to establish the depth to bedrock. Hollow-stem auger borings were advanced to a maximum depth of 100 feet or refusal at bedrock.

DESIGN METHODLOGY AND DISCUSSION

The first part of the rock slope design entailed compiling the data collected during the field reconnaissance. The Kleinfelder team looked for trends in the data to relate the strength properties to the different lithologies within the project area so that in places where actual strength data was not collected, material shear strengths, and weathering properties could be inferred. One for the primary keys to the rock slope design is the geomechanical rock mass classification.

Geomechanical Rock Mass Classification

The rock mass classification is a means to characterize the geomechanical characteristics of the rock mass and is accomplished using the field data and therefore it is more of a design tool than actual field data collection. However, the techniques force the investigator to look at and touch the rock up close. Two of the more widely accepted classifications systems are the Rock Mass Rating System (RMR) by Bieniawski (3) and the Geological Strength Index (GSI) from Hoek and Brown (4).

The RMR, also referred to as the geomechanics classification system, is based on the algebraic sum of six rock mass property ratings, namely:

- Strength of intact rock material
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities relative to the excavation or rock slope

Hoek and Brown and other subsequent investigators developed the GSI classification system after they established that the RMR alone was inadequate for relating failure criterion to geological observations in the field, especially for weak rock masses (4, 5, 6, 7, and 8). In addition, for slopes the RMR required unrealistic rating adjustments for discontinuity orientation (7). The GSI provides a technique for estimating the reduction of rock mass strength in differing geological conditions. The values are related to both degree of fracturing and condition of the fracture surfaces (8) and are selected from tables developed by Hoek and others.

Bieniawski's RMR classification can be related to the GSI. If the 1989 version of Bieniawski's RMR classification is used, the GSI = RMR_{89}' – 5 where RMR_{89}' has the "Groundwater" rating set to 15 and "Adjustment for Joint Orientation" set to zero.

Deere (9) developed the rock quality designation (RQD) technique, which is simply estimated from percent of rock core recovery ≥ 10 cm (4 inches) in length compared to the total run. However, in the field one may not have access to drill core. Therefore, Planström (10) developed a technique to estimate RQD while evaluating the rock face (Figure 5), where:

- RQD % = 115 – 3.3Jv,

In boreholes, RQD is a directionally dependent parameter and its value may change systematically depending upon borehole orientation. Therefore, using the volumetric joint count by Planström is useful in reducing this directional dependence. To evaluate J_v in the field one must select an open cleft in the rock which displays x, y and z dimensions. One then sums all the fractures along a 1-meter length in each of the 3-dimensions to obtain a volumetric joint count. The RQD is used in estimating the RMR values.

Figure 5: Chad estimating RQD

Along this stretch of US-89, the rock units are composed of primarily sandstone and siltstone, and local interbedded conglomerate layers. The sandstones range from weak to moderately strong rock and from very thinly bedded strata to massive prominent cliff forming units. RMRs for the sandstone range from 42 to 62, indicating fair to good quality rock. Weak to very weak interbedded siltstones crop out in some locations forming the more gentle slopes. Estimated RMRs for the siltstone units range from 40 to 45, indicating fair quality rock. A conglomeratic rock unit crops out in one location along the highway. Strength of the conglomerate varies based on the clast size and cementation ranging from weak to moderately strong. In some locations it tends to be moderately fractured and other locations massive. Estimated RMR for the conglomerate was about 49, indicating fair quality rock.

For design purposes, the team selected the following Hoek-Brown coefficient ranges for the rock units:

- Sandstone GSI = 50-60
- Sandstone m_i (intact rock constant) = 17
- Siltstone GSI = 30-40
- Siltstone m_i (intact rock constant) = 7
- Disturbance Factor, D = 0.7

Kanab Creek Cutbank Evaluation

Headward erosion and landsliding of reworked terrace deposits between US-89 and Kanab Creek have impinged on and jeopardized the stability of southbound lane of US-89.

Kanab Creek parallels the western aspect of US-89 along this stretch of the project between mile post (MP) 65.0 and MP 68.3 at the bridge crossing. Kanab Creek is a mature meandering stream with classic point bars and cutbanks. The cutbanks have formed as a result of headward erosion of the stream at the head of the meander loop into older terrace deposits. Typically during flood stage, the stream erodes the toe of the slope on the outside of the stream meander thus steeping the slope, removing lateral support and creating unstable conditions; sloughing and larger landslides towards the creek then ensue (Figures 6 and 7). Near MP 65.5 the team observed a small landslide (about 10,000 cy) in the terrace deposits. The headscarp of the slide was within five feet of the southbound shoulder of US-89. The landslide demonstrated typical morphology of landslide such as a bulbous toe, hummocky terrain, and a steep headscarp. Relief between the shoulder of the highway and the creek ranges between 100 and 200 feet.

Figure 6: Landslide present near MP 65.5

Based on investigation of the surficial materials and borehole logs, the terrace deposits consist primarily of silty sands (SM) to poorly graded sand (SP) with some minor interbeds of clays (CL). At the surface the deposits are very weakly cemented with low shear strengths. Drive blow counts demonstrate that the relative density increases with depth.

Borehole logs indicated no apparent evidence of a basal landslide shear zone or shears within the clayey strata. The reason the team did not see evidence of shearing may be because the boreholes were installed beyond the limits of the landslide headscarp or cutbank at the shoulder of the road. In addition, the noncohesive nature of the sandy deposits will typically not preserve evidence of shearing. Conversely, evidence of the shears would be more apparent in the cohesive clayey zones and the team observed no apparent shears.

Figure 7: Cutbank near MP 66

Near MP 66.25 the cutbank between the road and Kanab Creek consists of primarily massive sandstone with up to ten feet of sandy fill in some locations above the rock (Figure 8).

Stereonets and Markland Analyses

Figure 8: Cutbank near MP 66.25

One of the first steps to evaluate the stability of a rock slope is to evaluate the kinematic relationships between the major fracture sets and the slope face. Stereonets are a standard tool for evaluating the structural and kinematic relationships of the fracture sets. Where the stability of a rock cut is controlled by the structure of the rock mass, a Markland analysis was used to estimate the kinematic potential for rock blocks to fail out of the existing or proposed slopes. The information required to perform an analysis are the design slope dip and dip direction, the orientation of the discontinuities within the rock mass, and the friction angle of the discontinuities. A kinematically potential wedge failure is identified when a point defining the line of intersection of two planes falls within the area included between the great circle defining the slope face and a circle defined by the angle of friction, ϕ. A planar failure is a specialized form of a wedge failure that follows the same criteria above and also must fall within ± 20° of the dip direction of the slope face.

Using the discontinuity data the team collected at existing rock outcrops, pole plots were constructed on equal area stereonets facilitated with the computer programs Dips® Version 5.0 by Rocscience and ROCKPACK III by C. F. Watts (11). Both poles and dip vectors were plotted. The poles tend to accentuate the orientation of steeply dipping discontinuities while the dip vectors lend themselves to performing Markland analyses.

The Markland analysis does not consider a cohesion intercept when modeling the strength of discontinuities. This method also assumes that the discontinuities are continuous and through going with no “bridging” within the discontinuity. The effect of “bridging” would allow a tensional component (or cohesion intercept) of discontinuity strength. The Markland Analysis assumes that the factor of safety of individual rock blocks may be estimated as follows. When the dip of a discontinuity or the plunge of the line of intersection is greater than the friction angle, the factor of safety is less that 1.0. When the dip of a discontinuity or the plunge of the line of intersection is less than the friction angle, the factor of safety is greater than 1.0. In either case, the dip or plunge has to be less than the dip of the slope face, or the structure will not daylight the slope.

The team assumed rock discontinuity friction angle of 30 degrees based on the geomechanical information that was collected in the field, experience with similar rock types and guidance from the FHWA Rock Slopes Reference Manual. Based on the results of the analyses, the team provided recommendations for the slope ratio so that the slope is kinematically stable. Figure 9 is a Markland analysis displaying the relationships between the discontinuities and the slope face.

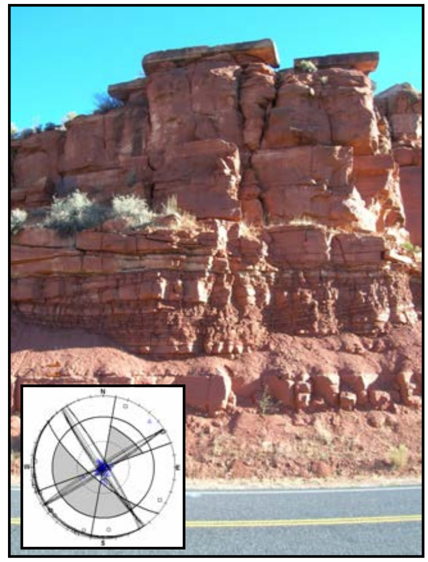


Figure 9: Cut slope and stereonet from slope near MP 68.1

Toppling failures of the sandstone may occur as a result of the differential weathering of the weaker siltstone and mudstone beds underlying the more resistant sandstone. Raveling may also be a problem with the relatively long 1H:1V siltstone and mudstone cuts with interspersed fractured sandstone layers. These potential failures can generate small-scale rock falls that will be contained in the proposed catchment areas.

Rock Fall Hazard Mitigation and Catchment Area Design

The rockfall catchment area is the zone between the shoulder of the road and the toe of the rock cut slope used to restrict and catch rockfall (12). As part of the design, the team performed computer modeling rock fall simulations for each cut location and proposed catchment area. To facilitate the investigation, the team employed the Colorado Rockfall Simulation Program (CRSP) for the analysis at the critical station (maximum height) for each rock cut slope section. In the simulations, the team modeled the average and maximum spherical (worst case) rock blocks expected to roll down the slope face. For each simulation, the model rolled 500 rock blocks. Normal coefficients, tangential coefficients, and surface roughness values were chosen for the CRSP runs based on field observations and recommendations from the CRSP Manual for different slope descriptions related to slope lithology (13). The team evaluated the proposed catchment area for catchment ranges between 90 and 99 percent.

Design and Construction Challenges

This project presented some major design challenges that are not uncommon to other road design and construction projects. Design and construction are invariably constrained by some environmental, archeological or historical limitation which intern may limit the geometry of the rock cut and the size of the rockfall catchment areas bordering the highway. In addition, existing cutbanks encroaching on roadway required design either of a retaining structure to harness the erosion or movement of the road alignment to accommodate the encroaching creek.

The area along Kanab Creek has been reported to be a habitat for the Northern Goshawk (*Accipter gentilis*). The Goshawk is a large, tough raptor which is known to have few enemies. Typically they nest in tall trees such as the Ponderosa Pine, building large stick nests. They defend against any would-be foe intruding on their space. In Utah, the Northern Goshawk is a

Species of Special Concern. A construction window is generally set up around the nesting period, May 1 through August 31, and work schedules are planned accordingly.

Native American artifacts are protected by the federal government. It is not uncommon to find evidence of encampments or scatter of fragmented artifacts along a water source such as Kanab Creek. Areas that have show evidence of potential fragments of artifacts must flagged and be thoroughly investigated. Designs of the road cuts may have to be modified such that the flagged site is not disturbed. The challenge is cutting the slope such that kinematically stable and maintaining a rock catchment zone that will retain at least 90 percent of the rockfall.

The National Historic Preservation Act of 1966 was enacted to preserve our national heritage. The act requires that one make every effort to preserve a structure that they may encounter say during construction of a highway. In the vicinity of MP 66, the team identified some historic foundations and apparent retaining walls. It was not totally clear as to what function these structures had serviced, however, it was clear that someone had taken the time to build them. Because of the potential historical significance, the team again was forced to design the rock slope below the historical structures that was steeper to avoid the site yet kinematically stable.

One of the challenges on any rock engineering project along a highway is to design the rock cut such that it is kinematically stable yet steep enough that rock fall will fall close to the toe of the slope. In states where rock fall is a prevalent problem along the highway, most have agreed that 90 percent catchment is an achievable goal from a cost standpoint. Further more legal counsel for both Oregon DOT and Caltrans have advised that judges, juries and the public understand that because of limited funds and resources, public transportation agencies can not be expected to correct every rockfall deficiency immediately and can not design a catchment system for 100 percent hazard reduction (12). In addition, they further advise that designing to less than 100 percent is legally defensible as long as it is established agency policy and accomplished as part of a rational slope/rockfall assessment (12). Furthermore, 100 percent catchment is statistically impossible. The typical goal for most state DOTs is 90 percent catchment. In all cases the team designed the slopes and catchment zones to retain at least 90 percent of the rockfall with a goal of 99 percent catchment. Costs, right-of-way, environmental, archeological and historical issues constrained the design of the cutslopes and catchment zones and the goal for 99 percent retention.

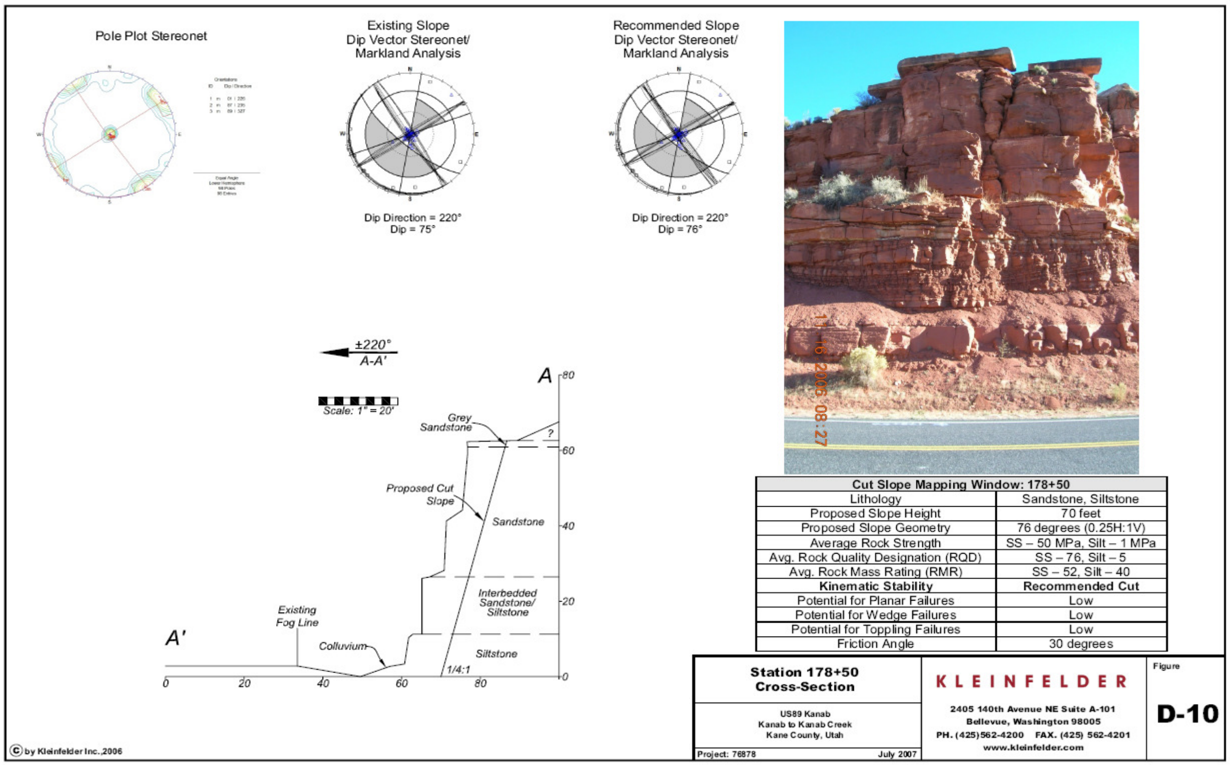
SLOPE RECOMMENDATIONS

Rock Cut Slopes

Figure 10 is an example of a mapping window displaying of how the authors displayed the data in a report. The goal is to develop each figure to stand alone as a tare sheet. The figure typically includes a photo of the slope, a profile and cross-section displaying the geology, a stereonet with Markland analysis, geomechanical information on the rock mass and cutslope recommendations.

Based on the geologic conditions, rock mass characterization interpretation of the stereonet and stability analyses, the recommended rock cut slopes ranged from 1H:1V (45 degrees) to 0.25H:1V (76 degrees). For simplicity and cost, the cut slopes inclinations were designed to be stable without the use of rock reinforcement such as rock bolts, dowels or shotcrete. As part of the slope design, soil and colluvium encountered during construction at the top of the cut slopes were trimmed to a stable inclination of 2H:1V (27 degrees).

In general, for most of the cut slopes, the proposed catchment area width was designed for 30 feet which included a six foot wide shoulder, a 13-foot 6H:1V catchment down slope and an 11-foot 4H:1V up slope prior to the base of the proposed cut slope. The catchment area width in a cut slope section near MP 66.25 was reduced to decrease the impact to sensitive historical areas on the cut slope. Based on CRSP analyses, the catchment areas will provide between 90 and 99 percent catchment from the proposed cut slopes.



Cut Slope Mapping Window: 178+50	
Lithology	Sandstone, Siltstone
Proposed Slope Height	70 feet
Proposed Slope Geometry	76 degrees (0.25H:1V)
Average Rock Strength	SS – 50 MPa, Silt – 1 MPa
Avg. Rock Quality Designation (RQD)	SS – 76, Silt – 5
Avg. Rock Mass Rating (RMR)	SS – 52, Silt – 40
Kinematic Stability	**Recommended Cut**
Potential for Planar Failures	Low
Potential for Wedge Failures	Low
Potential for Toppling Failures	Low
Friction Angle	30 degrees

Figure 10: Example of Cut Slope Mapping Window Data Presentation

Kanab Creek Cut Bank Slopes

The cutbanks and slopes within the terrace deposits between US-89 and Kanab Creek are marginally stable. Triggers which will induce the slope to fail include: flooding of the Kanab Creek, high groundwater, seismic activity and surcharging. Headward erosion will continue undercut the shoulder of the road because of the weak, noncohesive nature of the terrace deposits. To mitigate the marginally stable slopes and protect the road from being undercut by erosion, the following alternatives were considered:

1. Shift the road alignment to the east
2. Construct a retaining wall parallel to the road
3. Construct a rock buttress and keyway at the face and toe of the landslide

Based on right-of-way, flood plane and environmental issues and cost, the authors felt that the most appropriate option to mitigate against cutbank encroachment was to shift the roadway east away from the existing slopes by a few feet. Fortunately, within these stretches of the highway, there was sufficient room and right-of-way to move the road alignment at least ten feet to the east. The erosion of the shoulder of the road can be minimized by simply moving the alignment to the east by a few feet. Based on reported headward erosion rates, the authors assumed the average local slope erosion rates along Kanab Creek were approximately 0.3 feet per year. Simply shifting the road alignment about eight to ten feet to the east can extend the life of the road by about 25 years.

CONCLUSION

The biggest challenge during the project was designing stable cut slopes and rockfall catchment areas while considering client needs, cost, right-of-way, environmental, archeological and historical issues. The team conducted an in-depth geomechanical investigation of the existing cut slopes to address the project challenges. Since the bedding and lithologic contacts within the rock mass are sub-horizontal, mapping by simply a horizontal scan line at the road level may have missed critical geological elements of the overlying strata. By rappelling along the vertical scan lines; the team obtained the other geological and geomechanical information that is critical to assessment and design of the rock slopes. In working closely with the civil engineer and Utah State and Regional Department of Transportation offices, recommendations were made that provided stable cut slopes and fit within the issues and constraints along the highway corridor.

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GEOTECHNICAL CHALLENGES POSED BY WEAK CLAYSTONE IN DEEP CUT SLOPES

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Disclaimer

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ABSTRACT

Site characterization for a new 5.4-mile-long railroad alignment traversing mountainous terrain in western Pennsylvania indicated the presence of non-durable claystone bedrock of the Pennsylvanian Conemaugh Group. The excavation for the new alignment would expose generally horizontally-bedded coal measure rocks exhibiting highly variable engineering properties. The sequence of competent sandstone, siltstone and shale interbedded with weak, massive claystone (Red Beds) is responsible for numerous landslides and rockfalls in the region due to undercutting by differential weathering and general instability of the weak claystone layers. However, right-of-way restrictions on this project necessitated designing cut slopes significantly steeper than traditionally utilized in this material. As a complicating factor, slickensided discontinuities, laterally discontinuous strata, and erratic topography suggested the possible occurrence of pre-existing deep bedding plane shear surface(s) that presumably developed as the result of valley stress relief and elevated pore pressures related to Pleistocene glaciation. Excavation of the proposed railroad alignment would extend to depths of up to 150 feet with the potential for reactivating ancient landslides and/or triggering new slope movements. The design approach involved fitting the new cut slopes within the available right-of-way by developing a slope protection system to prevent degradation of the claystone bedrock. Additionally, sub-horizontal drains were installed to lower groundwater levels, and an instrumentation program, consisting of inclinometers and piezometers, was carried out to monitor slope movements and water levels during construction. As expected, some slope stability failures did arise during and after construction, however these were generally minor and thus could be dealt with primarily as a maintenance issue.

PROJECT DESCRIPTION

This project, which involved the first new freight railroad route to be constructed in the Eastern United States in recent years, was completed in Summer 2006. The project site is located approximately 40 miles east of Pittsburgh, Pennsylvania in Indiana County. The new 5.4-mile rail alignment was built for the purpose of creating a shorter route for the 130-car coal trains that serve the Keystone Generating Station in Shelocta, PA on a daily basis. The project involved significant earthwork due to the relatively mountainous terrain through which the new alignment traverses at only a 1% maximum vertical grade. Cuts of up to 150 feet deep, involving approximately 1.5 million cubic yards of excavated material, and embankments of up to 45 feet height were needed to establish the vertical grade. In addition, the new alignment necessitated one at-grade roadway crossing and two multi-span bridges for grade separation over two state routes and the adjacent streams. Two soil nail walls were constructed in order to retain a combined cut face area of 1,004 square yards, utilizing a total of 13,745 linear feet of soil nails. An extension of the soil nail wall technology was the innovative Shotcrete Slope Protection System (SSPS), which was applied to weak rock strata (i.e., claystone) that decomposed rapidly when exposed to the elements.

GEOLOGIC CONDITIONS

Overview

The project site is situated within the Pittsburgh Low Plateau Section of the Appalachian Plateaus Physiographic Province. This region is characterized by smooth undulating topography that is dissected by narrow and relatively shallow valleys. The local relief in this area is strongly influenced by the variable weathering resistance of the sedimentary rock units, whose bedding is typically gently folded and nearly horizontal. The project is underlain by the Pennsylvanian-aged Conemaugh Group, which consists of a cyclic series of sandstone, limestone, coal and mudrocks (i.e., shale, siltstone, and claystone). These “coal measure” deposits developed in an inland sea that was in the process of undergoing major shoreline transgressions and regressions (Wu, et al, 1987). Owing to this dynamic depositional environment, the stratigraphic section generally exhibits considerable lateral variations in lithology.

The Conemaugh Group is subdivided into the Casselman and Glenshaw Formations, which are separated by the Ames Limestone marker bed located at the top of the Glenshaw Formation. Figure 1 presents a geologic map of the area with the new railroad alignment indicated. The Conemaugh Group constituents are described as interbedded, strong and weak sedimentary rocks with considerable horizontal and vertical variation in strength and deformability but with a general tendency for vertical repetition of behavioral characteristics (Hamel, 1998). Structural features of the area include the Elders Ridge Synclinal Axis (to the northwest) and the Jacksonville Anticlinal Axis (to the southeast), both of which are oriented on a southwest-northeast trend and with rock bedding dipping at about 3 degrees to the northwest along the project corridor. The site soils are comprised of colluvial deposits and residual materials derived from weathering of the underlying bedrock.

A characteristic feature of the Conemaugh Group is the presence of mudrocks (particularly massive claystone) that have undergone oxidation to impart a predominantly red color. The local term "Pittsburgh Red Beds" is often used to refer to these weak, highly erodible claystone units that disintegrate rapidly upon exposure to form red-brown sandy, silty clay of medium plasticity (Hamel and Flint, 1972). The red beds are usually penetrated by a myriad of randomly oriented, closely spaced fractures, often with slickensided surfaces. This material is a troublesome component of many engineering projects and has historically caused numerous slope stability problems throughout the region (Ackenheil, 1954; Gray, et al. 1978; Hamel and Flint, 1972; Wu, et al. 1987, and Hamel and Adams, 1981).

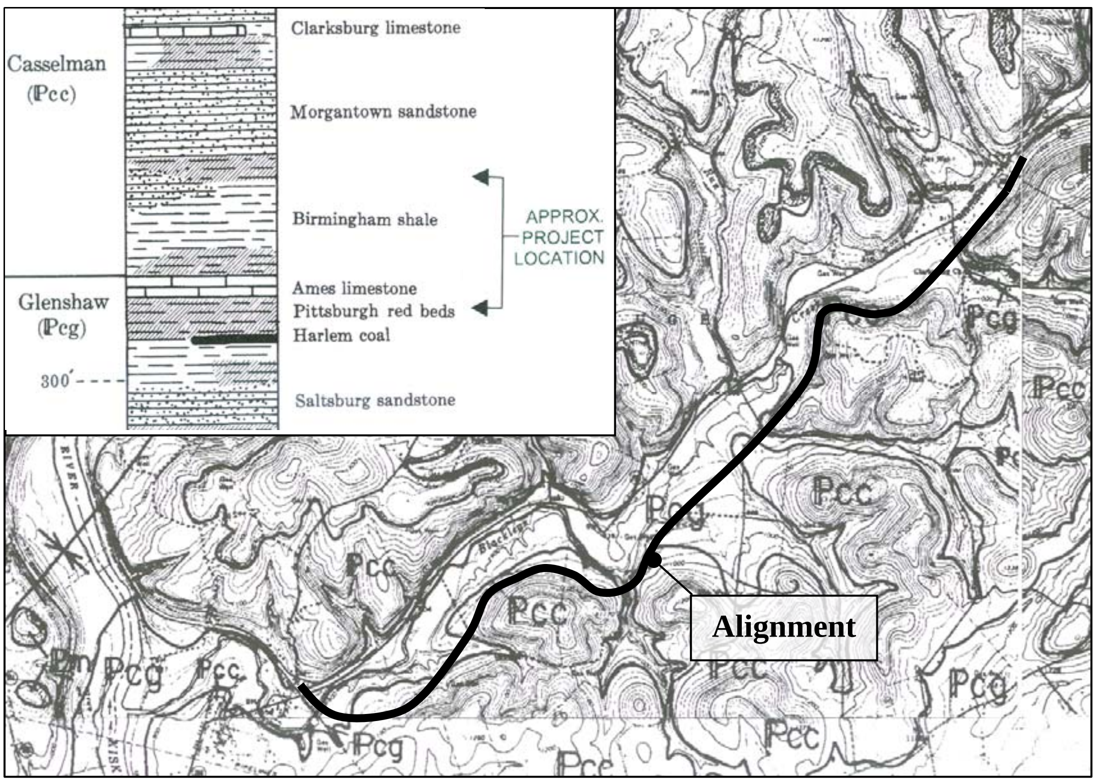

Figure 1 - Geologic Map of the Project Area

During the Pleistocene Period, the southward advance of continental glaciers terminated roughly 45 miles to the north of the project site. However, the surrounding area was subjected to a periglacial climate and heavy precipitation. Additionally, the ice sheets blocked north-flowing streams in the region, causing significant reservoirs to develop in many of the valleys. The elevated pore pressures in conjunction with lateral stress relief associated with valley down-cutting resulted in widespread slope failures of the valley walls (Ferguson and Hamel, 1981). In many cases, the slope movement is believed to have occurred along deep bedding plane shear zones at or near the base of a weak rock interval (Hamel and Adams, 1981). When the shear strain exceeds that which can be internally absorbed through elastic deformation, a bedding plane shear zone may develop within the claystone and these are typically accompanied by randomly oriented and curved slickenside surfaces as a secondary feature.

Test borings for this project were consistent with the published descriptions of the regional geology, as outlined above.

Potential for Deep-Seated Slope Instability

During the subsurface exploration phase of the project, several observations suggested that pre-historic deep-seated slope movements may have occurred at the project site, most notably in the vicinity of the proposed deep cut section at the far southwestern end of the alignment. Firstly, the rock core from this area included several highly polished, randomly oriented, and curved slickensides within zones of weak claystone. These are thought to be a secondary feature associated with bedding plane (basal) shear movements. Secondly, the core revealed zones of laterally discontinuous stratigraphy between adjacent borings. And finally, the valley sides exhibited abrupt changes in slope angle, which is often associated with prior slope movements. A review of aerial photographs (stereopairs) confirmed that the natural slope angle is relatively very flat between approximate elevations 900 ft and 1050 ft (coinciding with primarily claystone strata) throughout this particular area. Nonetheless, there was no direct evidence of any active tension cracking at the ground surface or of any shear offset in any of the rock cores.

Valley stress relief is considered to be a likely explanation for the slickensides, in which case any existing bedding plane shears would probably represent only small relative movements. However, if a continuous bedding plane shear zone were present, this plane of weakness could possibly give rise to a deep-seated translational slope movement in response to the excavation of soil and rock for the railroad cut. The resulting movement could range from minor relaxation to a slow-moving, large-scale basal slide.

During the design of the slopes in the southwestern cut area, the potential for reactivating a historic basal shear type of slope movement was given due consideration. Since the slickensided claystone occurred near the base of the proposed cut, a bedding plane shear movement could potentially involve a mass having a vertical thickness of several hundred feet. This realization was made more acute after reviewing case histories involving large-scale slope failures in the Pittsburgh Red Bed material throughout the region. These incidents provided stark confirmation that designing slopes cut into weak claystone demands diligence and caution. Two of the more extreme cases are highlighted below:

- On March 20, 1941, at the Brilliant Road cut in Pittsburgh, Pennsylvania, 120,000 cubic yards of material slid down onto three sets of railroad tracks, derailing a train in the process. This failure was classified as a rotational slide in which a layer of massive, slickensided claystone acted as the basal shear zone. Years earlier, a tension crack was observed to form along the top of the cut slope, extending downward into the vertically jointed Birmingham Shale. Under normal conditions, water pressure in the tension crack could dissipate quickly through the flat-lying strata without adverse impact to the underlying claystone. And despite an unsuccessful attempt to plug the crack with concrete, the favorable drainage allowed the slope to remain stable for several years. However, following a week of steady rainfall and cold weather, ice formation resulted in blocked drainage and the buildup of water pressure, leading to the massive slide. The failure surface followed a predictable path down the tension crack and out along the slickensided Red Bed layer.

- In 1968, earthwork associated with the construction of Interstate 79 (north of the Ohio River) resulted in the reactivation of old landslides within colluvium derived from the Pittsburgh Red Beds. Studies by Hamel and Flint (1969, 1972) and Hamel (1970) involved detailed mapping of numerous ancient and recent landslides above the new roadway. The authors concluded that movement along the ancient failure surfaces had resulted in the lowering of shear strengths to residual values. Additionally, the few deep-seated ancient slides that were mapped are thought to be related to bedding plane shear zones and valley stress relief jointing.

DESIGN ISSUES

The design of the rock cut slopes within this challenging geologic environment presented several problems. Site characterization indicated that the proposed excavation would expose rock layers that exhibit highly variable rates of weathering, both vertically and horizontally due to discontinuous strata. This condition made it impractical to design varying cut slope angles as a function of rock type. Additionally, the non-durable claystone rock is vulnerable to excessive erosion, slumping and debris flow, leading to undercutting of the more resistant overlying strata. Although these are surficial processes, undercutting of resistant beds (sandstone) can give rise to deeper failure modes such as rock falls and plane/wedge failures. The non-durable beds must be appropriately benched, cut back at very flat angles, or the material must be protected from the weathering processes that cause it to degrade. The 3H:1V finished slope angle that is traditionally recommended for cuts through Pittsburgh Red Beds was not acceptable for this project due to the extensive right-of-way requirements and the massive excavation quantities that this design would have required. Therefore, the primary design objective was to prohibit the degradation of non-durable rock exposed at the face of cut slopes by sealing this material with shotcrete. Shakoor (1995) indicates that the placement of shotcrete to protect non-durable rock surfaces is relatively common; however, based on discussions with specialty geotechnical contractors, this project would involve the largest known application of such as system to date.

Table 1 is a summary of the range of properties of the intact and residual claystone based on laboratory testing for this project. Note that the median slake durability value for this material is only 2%, suggesting extremely low durability following exposure to air and water in cut slopes.

Table 1 - Claystone Laboratory Test Data

Test	Range of Values	Median Value
Unconfined Compressive Strength, UC	60 psi < UC < 2450 psi	870 psi
Slake Durability, SD	1% < SD < 77 %	2%
Plastic Limit, PL	17% < PL < 37%	22%
Pasticity Index, PI	5% < PI < 21%	10%
Clay Content (≤ 0.005 mm), CC	0.1% < CC < 73.6%	18%

Slaking of these materials is attributed to pore-air compression that takes place when the material is immersed in water that is drawn into the pores as a result of capillary suction. The entrapped air in the pores causes the material to fail in tension. This mechanism is deemed to be the predominant cause of slaking in mudrocks composed primarily of non-expansive clay minerals such as kaolinite (Vallejo and Murphy, 2001, and Vallejo, et al., 1993). USDA (1991) mapping of the project area indicates that illite and kaolinite are the principal clay minerals noted in the project area soils.

The development of the SSPS enabled the design team to prepare a cut slope template that could be used throughout an entire cut section regardless of the variation in rock type encountered. The SSPS is applied to the non-durable strata to prevent this material from eroding and undermining the more resistant strata. This concept allowed the design team to daylight the cut slopes within the currently proposed right-of-way while utilizing a 0.75H:1V slope angle with intermediate benches. It should be noted that 0.75H:1V slopes are typically utilized for more resistant, but heavily fractured rock types, such as sandstone. As stated, the SSPS system only stabilizes the face of the rock cut slope. The only measure taken to improve the global stability of the cut slopes was the installation of sub-horizontal drains, consisting of 1.5-inch diameter perforated PVC pipe wrapped in geotextile and inserted into 4.5-inch diameter drilled holes. The sub-horizontal drains were drilled to a depth of 50 feet into the cut slopes, outletting near the bottom of the slopes and near the base of each intermediate bench.

The design of the SSPS is not based on global stability requirements, but rather it is based on swell pressure estimates generated at the slope face. The SSPS was designed to resist this swell pressure in the event that the claystone underneath the shotcrete should begin to degrade. Based on numerous published correlations to estimate swell potential, a design value of 950 psf was selected for design. It must be emphasized that the SSPS provides surficial protection, but would not act to stabilize a deep-seated slope movement that could potentially be triggered by the excavation operation.

In theory, the swell pressure generated by the decomposing claystone is resisted by the hardened shotcrete, which transmits the load to rock anchors drilled into the slope. The system was designed such that anchor pull-out would occur before shotcrete punching shear failure, thus allowing the relief of swell pressure without damaging the shotcrete face; note that anchor movement from this condition would be minimal. These systems generally incorporate a geocomposite material connected to weepholes in order to drain the rear of the shotcrete facing and to reduce or eliminate hydrostatic and ice forces.

The shotcrete facing is modeled as a two-way slab with the rock anchors interacting with each other to form a zone of uniformly reinforced rock. Uniform horizontal and vertical anchor spacing is preferred for ease of construction, and the vertical spacing must consider the ability of the SSPS contractor to work the slope. This project utilized a 6-foot vertical rock anchor spacing. Heights greater than 6 to 7 feet make it difficult to properly place shotcrete owing to an increase in rebound and overspray as well as a decrease in compaction if the nozzleman can not be safely lifted to place the shotcrete for the higher lifts.

The embedment depth of the rock anchors is based on the estimated pull-out resistance, which equals the load-transfer between the bearing material and grout. A minimum anchor embedment of 14.5 feet was specified for this project, which accounts for potential claystone degradation that could occur immediately behind the shotcrete. In addition, a minimum drill hole diameter of 4.5 inches was specified in order to allow rock drilling using an air-track rig, which is a very efficient rig type for drilling this type of material. With regards to the shotcrete, the project specifications required a water/cement ratio of no greater than 0.45 with minimum air entrainment of 7 - 10%, measured at the truck.

CONSTRUCTION

This project involved the installation of nearly 2,800 cubic yards of shotcrete and 2,340 rock anchors. The basic construction procedure of the SSPS is as follows: (1) excavate to the slope face; (2) drill holes, tremie grout and insert the rock anchors; (3) install the reinforcing mesh; (4) apply the shotcrete; (5) set the bearing plate, washer and nut assembly; and (6) cover bearing plate with shotcrete.

The SSPS specialty geotechnical subcontractor used a 4.5-in diameter drill bit to drill the 14.5-ft deep rock anchor drill holes at a rate of approximately three to four minutes per hole. Rock anchors consisted of No. 9, epoxy coated, all-thread bars. Fifteen verification tests were performed on sacrificial test nails prior to production work and 25 proof tests were performed on production nails for quality assurance. None of the verification or proof tests resulted in a creep or anchor pull-out failure, therefore the ultimate bond stress between the claystone and grout is unknown (but greater than the design value of 29 psi). Another quality control measure required the contractor to prepare shotcrete test panels for unconfined compressive strength and boiled absorption testing in order to verify nozzleman qualifications.

A total of 134 sub-horizontal drains were installed in conjunction with the shotcrete system in order to improve slope stability by lowering water levels in the slopes. The drain holes were drilled with the same equipment used to install the rock anchors. Flow rates of completed drains varied along the alignment from zero to 80 gal/hr, but typically ranged from 5 to 10 gal/hr during wet conditions. The sub-horizontal drains were generally most effective when installed immediately above impervious claystone layers, where seepage was most commonly observed.

The general contractor was responsible for all excavation and was required to provide the finished cut rock faces for the specialty geotechnical subcontractor to construct the SSPS system. The general contractor found it challenging to cut the weak rock slopes to the planned angles without large amounts of overbreak. The slopes were often cut back up to six-inches more than intended and there were pockets where the overbreak extended as much as three feet behind the slope face. The contractor was required to fill these deep voids with shotcrete in order to leave a fairly uniform finished surface.

INSTRUMENTATION

Although the SSPS provides critical surficial protection for the non-durable claystone, it is not designed to provide any stabilizing force against large-scale slope movements. Therefore, it was deemed prudent to utilize an instrumentation program to allow for the early detection of any slope movements that may develop during construction. Inclinometer casings and standpipe piezometers were installed in areas deemed to be potentially unstable, and three of the six inclinometer/piezometer pairs were located within the southwest cut section where an ancient basal shear was most strongly suspected (Inclinometers I-1 through I-3). The piezometers were installed to allow for correlation of groundwater levels to any possible movement and for monitoring the effectiveness of the sub-horizontal drain system. Depths of the inclinometer casings and corresponding standpipes ranged from about 95 to 165 feet, typically extending at least 10 feet below the proposed track elevation. In all cases, the casings were located approximately 10 feet beyond the crest of the proposed cut slopes.

Inclinometers I-1, I-2 and I-3 all indicated minor slope movements at a distinct zone within the claystone, roughly 30 feet above the proposed track grade (approximately elevation 950 feet). These movements are attributed to stress relaxation and blasting vibrations from excavation, and have asymptotically approached zero following the completion of construction operations. Figure 2 shows a cumulative displacement plot for elevation 952 feet in Inclinometer I-2. As shown, a total displacement of 0.5 inches had occurred as of July 2006, with the rate of movement clearly slowing down following the end of construction. It is possible that this movement has occurred along an ancient bedding plane shear

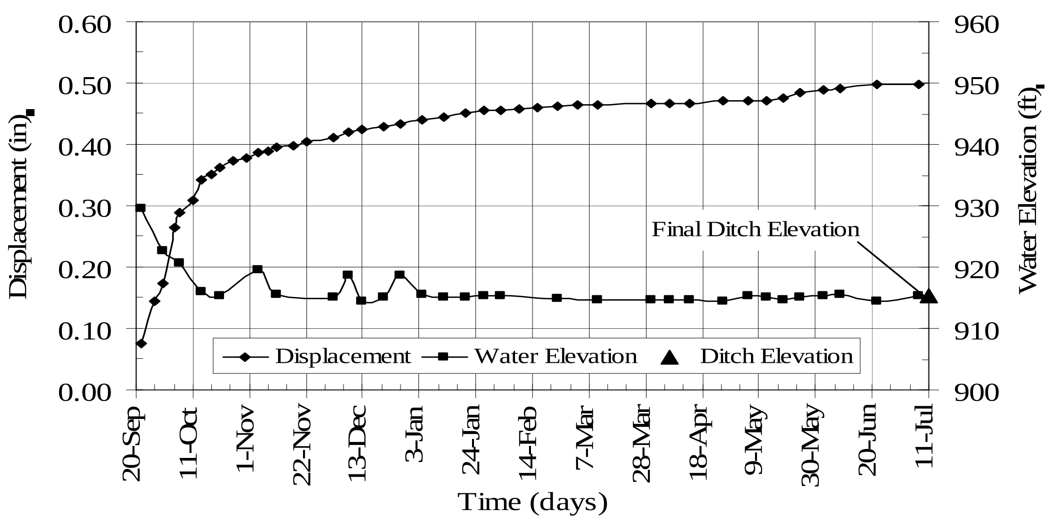

Figure 2 - Inclinometer 2, Cumulative Displacement Plot, EL. 952 (Sept 2005 – July 2006)

SLOPE MOVEMENTS FOLLOWING CONSTRUCTION

Although the creep at elevation 950 feet appears to have almost completely ceased, Inclinometers I-1 through I-3 have continued to show cycles of minor creep movement and subsequent stabilization at relatively shallow depths within the soil overburden. These periods of movement are believed to be related to the buildup of perched water following precipitation events. Following the completion of construction, soil slides occurred in the southwestern cut area in May 2006 and June 2007, and these features can be seen in the panoramic photograph in

Figure 3. Each of these slides involved the movement of roughly 5,000 to 10,000 cubic yards of material, consisting mostly of gravelly to clayey silt (colluvium and residual claystone).

Figure 3 – Panorama of SW Cut Area
(Circles represent inclinometer locations and arrows mark edges of discontinuous bed)

During the excavation of the southwestern cut, laterally discontinuous beds of sandstone and sandy siltstone were exhumed. Most notably, an approximate 500-foot-wide gap was discovered where a 6-foot-thick layer of sandy shale is laterally discontinuous. This sandy shale bed, which is present on either side of the gap, sits directly above the top of the claystone beds. The edges of this bed are visible at the lower left and right portions of Figure 3 (see arrows), and it can be seen that the recent landslides generally coincide with the gap. Also note that Inclinometers I-1, I-2, and I-3, which are spaced at 200 feet, are roughly centered about the slide area (inclinometers are numbered from right to left in photo).

It has been considered that the erratic stratigraphy could be the result of the original depositional environment. Alternatively, this particular feature may reflect an ancient slide, a more plausible scenario which would contribute to the overall instability of this area. Another aspect contributing to the instability was the fact that the cut slope design was based on the assumption that the bedrock surface was higher in the gap area, and thus there was insufficient right-of-way available to adequately flatten the slope during construction.

Following construction, inclinometer readings were generally obtained at a rate of about once or twice per month. But since both of the recent slides developed suddenly, there was no foreshadowing of these events by any of the readings. In the case of the May 2006 event (center of photograph), the slide mass was located entirely down-slope of the nearest casings (I-2 and I-3), and casing I-3 was destroyed during the slide repair. This slide was repaired by flattening the slope within the right-of-way and installing trench drains. The June 2007 slides can be seen at the left and right sides of photograph. Inclinometer I-1 was sheared off at a depth of about 20 feet during this event, leaving I-2 as the only functional casing in this entire cut area.

CONCLUSIONS

This project provided a unique opportunity to apply the techniques of stabilizing and monitoring cut slopes through weak rock on a large scale. Experience in the region indicated that the weak Red Bed strata can quickly weather to soil, and that its presence can contribute to slope instability. The subsurface investigations were focused on identifying these features in order to develop a prudent and economical design approach. The presence of soft, erodible claystone interbedded with layers of harder, more weather-resistant rock gave rise to the implementation of the SSPS, which allowed for the construction of the steep slopes that were necessary to make this project economical. This system is very adaptable to variable construction limits and can be adjusted to suit field conditions under the guidance of a qualified inspector. The SPSS, in conjunction with sub-horizontal drains and the instrumentation program, allowed the construction of a safe and cost efficient project. Given the nature of the geologic materials and the apparent history of slope instability in the region, the relatively minor slope failures that have occurred following construction were not unexpected.

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Karst-Related Experiences for a Highway Project in Southeastern Pennsylvania

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Disclaimer

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ABSTRACT

This paper summarizes experiences from the design, construction and maintenance of US 202, Section 300, a major State Highway in Southeastern Pennsylvania. The bedrock at the project site is Karstic limestone and dolomite with well developed rock pinnacles. Historical sinkhole collapses were observed, documented, analyzed, field verified, and inventoried. Factors that might trigger sinkhole collapses were proposed. Based on the knowledge gained from the historical sinkhole data within the area and other projects near the area, methods to minimize sinkhole hazards were used in the highway geotechnical investigation and design.

INTRODUCTION

The Route 202, Section 300 project site is situated in Chester County, PA. The existing 202 highway is a limited access highway connecting Exton with King of Prussia. Constructed in the late 1960's, the existing roadway section consists of two 12 ft lanes in each direction separated by a 44 ft wide median, and a 10 ft wide shoulder along the right side of the lanes. The existing rigid pavement will be reconstructed and widened by replacing the median with an additional lane and shoulders in each direction separated by a concrete barrier. Collector and distribute roads will also be constructed creating cut slopes at some of the locations.

SITE GEOLOGY

The site is located within a narrow strip of Piedmont Lowlands surrounded by Piedmont Uplands. The Piedmont Lowland Section consists of broad, moderately dissected valleys separated by broad low hills and is comprised of Cambrian (Elbrook (Ce) and Ledger (Cl) Formations) and Ordovician (Conestoga (OCc) formation) carbonate rock formations. These formations, which consist of limestone ($CaCO_3$ rich) and/or dolomite ($MgCO_3$ rich) bedrock, generally have a thin soil cover with a highly variable thickness. According to Geyer and Wilshusen (1982), the soil-rock interfaces in these formations are characterized by highly irregular rock pinnacles. Large boulders may also exist in the overburden soils near the soil-rock interface. The joint pattern of bedrock is reported to be irregular and moderately developed.

The steep dip angles and closely spaced joints of the bedrock were observed in a rock cut slope area near the project site (Figure 1). This photo was taken at a cut slope on State Route 29, 0.8 mile from the 29 and 202 interchange. This interchange is situated on the geologic contact of the Conestoga and the less metamorphosed Elbrook and Ledger Formations. A deep weathering zone exists between these formations at this location. As can be seen from the photo, steeply inclined beds of carbonates and phyllite with different degrees of weathering are present within close proximity. In the completely weathered limestone, "soil seams" can be observed between relatively unweathered bedrock. Similarly steeply inclined bedding planes can also be observed within an abandoned limestone quarry which is approximately 500 yard away from the project limits (Figure 2). These bedding planes together with intersecting fractures provide secondary porosity for the flow of groundwater as well as infiltrating surface water. Therefore, drilling fluid losses, soil seams within bedrock, and a sudden drop of drill rods during rock coring have been frequently observed during every stage of field investigation since the original design and construction of 202 in the 1960s. These "anomalies" are recorded in the boring logs as evidence of the development of Karst features. Figure 3 is a 3-D bedrock elevation map beneath the 29 over 202 bridge site. It was developed using 26 boring records including 10 borings drilled during the initial design of this bridge in the 1960s. The approximate bottom of footing for the proposed bridge is at Elevation 285 ft. The bedrock elevations within the footprint of this proposed three-span bridge vary significantly from an anticipated rock cut at the Far Abutment (right-hand-side) to over 85 ft of soil above the bedrock underneath Pier 1.

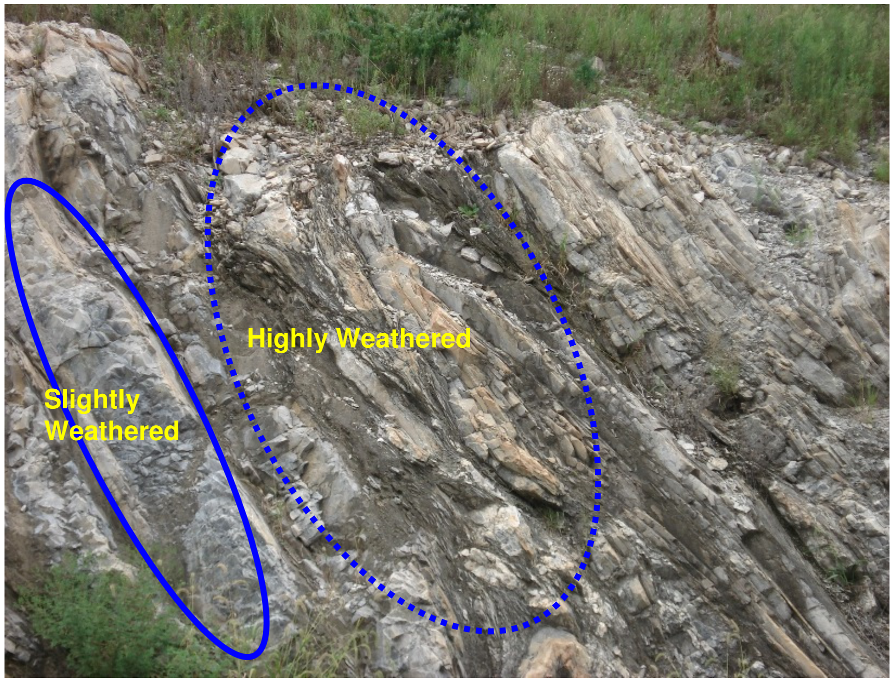

Figure 1. Rock Cut 0.8 mile north of the Project Site
Note the presence of phyllite in the highly weathered zone.
(Elbrook Formation, August 2007)

Figure 2. Steeply Dipping Elbrook Formation Limestone at an Abandoned Quarry near the Project Site (August, 2007)

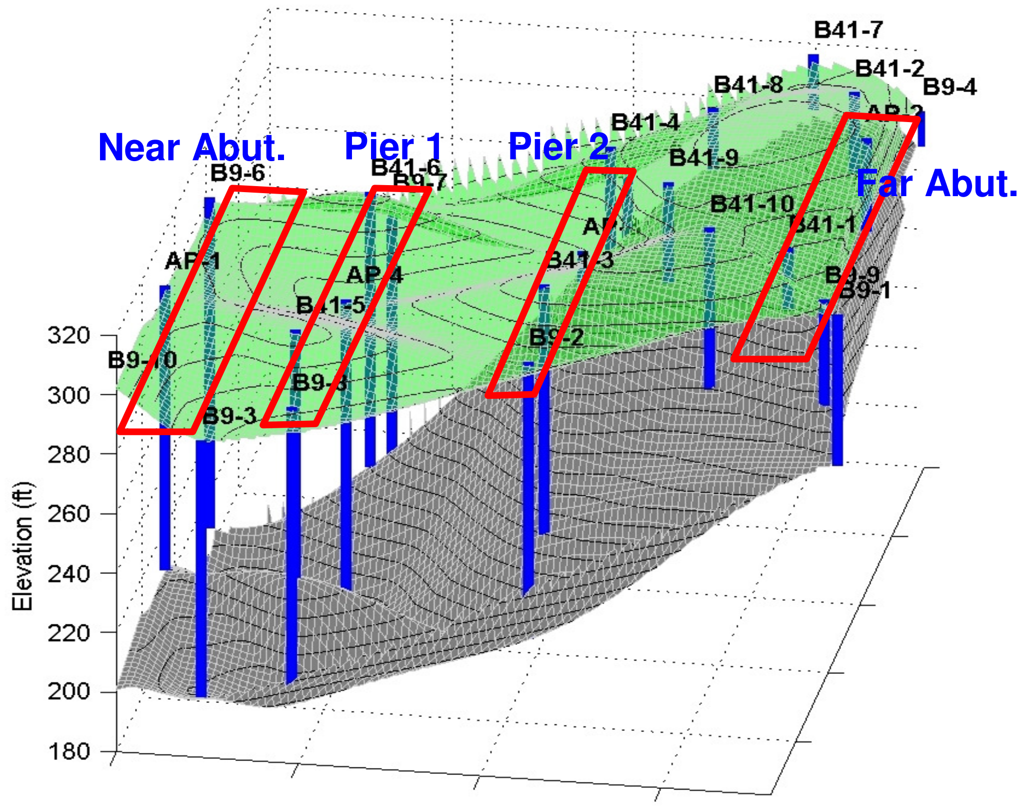

Figure 3. Highly Variable Bedrock Elevations at S.R. 29 over S.R. 202 Bridge Replacement Site

KARST EXPERIENCE NEAR THE PROJECT AREA

Based upon a review of available literature and historical construction and maintenance records, sinkhole events are the primary concern for 202 construction and maintenance. Using aerial photographic interpretation and field observations, Kochanov (1993) prepared an open-file report of Karst related features of the project site and the adjacent area. Numerous sinkhole features, mostly ground depressions, have been identified on a topographic map. A majority of these sinkhole features were concentrated on the north side of the existing highway site (Figure 4). Kochanov also developed a sinkhole database for the entire State of Pennsylvania. Of 45 sinkhole events recorded in the Chester County area, 45% were in the Elbrook formation, 40% were in the Ledger formation, and the rest 15% were in the Conestoga formation. The project limits are located in the contact areas of three major Geologic Formations: Elbrook & Conestoga and Ledger & Elbrook (Geyer, 1982).

Trojan (1974) reported a series of sinkhole events during roadway improvement of S.R. 0202 between the City of Philadelphia and King of Prussia, which is about four (4) miles northeast of the project area. The largest collapse had a diameter of 35 ft and was 30 ft deep. The

Ledger (Cl) Formation bedrock beneath the sinkhole was at least 50 ft deep from the original surface. These collapses resulted in a relocation of the highway alignment.

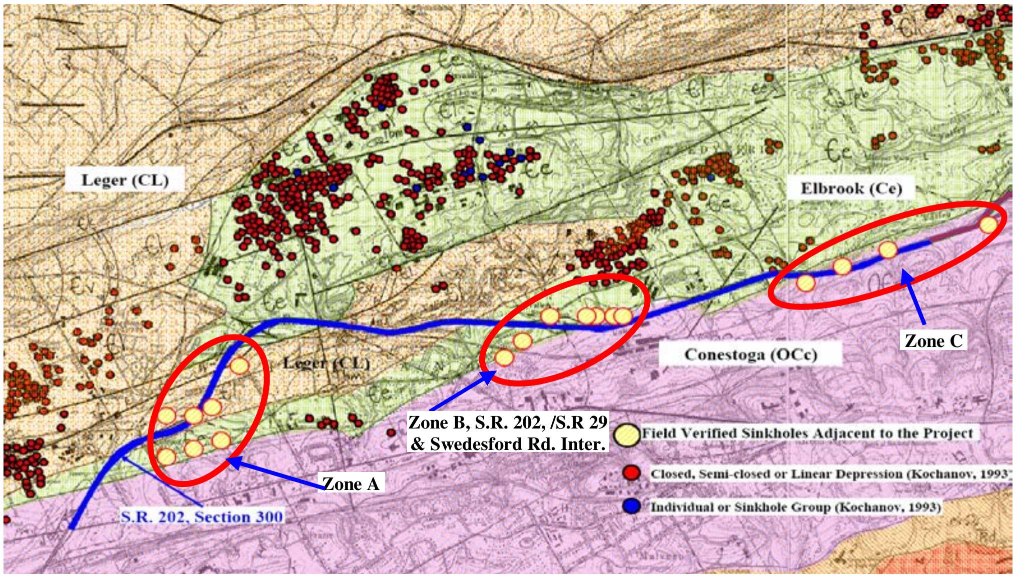

Figure 4. Historical Sinkhole/Depression Events Near the Site (Modified from Kochanov (1993))

At the east side of the project limits, a single span Conrail over Henderson Road Bridge was under construction in 1990. The bridge abutment was originally designed to be supported by HP piles (14X73) driven to Karst bedrock. During pile driving, many piles became misaligned and the majority of piles were severely damaged due to the presence of rock pinnacles beneath the bridge abutment (Figure 5). PENNDOT and the design team decided to extract all piles at this abutment. The abutment was successfully constructed by using rock socketed micro piles.

During the construction of the adjacent S.R. 202, Section 400, which is at the north of the project limits, a significant number of sinkhole events were observed. The relatively smaller sinkhole collapses were concentrated in drainage swales alongside of 202. Figure 6 illustrates a sinkhole collapse at a newly constructed storm-water management pond. The pond was lined with a geomembrane. Usually, these small size collapses indicate relatively shallow bedrock depths. At a new ramp (Ramp L) at the S.R. 422, 202, and I-76 interchange, where a significant soil cut (between 15 ft to 20 ft) was anticipated, larger sinkholes were encountered during construction. Figure 7 presents a large sinkhole collapse at a bridge pier during construction. These large collapses usually correlate with deep bedrock and often occur during construction and after unusual heavy rains following a prolonged draught. Peterson (2003) reported that the bedrock at Ramp L is greater than 100 ft within 202, Section 300. A sinkhole approximately 25ft x 20ft was identified (April, 2001) at a basin located within the Penn State University campus. The basin is located adjacent to 0202 southbound between 29 and Ramp B of 202. The geomembrane-lined basin was excavated from the original ground. The sinkhole developed near the stormwater pipe discharging water into the basin (Figure 9). Stiff sandy silt soil was found at the edges of the collapsed area. This collapse was most likely caused by leakage through the pond.

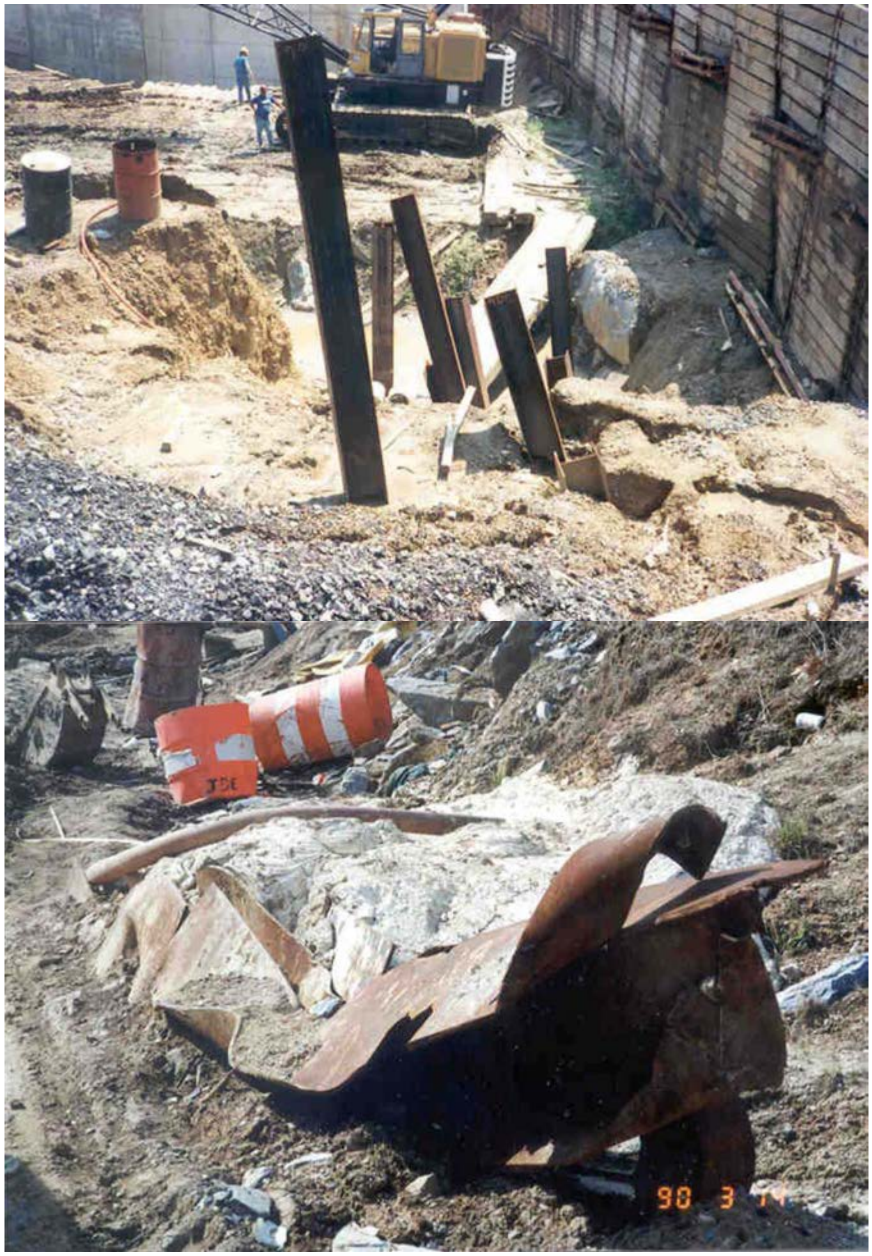

Figure 5. Misaligned and Severely Damaged Piles (HP14×73) in Rock Pinnacles (At One Abutment of Conrail Bridge over Henderson Road, King of Prussia, PA)

Figure 6. Small Scale Sinkhole Collapse at a Storm-Water Management Pond after Hurricane Floyd (S.R. 202, Section 400) August, 1999)

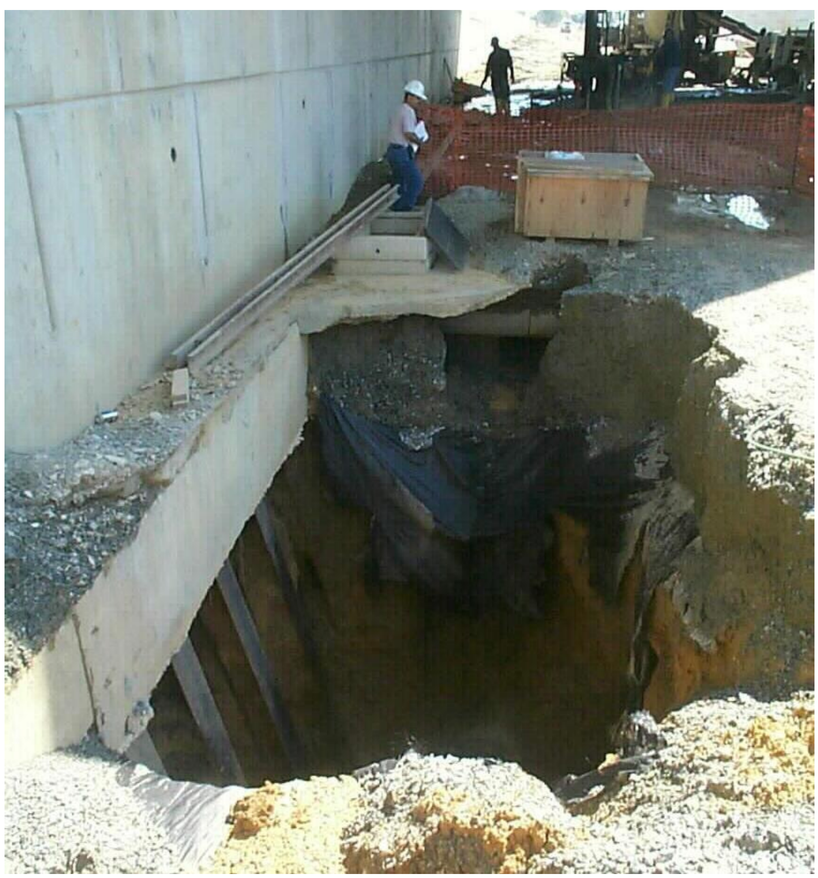

Figure 7. Sinkhole Collapse at a Bridge Abutment during Construction (S. R. 202, Section 400, June 2001)

A field reconnaissance by technical staff of the project team and PENNDOT was held on November 15, 2001. Sinkholes and other Karst features identified and confirmed within the 202, Section 300 project limits were inventoried. Figure 4 also presents the approximate locations of identified Karst features located adjacent to the proposed structures. The field verified sinkholes near the Section 300 project limits are concentrated in three areas: Zones A, B and C in Figure 4.

KARST RELATED HAZARDS IN HIGHWAY SYSTEMS

Field sinkhole investigations in the US (Newton, 1984; Williams and Vineyard, 1976) indicate that most sinkhole problems are caused by the failure of soil voids developed in the residual soil at the soil-Karst rock interface. According to a field sinkhole survey of the Eastern United States (Newton, 1984; Sowers, 1996), the majority of sinkholes and depressions observed were rainfall and/or construction triggered. Rainfall-triggered sinkhole collapses were usually at locations near man-made or natural drainage channels.

A soft soil zone, in which the soil has a higher moisture content and lower shear strength than the surrounding soils, was usually observed near the bedrock crevice under these channels. During a field investigation, the SPT blow counts can be used to identify these soft zones. If the N value has a sudden reduction of blow counts to less than 2 blows per foot and even “weight of hammer (or rods)” and the moisture content of the soil sample from the split spoon sampler increases significantly, it may indicate the presence of a soft soil zone and that bedrock is very close at that depth.

The infiltration of water increases the unit weight of the soil and induces seepage forces within the soft soil zone. In addition, the rise in saturation reduces existing suction in the unsaturated surrounding soils, which decreases the effective strength in the soil. If the strength of soft soil at the rock crevice could not support the weight of soil above, a “local failure” zone is developed and the seepage water in the crevice washes these softened soils away. Finally, a small soil void is created above the rock crevice and gradually enlarges when the hydrological conditions are favorable. The stability of this soil-void system is maintained by soils above the void through a mechanism called the “arch effect”. When the soil above the void does not have enough thickness due to construction or soil erosion, or infiltrated water reduces the strength of the above soil, the “arch effect” in the soil-void system could not be maintained; therefore, a collapse will occur and will result in an open sinkhole or depression at the ground surface. A field investigation in East Tennessee (Newton and Turner, 1986) indicated that rainfall triggered 85% of the approximately 300 sinkhole events surveyed. 70% of the sinkholes identified within the 202, Section 300 project limits were associated with surface drainage paths while some others were associated with the breakage of water mains or sewer lines. Figure 8 indicates a good example of a sinkhole collapse caused by the water leakage from a utility line at the station 265 in zone A area (Figure 4).

Figure 8. Sinkhole Induced by the Water Leakage from the Utility Line (Station 265+00, Zone A in Figure 4, April, 2001)

Figure 9. Sinkhole Collapse at a Newly Constructed Storm Water Management Pond (S.R. 202, Section 300, Station 436+50, 160 ft Left April, 2001)

Karst related engineering problems in highway construction are:

Rock pinnacles: The highly irregular and sharp bedrock interface within a rock pinnacle system may result in differential settlement of shallow foundations supported by soils above the rock. The rock pinnacles may also create pile-driving problems such as misalignment of piles and severe damage of driven piles (Figure 5), thereby necessitating predrilling. Figure 10 presents an extremely well developed Karst rock pinnacle system. In this case, the "top of bedrock" interpreted from the limited amount of borings will be highly dependent upon the location of the borings. If the number of borings is not sufficient, the presence of rock pinnacles may be missed.

Figure 10. Example of a Well Developed Karst Rock Pinnacle System

Soil domes and solution channels developed at the soil-bedrock interface: Field investigations indicated that most sinkhole problems are caused by voids developed in the overburden soil. Usually a soil void begins at a slot between the rock blocks or pinnacles. In a uniform soil layer, a typical soil void or dome resembles an arch. The sides at the bottom generally coincide with pinnacles or irregularities in the rock, and the dome walls are usually vertical (Newton, 1984). A number of highway repairs were reported in the south eastern PA due to the sudden collapse of soil covers (Trojian, 1974, Peterson et al, 2003). Figures 8 and 10 present typical examples of construction-related (removal of soil cover) sinkhole events. Figure 11 is a sinkhole created during construction of 202, Section 400 at the S.R. 422 and I-76 interchange. It is noted that, at level ground and in relatively homogeneous soil, the shape of the sinkhole is circular.

Figure 11. Sinkhole Collapse at S.R. 422/I-76 Interchange (S.R. 202, Section 400, June, 2001)

Rock cavities: Carbonate rocks are soluble in acidic water. Rock cavities may occur before the formation of soil domes and soil channels. Under favorable hydrologic conditions, rock cavities caused by acidic erosion will increase in size and may result in a sudden collapse. Field investigations (Newton, 1984) indicate that the development of rock cavities is very slow compared to the service life of highway engineering facilities. The probability of sudden failure of rock cavities is very low and a collapse is rarely observed.

Both field observations and mechanical analyses suggest that the top three factors inducing collapse and other significant ground subsidence in Karst terrain are:

- A sudden change of surface and subsurface water flow patterns. The probability of a collapse/depression at a location where a sudden influx of a large amount of water will increase. Figure 12 is a photo of sinkhole created along 202, Section 400 roadway during a heavy rainfall event (Hurricane Floyd).

- Reduction of soil cover above the Karst bedrock interface. The reduction of soil cover due to construction or soil erosion will reduce both the soil thickness required to develop an “arch effect” for a certain soil void size and the seepage length to the soil-rock interface. The probability of a collapse/depression will increase due to the reduction of overburden soil thickness. Examples can be seen from Figures 6, 7, 9 and 11.

- A large amount of surface water infiltration to the overburden soil and soil-rock interface. The infiltration of water within a short time due to a utility breakage, such as water or sewer lines, will likely trigger a sinkhole/depression event. Figure 13 is a photograph showing a series of sinkholes being repaired along Swedesford Road (September, 2002), which is only half mile away from the 202, Section 300 project limits. The broken water mains underneath the road triggered the sinkhole collapses.

Figure 12. Road Side Sinkholes Created by Heavy Rain at S.R. 202, Section 400 (During Hurricane Floyd, September 1999)

The application of geophysical methods, such as, Electrical Resistivity, Ground Penetration Radar, Microgravity and Frequency Domain Electromagnetic, is becoming more popular in highway infrastructure design and construction, especially, in Karst terrain. These methods provide information between and below standard geotechnical test borings and other in-situ test borings; they often allow collection of data over large areas in much shorter times than most destructive methods; and they generally cost less per data point than most invasive methods, such as, standard penetration test methods.

Although geophysical methods provide the above advantages, it is important to note that the information obtained in geophysical surveys is often subject to more than one reasonable interpretation. Also, depending on specific site-conditions such as geology, target dimensions, and the engineering problem to be investigated, a combination of methods or techniques may be utilized for a given investigation. In other words, there is no one, unique interpretation to a set of geophysical data.

Figure 13 Repair of Sinkholes underneath Swedesford Road (S.R. 202, Section 300, September 2002)

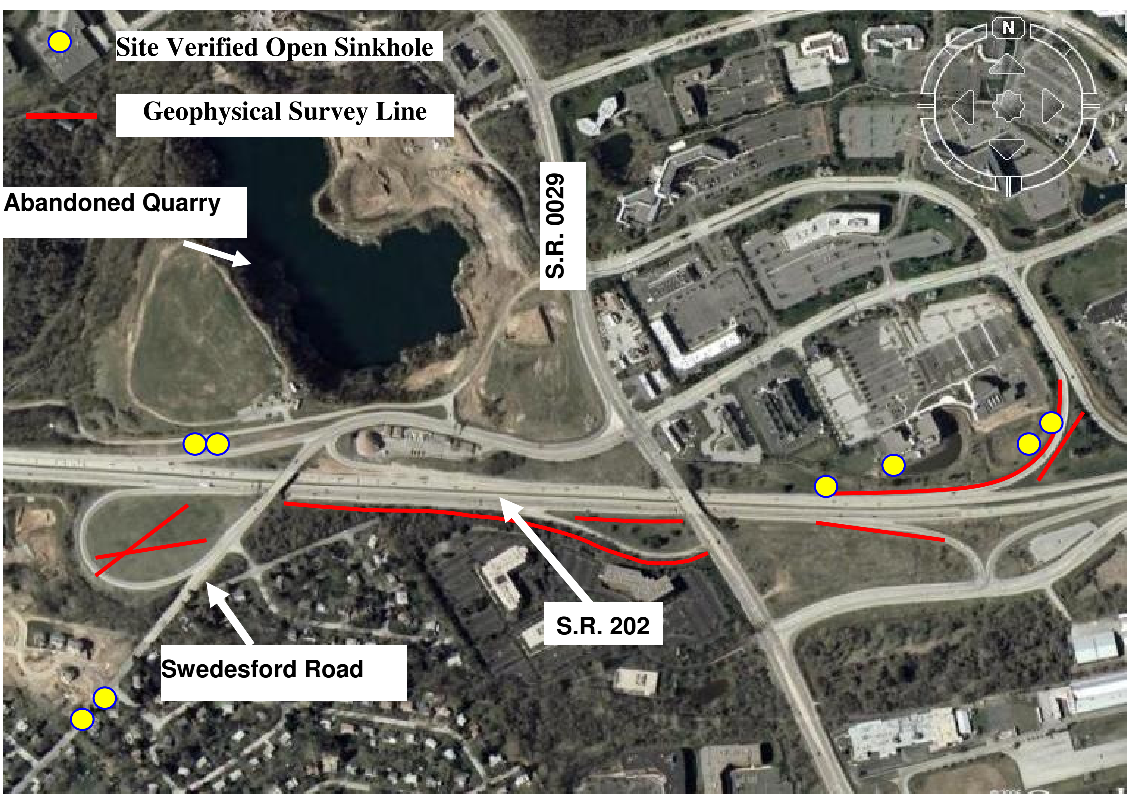

Figure 14. Aerial Photo at S.R. 0029 and S.R. 202 Interchange

Due to the interpretative nature of geophysical survey results, geophysical methods should be used in combination with test boring and other in-situ testing methods to accurately characterize Karst features.

DESIGN CONSIDERATIONS

For the 202, Section 300 project, the largest amount of earthwork is concentrated in the area between the Swedesford Road over 202 Bridge and the 29 over 202 Bridge. In addition to adding traffic lanes in the existing median area, a collector and distribution road (C-D Road) will be constructed. As a result of this, a retaining wall will also be required due to the approximately 10-15 ft of cut into the existing slope at the south side of 202 (northbound). An aerial photo of this general area is shown in Figure 14. A significant cut (~20 ft) is also proposed for the storm water management pond at Swedesford Road to the 202 northbound ramp. The two existing bridges will be reconstructed to accommodate the additional C-D Road. For the 29 over 202 Bridge, the existing two-span bridge will be replaced with a three-span bridge. The proposed bridge plan is shown in Figure 15. To provide the required space for the proposed C-D Road, a soil cut is anticipated between Pier 1 and the Near Abutment.

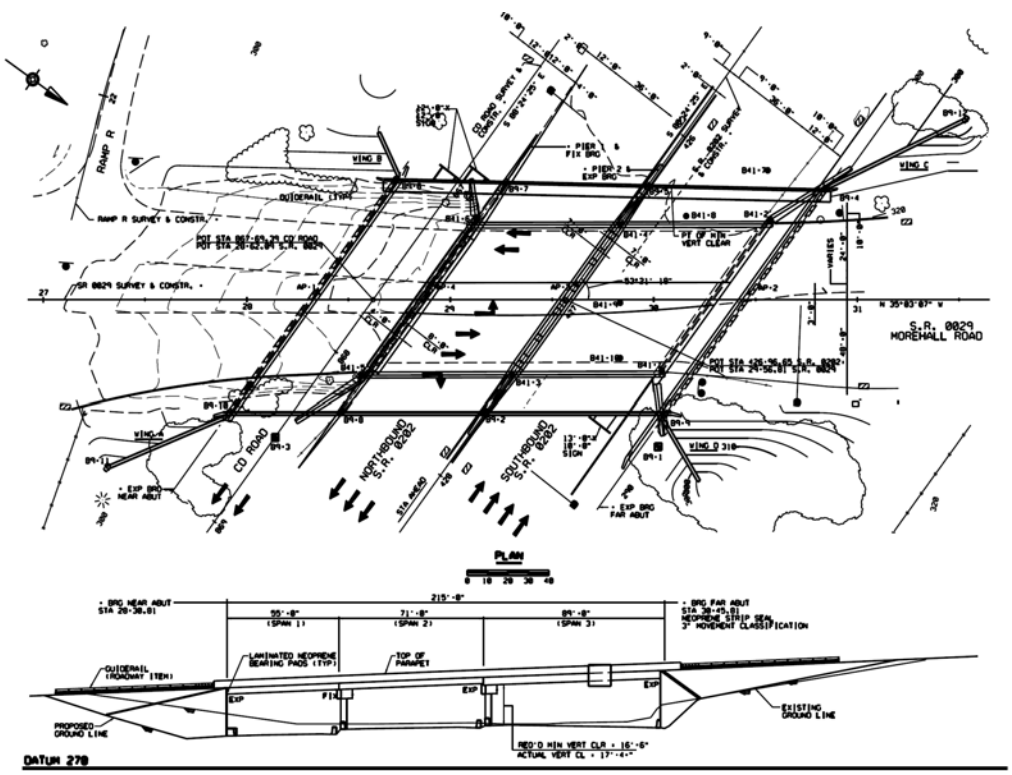

Figure 15 Proposed S.R. 0029 over S.R. 202 Bridge Plan

As shown in Figure 4, several open sinkholes have been field verified in this area (Zone B) and major sinkhole repairs were also observed (Figures 9 and 13). From the rock outcrops

(Figure 1), rock cuts at the abandoned quarry (Figure 2), and boring information (Figure 3), it is clear that the rock pinnacles are well developed due to the highly folded and steeply dipping rock bedding planes.

To improve the characterization of the subsurface information in these cut areas, an Electrical Resistivity survey was conducted at this area before the final geotechnical investigations for the retaining walls and bridges; therefore, only a small number of the test borings from the preliminary design stage could be used for "calibration" purposes. The survey line distributions are shown in Figure 14. The survey results were compared with information from a large number of test borings from the final geotechnical investigations for the bridges, retaining walls, and noise walls in this area. It was concluded that the geophysical survey indicated a consistent result at locations which could be verified by test boring information. Therefore, for areas without borings, the geophysical survey provided a more reliable interpretation as a result of this "calibration process". No significant soil cavities were found in test borings and geophysical survey results. No low blow count materials (SPT N<2) were encountered in the test borings.

For design-build retaining walls in this area, wall types with less cut requirements, such as, soldier pile and lagging walls and soil nail walls were recommended as the permissible wall types. Drilled shafts instead of spread footings were the recommended foundation types for noise walls.

The geophysical survey result in the proposed 20 ft cut at the storm water management pond indicates that the soil resistivity is high, which means that the soil material is dense and perhaps, above the groundwater. Therefore the probability of sinkhole collapse at the proposed storm water management pond is low.

Although a significant cut was anticipated for the existing 202 northbound lanes, shallow foundations were recommended for the bridge foundations. The basis for the shallow foundation recommendations were (29 over 202 Bridge, as an example):

- Rock pinnacles are anticipated and the bedrock elevations vary significantly and based on past experience difficulty can be expected installing deep foundations.

- The adjacent abandoned quarry provided a good "groundwater observation well". It was concluded that groundwater levels were low and groundwater fluctuations at the soil/rock interface, which would increase the probability of void development or propagation, are not anticipated.

- The bedrock underneath the area between the Near Abutment and Pier 1 is very deep (greater than 50 ft, Figure 3). The stability of a soil dome (if any) at the soil/rock interface can be maintained through "arch effect" due to the thick layer of soil between the interface and the foundation.

- Soft soil zones with SPT N values less than 2 were not encountered in any test borings beneath the bridge foundations. The geophysical survey (Figure 16) indicated that the soil at the proposed cut areas is uniform (at approximate Ramp R Station 21+00).

- The existing 202 in this area was constructed within a 15 to 20 ft cut excavation. Historically, sinkhole events observed in Zone B, including sinkholes in Figures 9 and 12, were induced by surface water infiltration. If surface water is appropriately diverted away from the bridge substructures during bridge construction and service, the probability of sinkhole collapses due to water infiltration can be minimized.

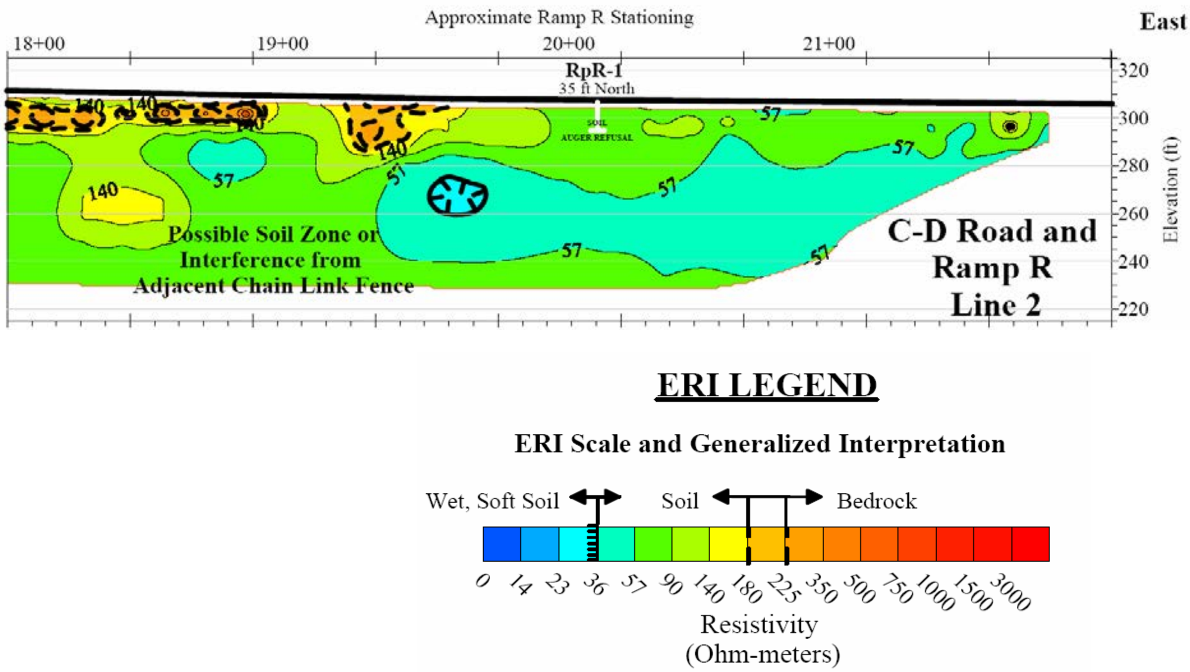

Figure 16 Electrical Resistivity Survey Result at Ramp R (S.R. 202, Section 300)

CONCLUSIONS

Based on the review of the construction and maintenance records of the existing construction, field sinkhole verification, and experience gained from adjacent highway projects areas, the sinkhole events observed in the 202, Section 300 project limits could be classified into three categories: removal of soil cover; sudden change of flow path; and leakage from water mains. The majority of the observed sinkhole collapses were triggered by heavy rains.

During the design of the highway structures in Section 300, the following measures were used to minimize the potential for sinkhole collapses at roadway and structure locations: 1) evaluation of the probability of sinkhole collapse in combination with historical sinkhole locations and geotechnical investigation results, 2) from the experiences gained from adjacent projects, minimization of cut excavations is critical for minimizing the sinkhole potential, especially, in areas with relatively thin soil covers, 3) diversion of surface water away from structures and 4) use of geophysical survey methods as part of subsurface soil characterization tools.

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Lessons Learned from Ground Modification Program Using Grout in Karst for I-70 in Frederick, Maryland.

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ABSTRACT

The I-70/ MD 85 Interchange, MD 355 and MD 475 Interchange Reconstruction Project is an improvement project for the existing alignment of I-70 as it passes on the south side of the city of Frederick, Maryland.

This project occupies an active karst area underlain by Cambrian and Ordovician limestones. Historically, sinkholes have been very problematic for transportation facilities. Consequently, major aspects of this project are ground modification for the sinkhole prone areas and unique storm water management facilities. The storm water management system includes three lined ponds, a 600-foot long earth dam, geotechnical instrumentation, and two pump stations.

The past success of Maryland's subsurface grouting program for addressing sinkholes in existing roadways led to the decision to incorporate subsurface grouting as a preventive measure in capital highway projects in sinkhole prone areas. In 2006, the grouting program for the I-70 improvement project included 681 grout holes and 42,227 cubic yards of grout. Management of this program required daily interpretation of a plethora of subsurface data. Drill and grout data was entered into a computer with a 3D software program to develop grout hole location maps, depth to top of rock, and grout-take contour maps. These maps were used to guide grouting operations, and to determine the need for secondary grout holes.

This paper will also cover the geologic setting, the field exploration, and sinkhole-related designs. The subsurface karst conditions encountered during construction were more developed than predicted, resulting in large grout volumes and impacts to the project schedule. Modifications were made to the drilling and grouting strategy. Lessons learned will be discussed. Evaluation is still in progress; however, the grouting program appears to have achieved stabilization at the roadway and structure locations

INTRODUCTION

Project Description

The I-70/MD 85 Interchange, MD 355 and MD 475 Reconstruction Project is located between the MD 355 and South Street intersections along I-70 in Frederick, Maryland (Figure 1). This project involves the construction of two overpasses and approximately a half mile of roadway (extensions of Monocacy Boulevard and MD- 85 to proposed East Street). There is also a large storm water management system that includes a 600-foot long earth dam, three lined ponds, geotechnical instrumentation, and two pump stations. An aerial photo of project site identifying proposed construction is shown on Figure 2. This project occupies an active karst area underlain by Cambrian and Ordovician limestones. Historically, sinkholes have been very problematic for transportation facilities. Consequently, major aspects of this project are ground modification for the sinkhole prone areas by various methods and to develop unique storm water management facilities.

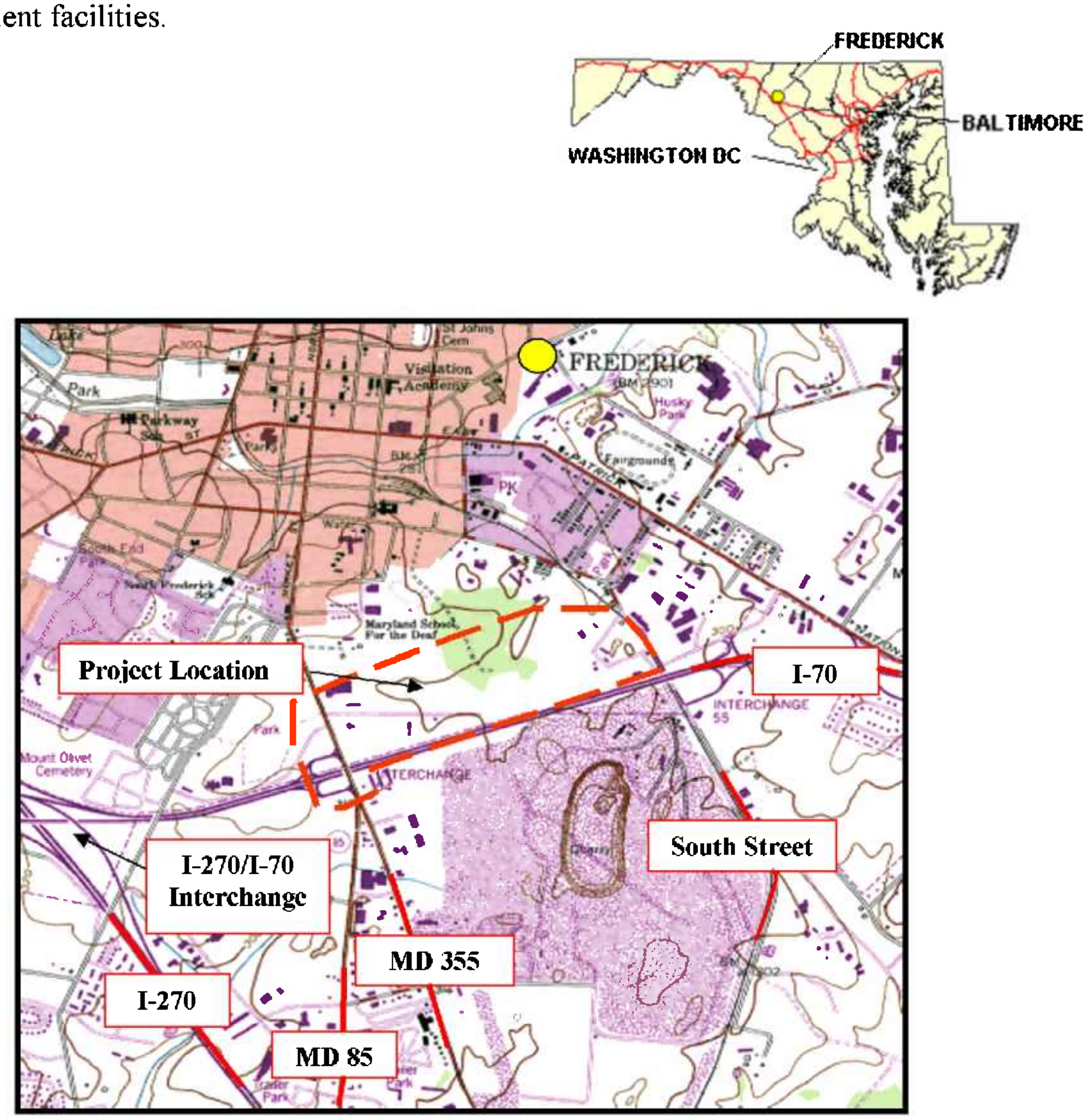


Figure 1 - Project Location Map

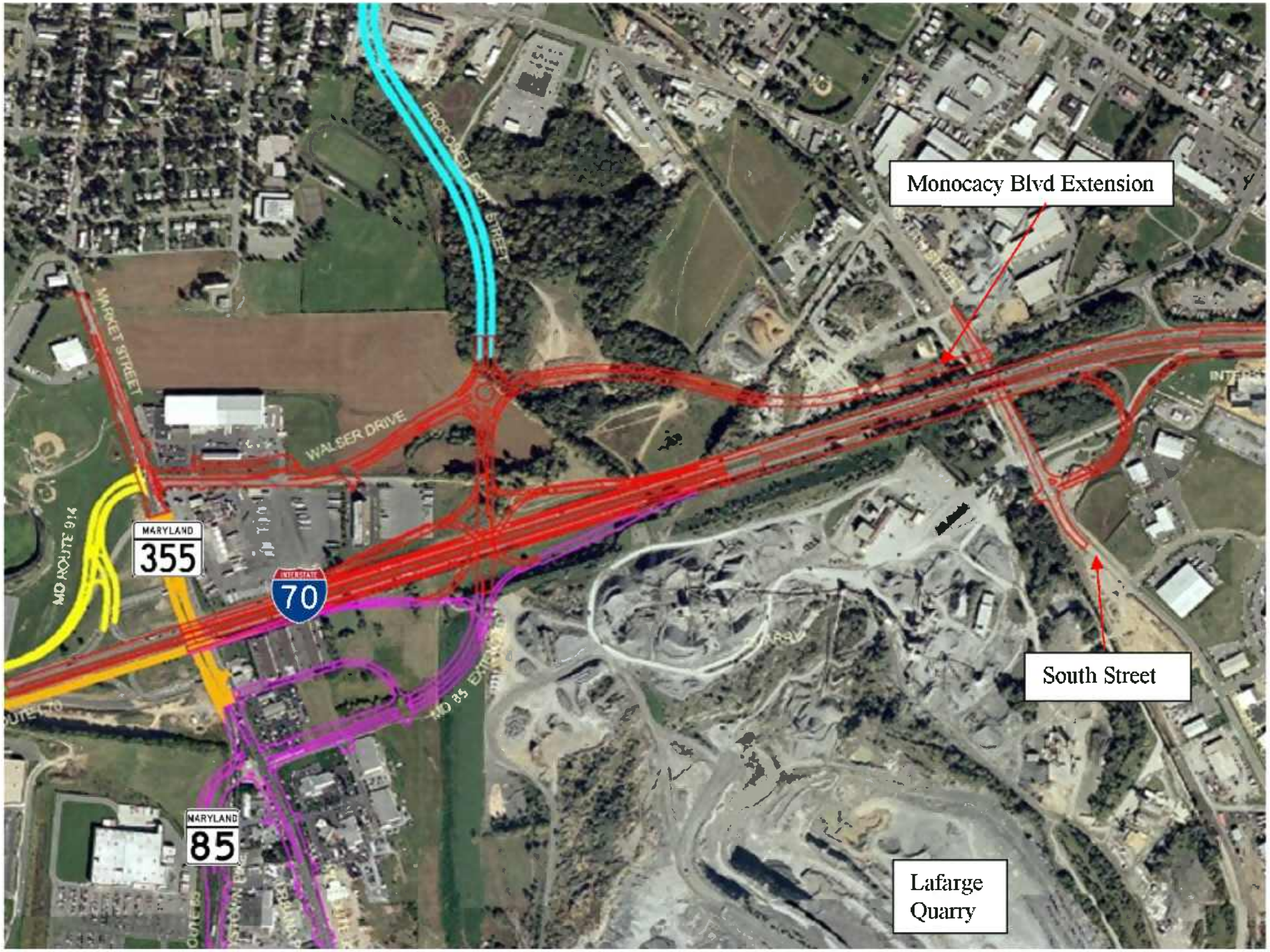


Figure 2 - Aerial Photo of Project Site

Purpose

The past success of Maryland's subsurface grouting program for addressing sinkholes in existing roadways led to the decision to incorporate subsurface grouting as a preventive measure in capital highway projects in sinkhole prone areas. In 2006, the grouting program for the I-70 improvement project included 681 grout holes and 42,227 cubic yards of grout. Management of this program required daily interpretation of a plethora of subsurface data. Drill and grout data was entered into a computer with a 3D software program to develop grout hole location maps, depth to top of rock, and grout-take contour maps. These maps were used to guide grouting operations, and to determine the need for secondary grout holes.

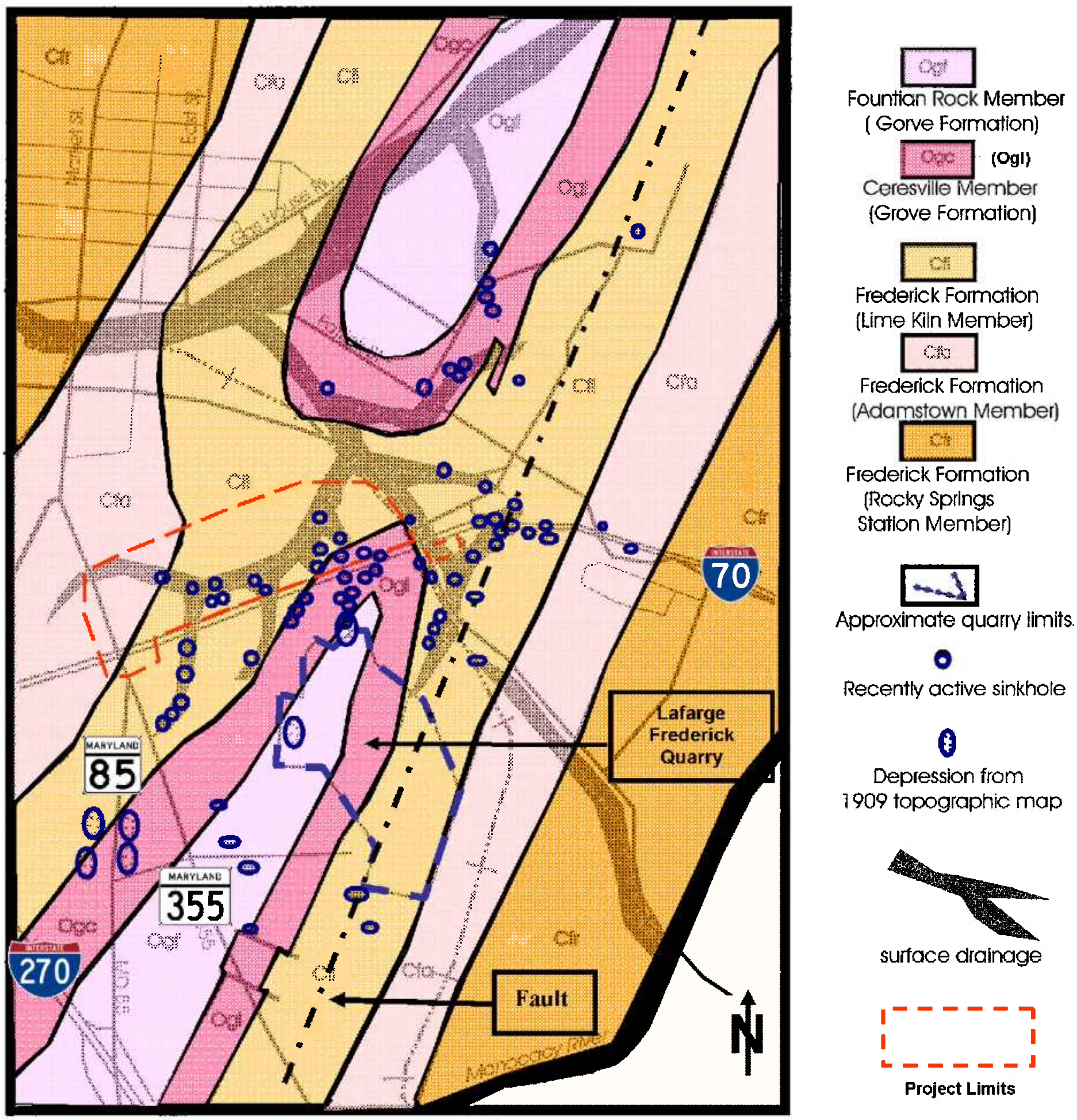
A major aspect of this construction project is subsurface improvements to the sinkhole prone areas within the project limits. In order to address the on-going subsurface problems, a low mobility displacement grouting (LMDG) program was implemented to treat the sinkholes and attempt to prevent future sinkholes from developing at roadway and structure locations. The goal of the grouting was to fill the subsurface voids and associated solution channels in the epikarst. The grout was injected under pressure to create a grout cap between the soil overburden and rock to prevent the migration of soil and water in the solution channels.

This paper will also cover the geologic setting, the field exploration, and karst-related issues. The subsurface karst conditions encountered during construction were more developed than predicted, resulting in large grout volumes and impacts to the project schedule. Modifications were made to the drilling and grouting strategy. Lessons learned will be discussed. Evaluation is still in progress; however, the grouting program appears to have achieved stabilization at the roadway and structure locations.

SITE GEOLOGY

The Frederick Valley is a lowland located in the Western Piedmont Physiographic Province of Maryland between the Blue Ridge Province and the Eastern Piedmont Province. Catoctin Mountain, in the Blue Ridge Province immediately to the west, is composed of late Precambrian to early Paleozoic metavolcanics and metasediments. The limit of the Frederick Valley on the east is generally placed at the Martic Line, a deep fault which separates the greenschist facies of the Western Piedmont and the virtually unmetamorphosed sediments of the Frederick Valley.

The Frederick Valley, and the project area, is underlain by carbonate units of the Upper Cambrian Frederick Formation and the Upper Cambrian to Lower Ordovician Grove Formation (Figure 3). In the northern and western areas of the valley, but not in the project area, Triassic redbeds of the New Oxford Formation overlie the carbonate units. The Frederick Formation is divided into four members by David Brezinski of the Maryland Geological Survey in the most recent study beginning in 2001 and published in 2004 (Figure 4). The Monocacy member makes up the lowest strata of the Frederick Formation, which consists of interbedded black shale and limestone breccia that is less than 33 feet thick. Above the Monocacy member, the Rocky Springs Station member consists of light to dark gray, interbedded, very thinly bedded, shaly



Map source: Modified from Brezinski, D. K., 2004

Figure 3 – Geologic Map of Project Site

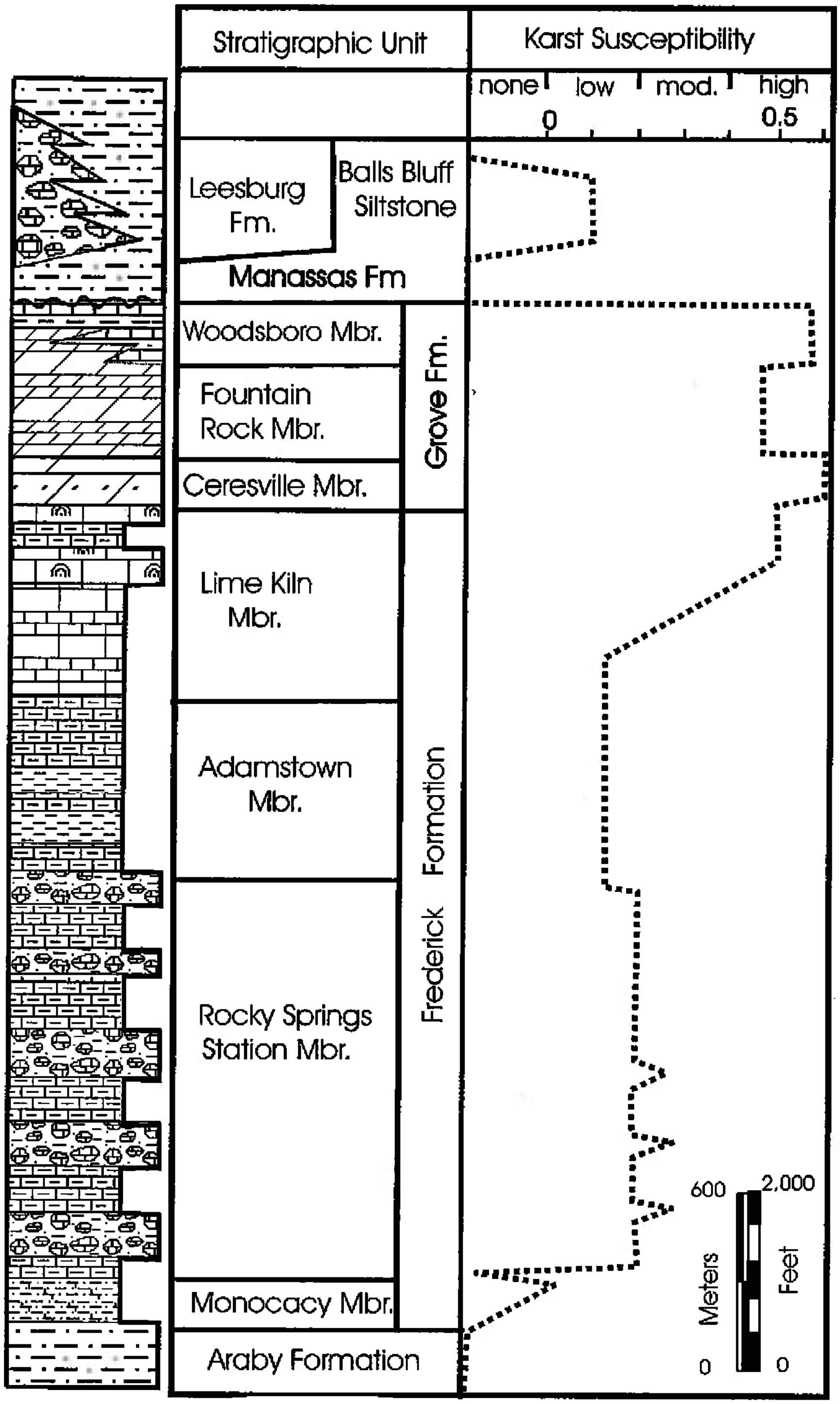


Figure 4 - Stratigraphic Column and Relative Karst Susceptibility of Frederick Valley

limestone; medium bedded sandy limestone; and thickly bedded, polymitic limestone breccia. This member is less than 1,000 feet thick and is encountered both east and west of the project area. The Adamstown member is characterized by dark gray, thinly bedded limestone with shaley partings and thick polymitic breccias that may represent submarine slides at the distal edge of large submarine fans. The assumed thickness in the project area is approximately 1200 feet. The Lime Kiln member is lithologically variable, but is characterized by thin bedded, dark gray limestone that grades upward into thicker bedded, medium gray grainstone and packstone. Most intervals have abundant sand grains. The upper 200 feet is marked by algal thrombolitic limely mudstone and stromatolitic beds. Within the project area, the thickness is estimated at 750 feet.

The overlying Grove Formation is represented by its lowest Ceresville member, due to erosion, and is estimated to be approximately 300 feet thick in the project area. Typical outcrops expose thickly bedded sandy dolomite that is light gray and fine-grained. Sand grains compose 5% or more of the rock and are large and well rounded. Some layers are cross-bedded.

The carbonate rocks form a double plunging synclinorium that strikes north-northeast (average N 25-30E) and is overturned to the west. The project area lies on the western limb of this structure and bedding dips gently to the southeast. There is also a predominant near vertical fault that has a northeast-southwest trend that serves as a hydraulic barrier to the east of the project site.

Factors Contributing to Karst

The sinkhole activity, predominately consisting of cover and collapse, is attributed to solution erosion of the underlying limestone bedrock caused by migration of water through the subsurface. This activity results in zone of large voids, opened fissures, and weathered rock called epikarst, which is typically the uppermost layer of a karstified rock. Large precipitation events, surface water runoff due to local site development, significant groundwater fluctuations caused by a nearby limestone quarry operations, rock type, bed orientation, and joint spacing are factors that contribute to karst formation. This upper Lime Kilm member of the Frederick Formation has the highest karst susceptibility (Figure 4).

PROJECT HISTORY

The city of Frederick is a rapidly growing urban area in central Maryland. It is situated in a limestone valley at the intersection of US Highways 40 and 15. US Highway 40 was originally known as the Baltimore National Pike which connected Cumberland, Maryland to the city of Baltimore and its port. The Baltimore National Pike was the logical eastern extension of the National Road which connected Cumberland to the Ohio River Valley at Pittsburgh, Pennsylvania. The National Road, of 1806, was the first federally funded roadway in the U.S. The Baltimore Pike was funded by the state of Maryland through bonds and toll franchises. All the toll segments were removed by 1915 and the roadway was designated US Highway 40 when the federal numbering system came into being. US Highway 15 is a primary north-south route for commercial traffic. To the west of Frederick, I-70 interchanges with I-270 which provides a direct connection to the metropolitan area of Washington D.C. Nineteen miles west of Frederick, I-70 interchanges with I-81. The presence of the city at this junction of roadways, and the desirability of the area as a bedroom community to the nation's capital has spurred the growth rate.

In the 1950s, traffic volumes on US Highway 40 became large enough to justify a bypass around the city of Frederick. Construction of the bypass, also known as the "Frederick Freeway", began in 1956 and was completed by 1960. This roadway was constructed to interstate standards, south of the city. It was later incorporated into the alignment of I-70.

In the mid 1980s, the average daily traffic count for this roadway was over 70,000 with projections for rapid growth. While the number of lanes was proving inadequate, particular concern was generated by the deteriorating bridges and the limited capacity of the interchanges which were designed for 1950s traffic volumes.

Planning studies were initiated in 1984. By then, requirements for environmental impact studies for highways, including hydraulics, had become much more comprehensive than they were in 1956. The initial studies prepared for State Highway Administration (SHA) by Greiner (URS) identified that all the existing highway drainage was handled by the karst geology. There are no surface streams in this drainage basin. All storm water leaves the area through the sinkhole system. The study further noted that the existing sinkhole system would not handle the additional run off from the improvements in a reasonable amount of time. Planning work slowed during the late 1980s and early 1990s due to economic conditions and other priorities.

Although planning work had ceased, it was obvious that numerous sinkholes were beginning to appear in the area. Maryland had suffered a fatality when an individual drove into a catastrophic failure on another roadway in 1994, so the SHA was extremely sensitive to the potential dangers of roadway sinkholes.

In the mid and late 1990s, work began again in earnest. It was determined by senior management that the design for this roadway would incorporate elements originally developed and recommended by the Tennessee Department of Transportation for the control of storm water. All ponds and ditches would be lined, and all pipe trenches would be backfilled with flowable backfill instead of crushed stone. Various alternates were considered for the removal of the storm water. Because the area is a natural shallow basin, all the alternates that depended on traditional gravity flow required unacceptable levels of excavation, excessive amounts of property acquisition, and they created their own sinkhole issues. Therefore, SHA had to adopt a pumping alternate for the storm water management. Due to the practical limits of the pumps, large storage ponds were needed. The project originated with two ponds totaling 12.5 acres. The addition of a third 4 acre pond with its own pumping station was prompted by the sudden appearance of a 6000 cubic yard sinkhole that threatened to undercut the existing roadway embankment. Construction for the I-70 improvements began in 2003.

Maryland has a successful program for treating sinkholes on existing roads through cap grouting. In 1994, the state suffered the loss of a driver who drove his van into a sinkhole that had rapidly and catastrophically failed. This fatality, and a similar non-fatal accident in 1996, heightened the awareness of the SHA to the danger that sinkholes posed to the safety of the state's drivers. Initially, sinkholes were repaired by excavation and backfill. While this method provided acceptable performance, it required road closure, and the interference to traffic flow was deemed unacceptable, especially on the interstates. SHA has adopted the process of filling sinkholes with flowable backfill from the surface without major excavation, and following up with cap grouting. Approximately thirty sites have been treated since 1996. Of these, only two sites have needed more than one grouting treatment. This process is faster than excavation and backfill, and has the added advantage of being scheduled during times of low traffic flow. The success of this process led to its inclusion as a preventative measure in new road construction projects where the potential for sinkhole development was high.

Cap grouting was included in the several sinkhole remediation measures for the I-70 improvement project. The existing sinkholes that were not under the pavement areas, and where rock could be exposed in the bottom, were treated by conventional excavation and backfill methods. For sinkholes where rock could not be reached with conventional excavators, grouting was planned to provide the support between the bottom of the surface fill and the top of rock.

FIELD EXPLORATION

Reconnaissance and Aerial Photography

SHA's program for anticipating, monitoring, and remediating sinkholes includes the maintenance of a library of aerial photographs of all state roadways in carbonate areas. These 250 feet to the inch stereo photographs have proved valuable in understanding the formation of sinkholes on the state's roadways. The project area was completely covered with stereo aerial photographs. They were used in hydraulic design, erosion and sediment control, interpretation of the geophysical surveys, the location of the borings, the location of the surface dumps and the location of the existing sinkholes. Figure 5 shows the locations of existing sinkholes on regional topographic map that includes the project site.

Geophysics

Geophysical surveys offered the best method for the broad coverage needed for the identification of potential sinkholes. Because this project included new roadways on undeveloped land, the use of geophysics would be unhampered by existing surface improvements. The consultants elected to use the dipole-dipole electrical resistivity method. The cross-sections produced by the study confirmed the potential for sinkhole formation. Figures 6 and 7 show the locations of geophysical survey and an example cross-section developed from the interpretation of the geophysical data, respectively. Correlations between the drill and geophysical data on specific cross-sections were not obvious (Figure 8). However, by plotting the top of rock and the epikarst as contour maps based on the geophysical survey and using the drainage vectors plot function available through the use of Surfer 8™, the potential sinkhole areas became clear. The areas where 3 or 4 drainage vectors point toward one another corresponded to areas of existing sinkhole activity and were potential sinkhole areas (Figure 9). The drainage vector plot indicates the direction of water flow derived from surface or

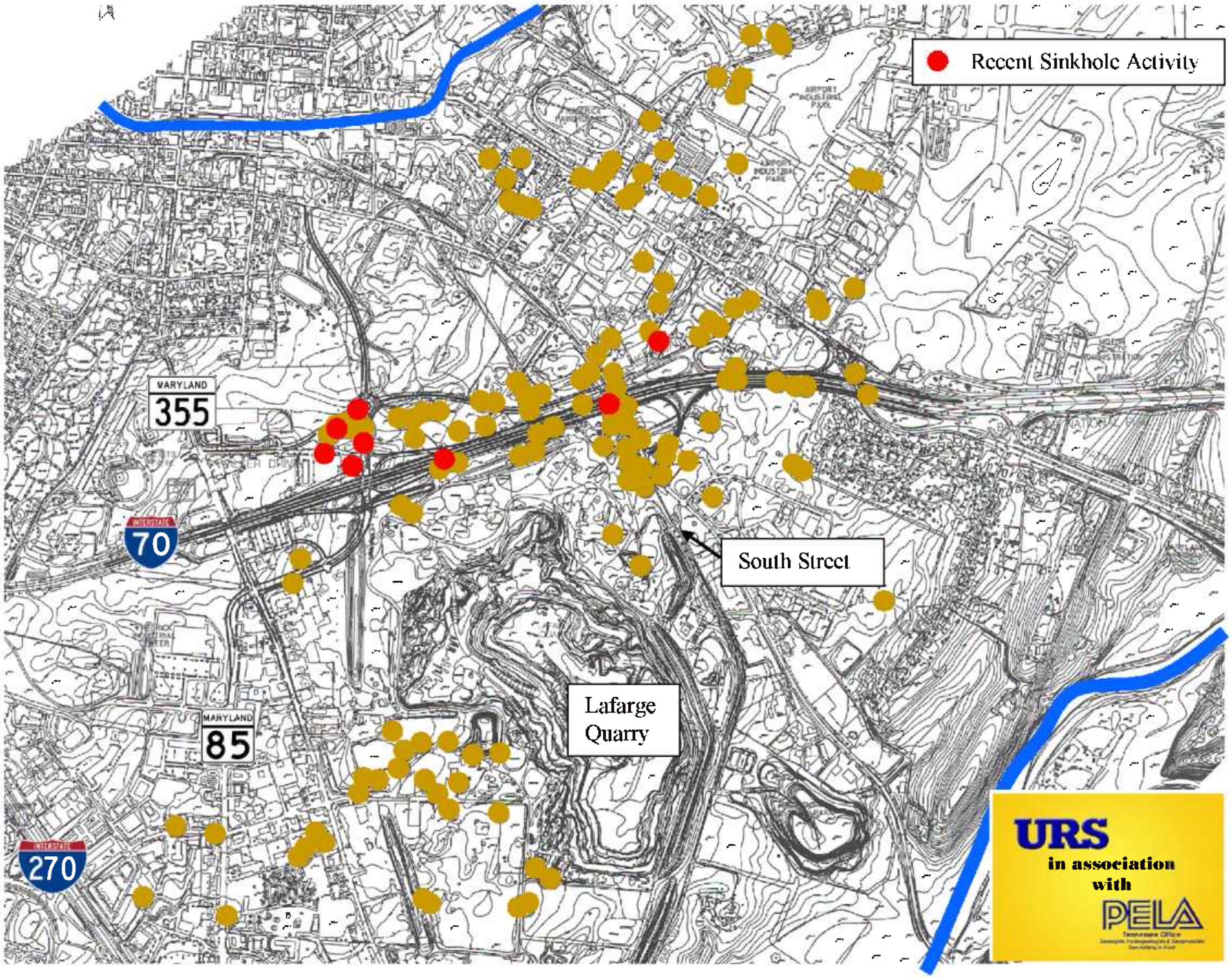


Figure 5 – Sinkhole Inventory on Regional Topographic Map

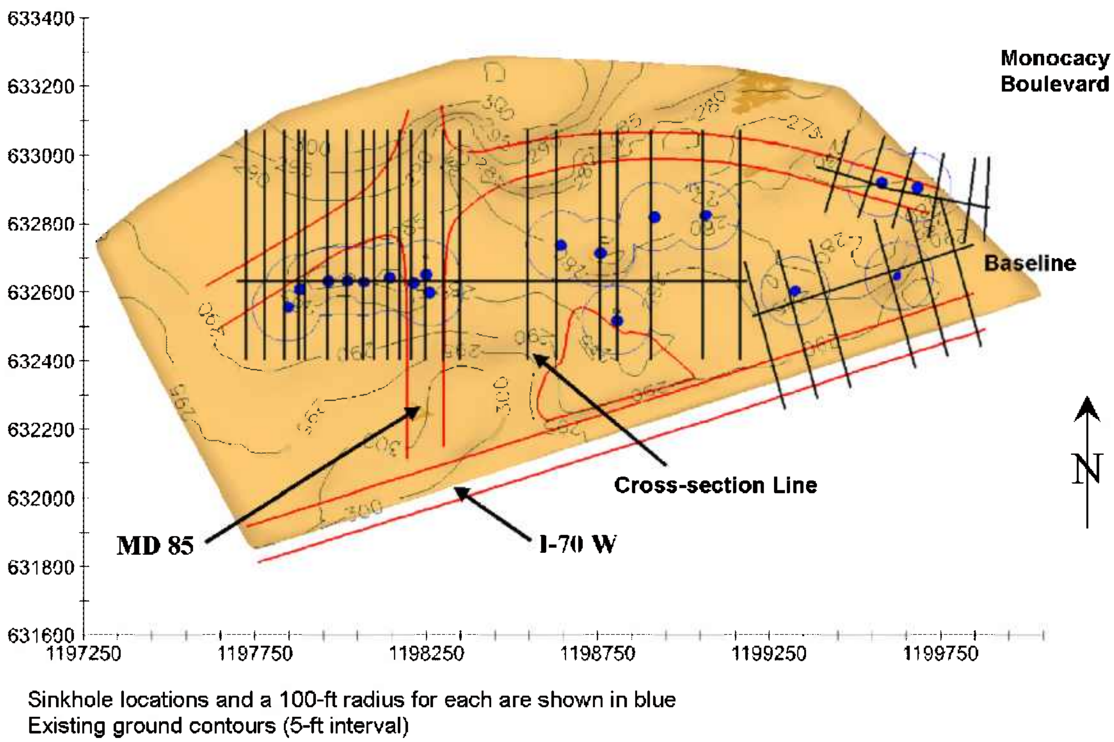


Figure 6 – Plan View of Geophysical Survey Cross-sections

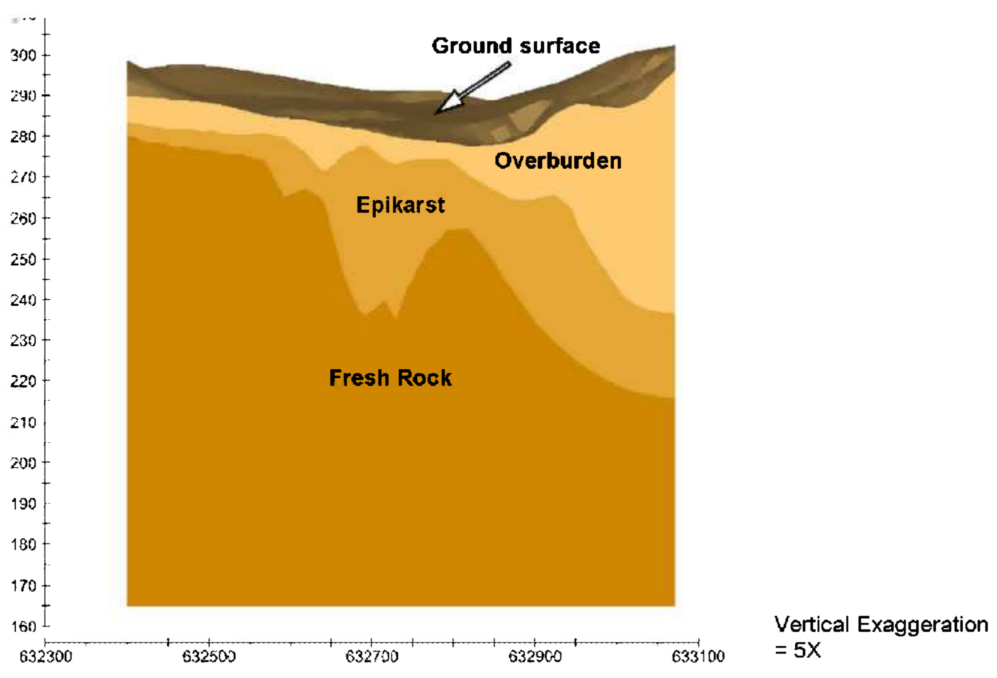


Figure 7 – Example Interpreted Cross-section from Geophysical Survey

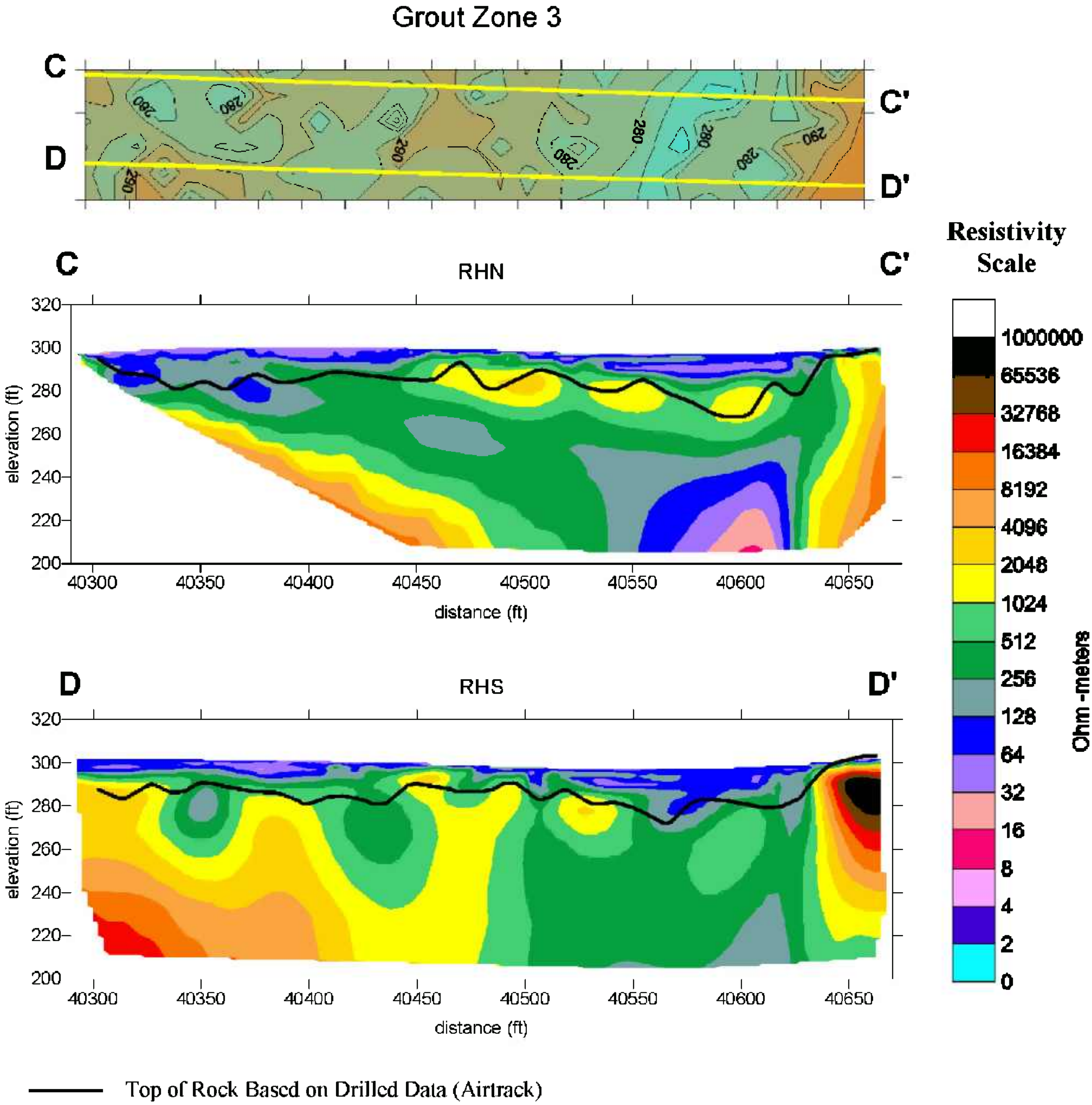
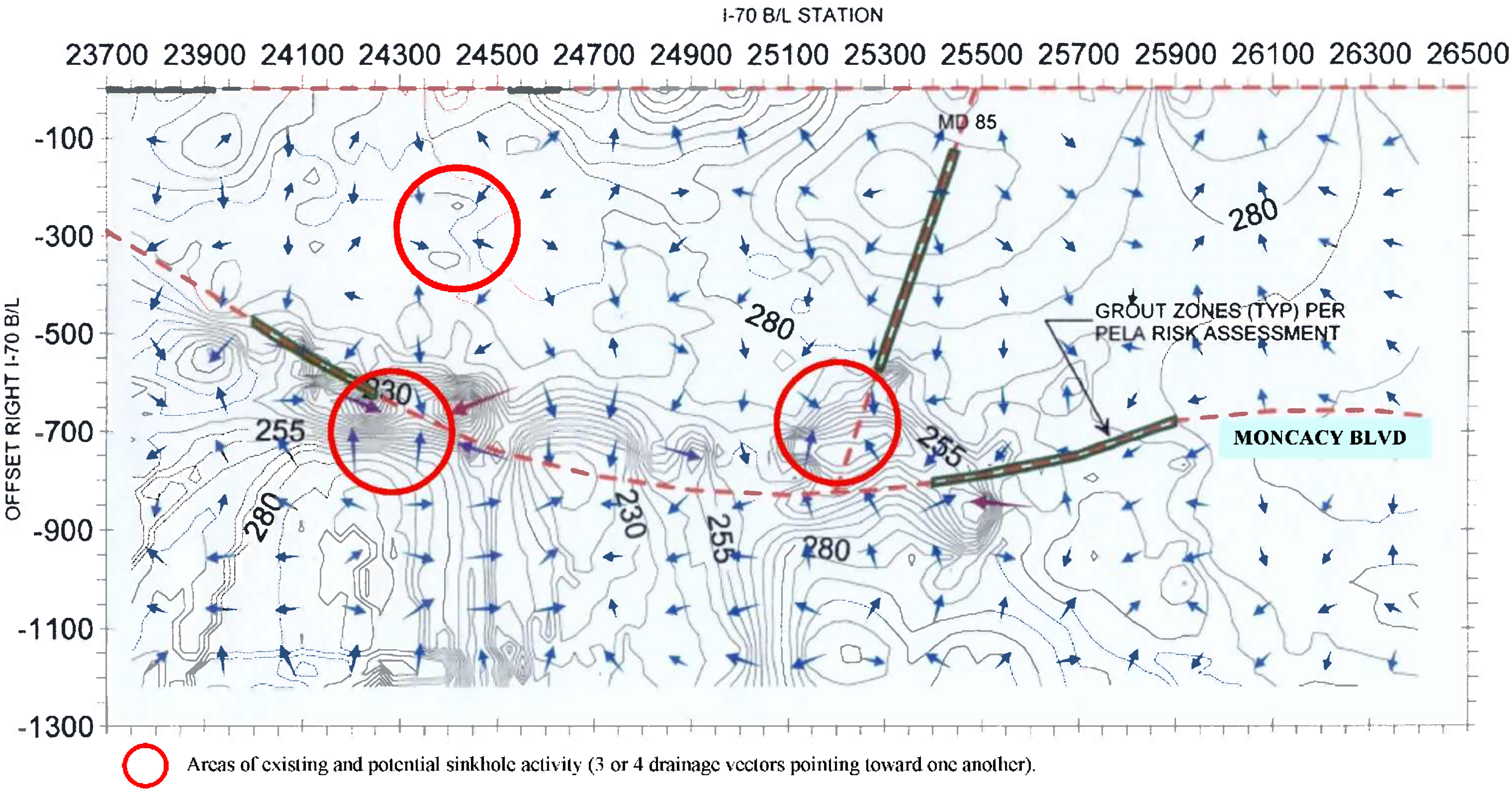


Figure 8 – Example Cross-sections Showing Top of Rock from Drilling Plotted on Geophysical Section



Areas of existing and potential sinkhole activity (3 or 4 drainage vectors pointing toward one another).

Figure 9 – Drainage Vector Plot Along Top of Epikarst

groundwater due to the epikarst or rock contours. The predominate karst features revealed by the geophysics and when represented in the Surfer based drainage vector plot were used to determine grout zone locations. The majority of design decisions concerning sinkhole prevention were based on the geophysics.

Drilling

The project includes two bridge structures, a dam, two pump stations, and numerous incidental structures. Foundation (SPT and core) borings were made for each structure. In the past ten years, foundation borings were also made for the several sinkholes that opened on or near the pavement of I-70 within the project limits. Soil auger borings were made to collect bulk samples for earthwork and general soil classification. Foundation borings were also made to calibrate the geophysical survey data. All boring data was included in the contract documents. All boring locations are presented in Figure 10.

DESIGNS RELATED TO SINKHOLE PREVENTION

Storm Water Management

The earliest planning studies found that storm water from the existing roadways, as well as the surrounding area, all drained into the existing sinkhole system. The studies also found that the drainage system was taxed to its limit during heavy precipitation events, and could not be depended upon to handle the additional loads imposed by the proposed roadway improvements.

Several strategies for handling the storm water by conventional gravity flow were investigated. All were found to require excessive excavation and excessive right of way. It was finally determined that the best approach was for the collection of the storm water in a lined pond from where it would be pumped to a creek about 3,000 feet to the north. The pond needed to be large enough to store the drainage from several 100 year storms to accommodate the limits of the pumps. Thus, pond A-B was over 12 acres and 16 feet deep. The confining embankment for this pond was large enough to be designated as a dam. Water retention was provided by a 60 mill liner over a clay covering on the embankment. The zone under the dam and east-west approach embankments was grouted. The embankment was instrumented with time domain reflectrometry (TDR), and was reinforced with geotextile under the pavement.

Sinkhole Treatment

There were four typical sinkhole treatments based on the depth of the sinkhole and location to roadways or ponds. Treatment type was directed by the engineer in the field to insure the best construction practices were followed.

Type A: The highest level of protection was afforded to sinkholes that are under the roadway or roadway embankments. After cleaning/excavating the sinkhole, the throat is to be choked by low slump concrete, the opening is backfilled with class 2 (500 lbs – nominal) riprap, capped with 12 inches of No.2 (AASHTO No. 2) stone and covered by a

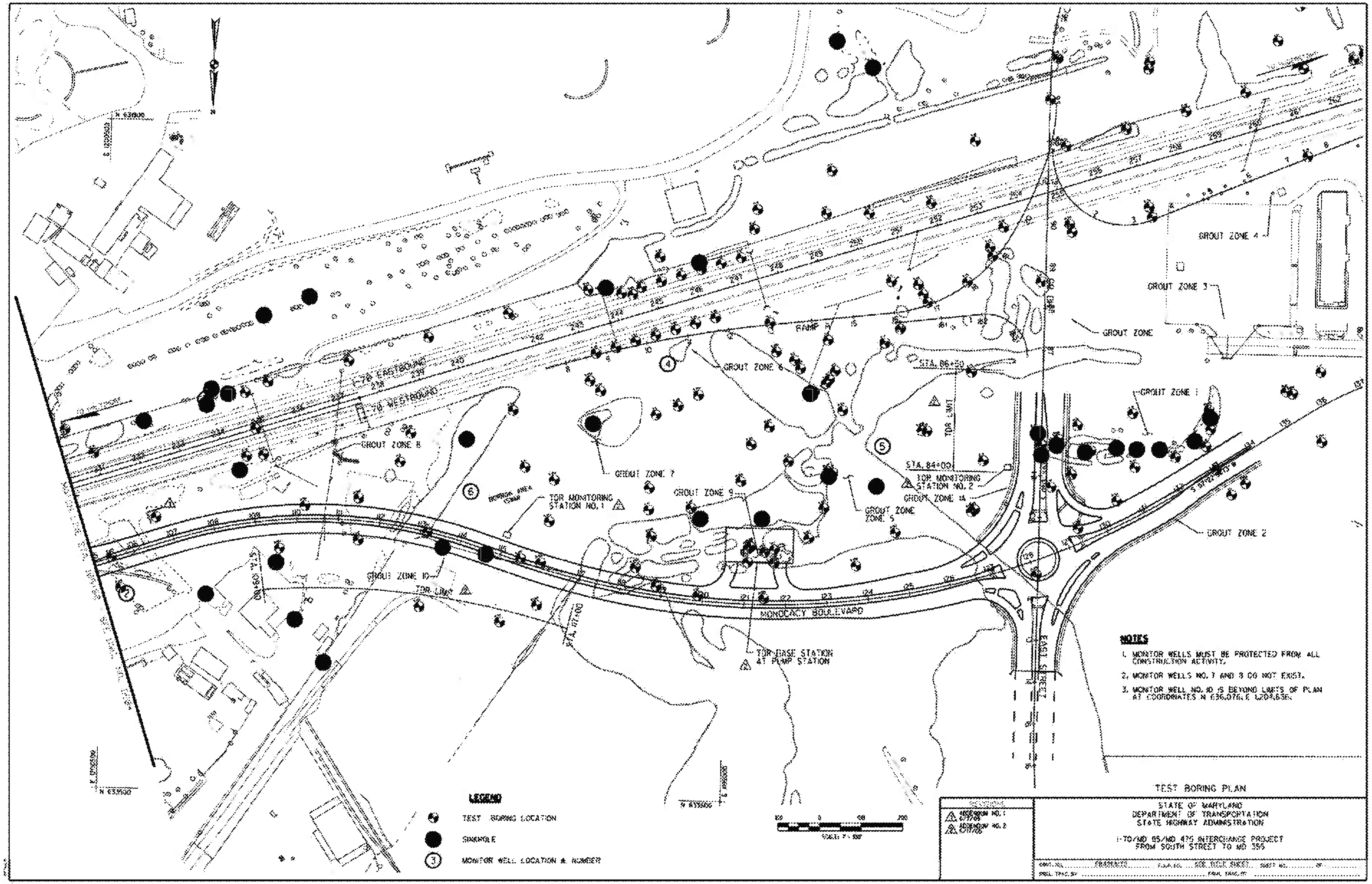


Figure 10 – Boring Location Plan

24-inch thick concrete pad. The pad is reinforced with No.8 rebar on 12-inch centers. The pad width is extended a minimum of 3 feet beyond the sinkhole limits (Figure 11).

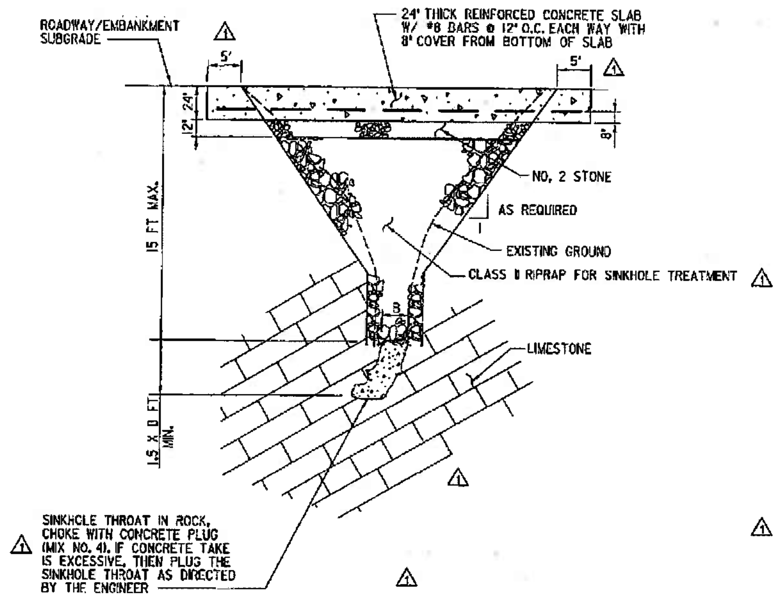


Figure 11 – Type A Detail: Typical Sinkhole Treatment Under Proposed Pavement

Type B For sinkholes beneath the pond liner, the throat is choked with low slump concrete after cleaning and the sinkhole is filled with class 2 riprap. The top treatment requires 24 inches of No 2.stone beneath the pond liner backfill (Figure 12).

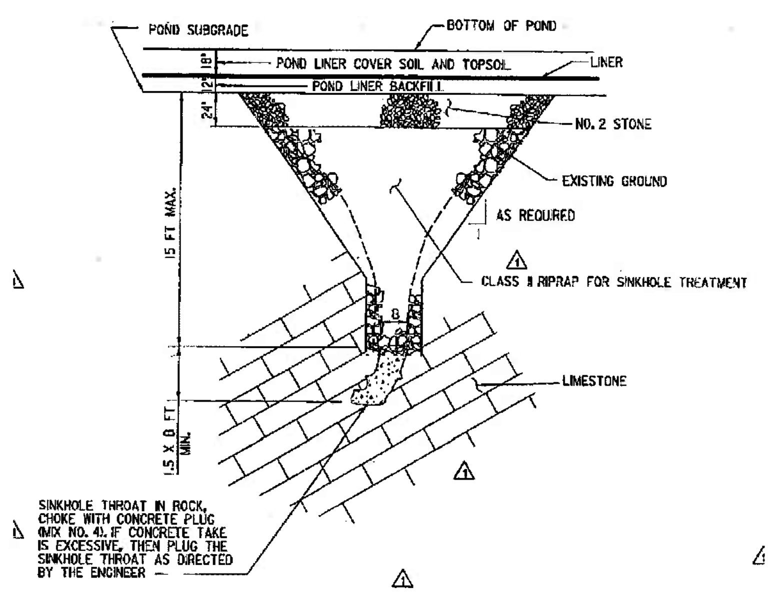


Figure 12 – Type B Detail: Typical Sinkhole Treatment Under Proposed Pond

Type C Shallow sinkholes in open ground are cleaned and backfilled with mix No. 4 (350 psi) concrete and covered with 8 inches of common borrow and finished with 4 inches of top soil (Figure 13). Sinkholes greater than 15 feet deep are filled with on-site rock and are grouted on a 20-foot grid pattern that encompasses the throat and surface depression (Figure 14). Secondary grout holes were incorporated on an as needed basis.

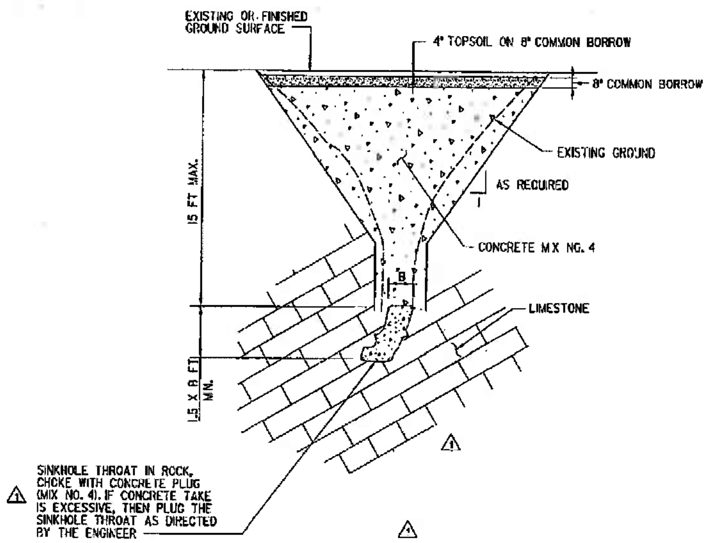


Figure 13 – Type C Detail: Typical Sinkhole Treatment for Shallow Sinkhole at Open Ground

LMDG Program

In order to address the on-going subsurface problems, in addition to surface treatments, a low mobility displacement grouting (LMDG) program was implemented to treat the sinkholes and attempt to prevent future sinkholes from developing at roadway and structure locations. Each grout zone was individually designed based on surface exposures of sinkholes and dolines, from mapping of paleo stream channels and soil types, fracture traces, detailed geologic maps and aerial photography. Geophysical studies, boring data and groundwater vector studies derived from computer analysis were also included. Grout zones limits were considered flexible and were varied in the field based on real time grouting results. The goal of the grouting was to fill the subsurface voids and associated solution channels in the epikarst. The grout was injected under pressure to create a grout cap between the soil overburden and epikarst zone or competent rock to prevent the migration of soil and water in a specific area.

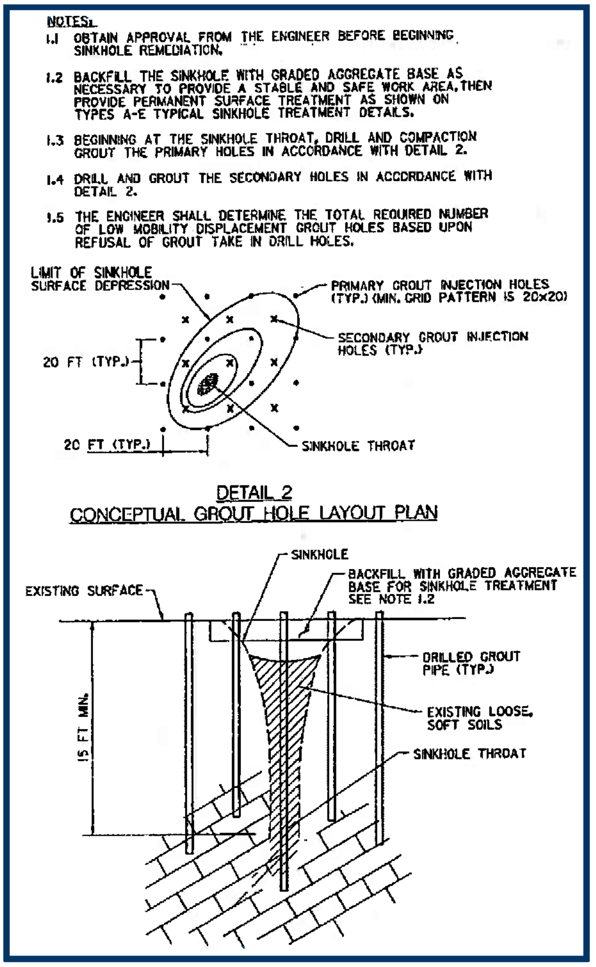


Figure 14 – Typical Detail for Sinkhole Treatment for Sinkholes Deeper than 15 ft

The LMDG program consisted of 13 grout zones for this project. The locations of each zone were determined based the proposed roadway, structure, pond, and earth embankment locations as well as past history of sinkhole activity. These zone locations are presented on Figure 15. During construction, substantial earthwork occurred in zones 1, 1A, 2, 3, 5 and 7 where sinkholes were exposed and addressed by previously discussed surface sinkhole treatment methods and were not grouted. Therefore, the LMDG program was reduced to 7 primary grout zones, which included 4, 6, 8, 9, 10, 11, and 12. In 2006, the grouting program for the I-70 improvement project included 681 grout holes and 42,227 cubic yards of grout.

Grouting for this project is currently on hold due to construction and utility constraints. Grouting is anticipated to resume during late fall or winter of 2007. This paper will focus on the grouting completed during spring and summer of 2006, specifically sinkhole related treatments pertaining to Monocacy Boulevard Extension and the storm water management facility, which included an 600-foot earth dam, the eastern and western approach embankments to the dam, and

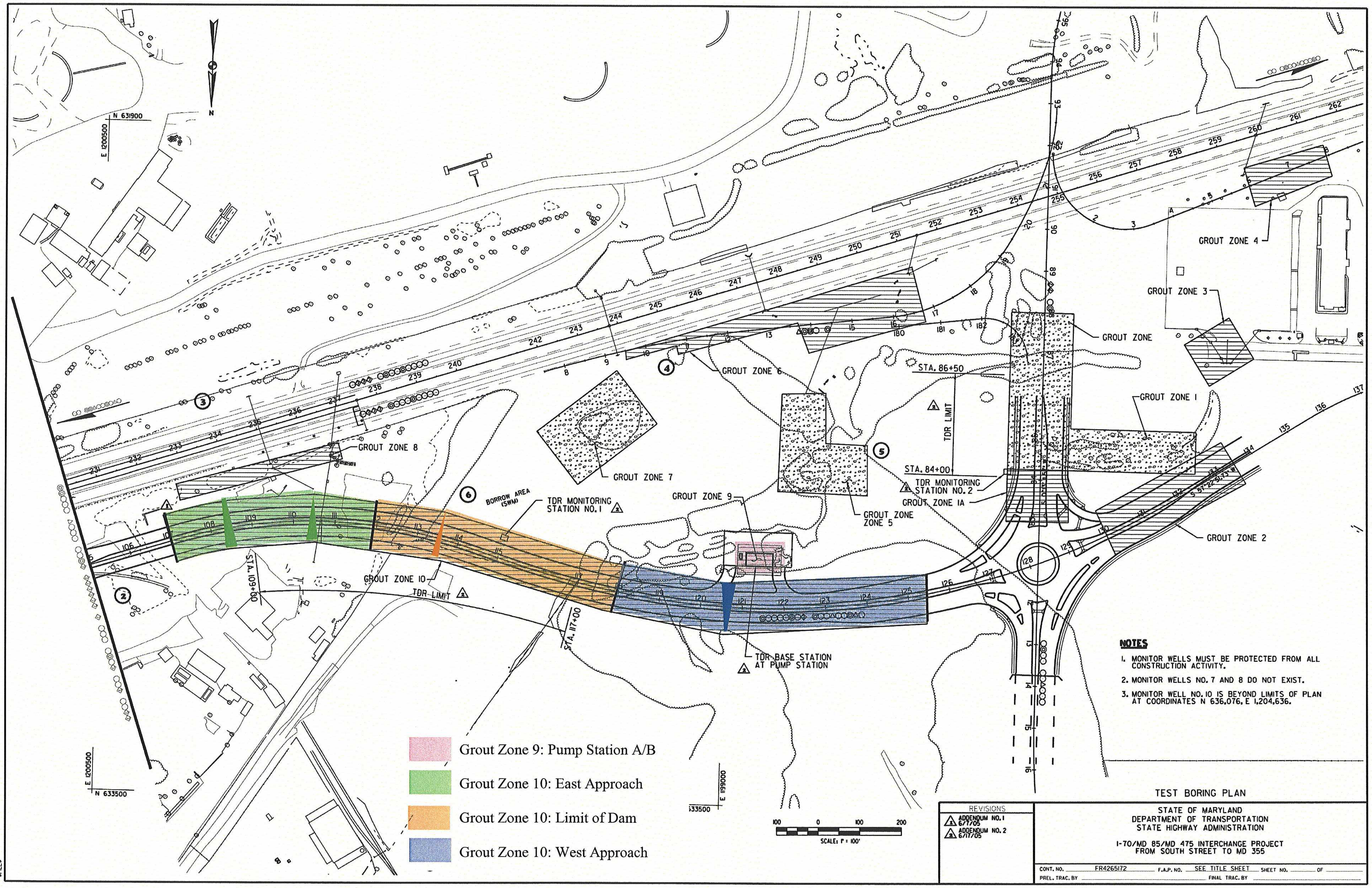


Figure 15 – Sinkhole Treatment Plan

pump station. Grout zone 9 addressed subsurface improvements for micropile foundations of Pump Station A/B between stations 120+95 and 122+06. Grout zone 10 was the largest zone in the LMDG program that addressed subsurface improvements along Monocacy Boulevard Dam and the east and west approaches between stations 107+00 and 126+20.

OVERVIEW OF GROUTING PROGRAM

A Geotechnical Specialty Contractor conducted the drilling and grouting operations for the I-70/ MD 85 Interchange, MD 355 and MD 475 Reconstruction Project. From March to August, 2006, grouting of zones 9 and 10 were conducted for a total of 523 grout holes and 35,648 cubic yards of grout. The LMDG program was conducted in accordance with the Low Mobility Displacement Grouting Special Provision. Some to the key aspects of this grouting specification were:

1. Mix Design: Portland Cement, Type F Fly ash, #10 Rock Dust, Water
2. 24hr compression tests on cylinders per T 22: 100 psi minimum
3. Slump test per T 119: less than 6 inches
4. Primary injection holes shall be drilled a minimum of 5 feet of rock
5. Appropriate measures should be ensure that the holes do not collapse or cave in prior to injection of grout
6. Grout injection should be started at maximum depth and continued as casing is withdrawn in 2-foot intervals

The grout zones generally consist of primary grout holes on 20-foot grid pattern. Secondary grout holes were normally located in the center of four primary grout holes. The grout holes within each zone were located based on survey reference point(s) and corresponding station and offset. Each primary grout hole was predrilled through the soil overburden, epikarst, and into five feet of competent rock using track or truck air rotary rig (Figure 16). Competent rock was determined by evaluation of the rock cuttings (color, hardness, etc.), rotation and down pressure and sound of hammer by the inspector and driller. For the purpose of this grouting program, top of rock was taken to be at the top of the five-foot continuous section of competent rock. If voids or soft zones were encountered within the rock, the grout hole was advanced until five feet of continuous competent rock was obtained. Each primary grout hole was logged by the driller generally indicating soil type, broken rock, or rock at corresponding depths. At the completion of each grout hole, the upper three feet of the grout hole was cased with four inch PVC pipe to prevent caving near the surface (Figure 17).

A second air-rotary track rig with a continuous grout mandrel (casing), was used for grouting the primary grout holes (Figure 18). This mandrel was used as the injection pipe during the grouting process and primarily consisted of five-foot threaded steel casings having a nominal three-inch inside diameter. Generally, the casing was lowered to within five feet of top

Figure 16 – Photo of EGT MD 5000 air-rotary track rig drilling primary grout holes

Figure 17 – Photo of Casagrande C-8 rotary track- rig with a lowered steel mandrel (casing) used for grouting of primary hole.

Figure 18 – Photo of Grout Zone 10 Where Grout was Pumped over 1000 lineal ft, note the use of PVC casing to prevent caving of hole.

of rock and grouted from the bottom to the ground surface. Bottom of casing depths were verified by the on-site inspector for each grout hole. The grout pressures were monitored at the injection point with a pressure gauge at the intake line/ gauge saver connection on the mandrel. The grout casing was raised when one of following occurred during grouting:

1. Excessive pressure built up in grout hole,

2. Ground heaved, and/or

3. Grout flowed from around casing or communicated with surrounding grout hole(s).

The design grout mix consisted of Portland Cement, water, Type F Flyash, and Rock Dust (#10). The base design mix is summarized in **Table 1** below and met the requirements of LMDG Special Provision. The cement, rock dust (#10), and flyash were trucked to the site daily by local suppliers.

Table 1 – Base Design Grout Mix				
Material	**Mix Weight (lbs)**	**Specific Gravity**	**Volume CF**	**Suppliers**
Portland Cement	281	3.15	1.4	St Lawrence Cement
Water	473	1.0	7.6	Frederick County
Type F Flyash	657	2.25	4.7	Mirant Energy
Rock Dust (#10)	2,226	2.69	13.3	Lafarge
Totals	3,637		27 CF	
Specification per Category 200 Special Provision				
Specified Item	**Specification Requirements**		**Actual per Design Mix**	
Cement Concrete	Minimum 150 lb per CY		281 lb per CY	
Minimum Strength, T22	100 psi after 24 hours		200 psi @ 24 hours	
Slump, T119	Less than 6 inches		5.75 inches	

Once the mixture was prepared by the plant, the grout was fed directly into a hopper of the concrete pump. The concrete pump contained a stroke counter to track grout quantities being injected into the grout hole. The pump was periodically calibrated with a 55-gallon drum (0.27 cubic yards) to determine the number of strokes per cubic yards for accurate calculation of grout volumes. Pump calibration varied from 18.5 to 48.15 strokes per cubic yards, because multiple pumps were used throughout the LMDG program. The grout was transported by the pump to the injection point by 4-inch diameter slick line.

Ground monitoring for heave was conducted throughout the low-mobility displacement grouting program. Monitoring was performed with a laser level mounted on a tripod and targeted monitoring points spread across the area of interest, which included existing roadway embankments, utilities, and structures. Every attempt was made during grouting operations to minimize heave. However, all heave could not be avoided, because heave occurs suddenly in unforeseen directions.

DATA COLLECTION AND INTERPETATION

This large LMDG program consisted of more than 500 grout holes in grout zones 9 and 10. The organization and management of the plethora of drill and grout data was key to the program's success. On any given day, there were multiple air-track rigs drilling and grouting holes. Air track rigs drilled a total of 29,436 linear feet during the grouting program and an approximate averaged of 290 feet per day. Two batch plants were operated simultaneously to supply grout to as many as three concrete pumps, which transported grout to three rigs or injection points. This two batch plant and three rig operation produced 35,648 cubic yards of grout for zones 9 and 10 with an average of 450 cubic yards per day and a maximum production of more than 880 cubic yards per day.

Given the numerous drill and grout data being produced daily, a tracking system was developed to monitor/record contractor's progress, quantities, and relative subsurface conditions per grout holes for each grout zone. Data recorded included hole number, depth to top of rock, drill (bottom) depth, casing depth, taped depth, pump calibration and strokes, grout take or communication, grout dated, and field observations (Figure 19). Part of the quality control of the grouting operations in the field was to verify that grout was being injected at the correct depths to create the grout cap. Grout casing depths were recorded on each hole and compared to the top of rock depths. Casing depths were not achieved (to top of rock) on some primary grout holes due to grout migration, sand caving due to grout migration, and/or sand/rock collapse prior to grouting. These primary grout holes were identified where there were significant differences (15 feet or greater) between bottom of casing and grout hole depths. As grouting operations were occurring, surrounding drilled grout holes were periodically taped using a 150-foot tape to monitor open depth and grout communication. Communication refers to the migration of grout from a grout hole that is being pumped to another grout hole. Field observations were noted based on casing depths, communication observed from taped grout holes (grout that did not reach ground surface in adjacent grout hole, but was observed from grout hole depth verification), areas of ground heave, communicated grout holes (grout holes that filled to the surface due to pumping of adjacent grout holes), and grout takes, where possible secondary grout hole locations were needed.

Driller and grout logs were submitted daily by the Contractor and reviewed by SHA and support staff. The subsurface conditions were generally described by the driller in four ways, which included sand/clay, void, broken rock, and rock. This subsurface data from the driller's logs was interpreted to identify three material types of interest when conducting low mobility grouting: depth of overburden, epikarst zones or features, and depth to top of rock (competent rock). Overburden was taken as the soil zone from the surface to top of rock or broken rock. Epikarst zones were defined as interbedded zones of broken rock, soil, and/or voids below the overburden. Depth to top of rock was defined as the depth where three to five feet of continuous, competent rock was encountered. An example drillers log with areas of interest (overburden, epikarst, and top of rock) identified is shown in Figure 20.

The quantities on the grout logs reflect the strokes per grout hole, pump calibration, and corresponding volume in cubic yards per day. The grout quantities were measured based on the stroke counter on the concrete pump. Upon the completion each grout hole, grout strokes were verified by the inspector. A grout hole was considered complete when the grout reached the surface and grout casing was removed from the grout hole. During grouting of certain grout holes, communication would occur with an adjacent grout hole. If grout from a pumped grout hole filled an adjacent grout hole to the surface, the communicated grout hole was deemed complete. A grout log was completed identifying the grout hole that was being pumped during communication. No grout quantity was assigned to the communicated grout hole.

Once the individual drill/grout hole data was recorded, reviewed, and interpreted, the goal was to efficiently incorporate this with other previous project data. Computer software

I-70/ MD 85 Interchange Project
FR4265172
Grout Zone 10 Tracking Sheet
August 2006

Boring	Approx. Elevation	Station	Offset* (ft)	Depth to Top of Rock (ft)	Top of Rock Elevation	Bottom Depth (ft)	Taped Depth (ft)	Bottom of Casing (ft)	Drillers Log	Grout Log	Entered into Surfer 8	Strokes	Calib.	Grout Take (CY)	Total Grout Take/ Boring (CY)	Grout Date	Working Days*	Status	Remarks
A2	286.26	10720	60	63	223.26	66	46, 36	40	x	x	x	980	24.07	40.71	87.54	8-Jun	45	C	Gravity grouted from 39'
												1127	24.07	46.82		12-Jun	46		
A4	285.99	10760	60	37	248.98	40	37	36	x	x	x	1550	24.07	64.40	205.94	7-Jun	44	C	
								38				3407	24.07	141.55		8-Jun	45		Gravity grouted from 34'
A6	286.07	10800	60	43	243.07	46	43	36	x	x	x	911	24.06	37.86	37.86	7-Jun	44	C	Gravity grouted from 23'
A8	285.91	10840	60	25	260.91	28	25	25	x	x	x	38	24.07	1.58	1.58	31-May	39	C	
A10	286.29	10880	60	35	251.29	38	25	25	x	x	x	56	24.07	2.33	2.33	31-May	39	C	
A12	286.13	10920	60	33	253.13	36	7.5	31	x	x	x	3137	24.07	130.33	130.33	30-May	38	C	
A14	285.73	10960	60	22	263.73	25		21	x	x	x	172	24.07	7.15	7.15	21-Jun	52	C	
A16	286.77	11000	60	26	260.77	29	23	23	x	x	x	427	24.07	17.74	17.74	24-May	36	C	
A18	286.99	11040	60	29	257.99	32		28	x	x	x	132	24.07	5.48	5.48	21-Jun	52	C	
A20	286.94	11080	60	32	254.94	35	33	33	x	x	x	875	18.5	47.30	47.30	16-May	31	C	
A22	287.52	11120	60	26	261.52	29		26	x	x	x	827	18.5	44.70	51.89	10-May	28	C	
												133	18.5	7.19		11-May	29		
A24	288.73	11160	60	35	253.73	38			x	x	x	27	18.5	1.46	1.46	15-May	30	C	
A26	289.45	11200	60	124	165.45	129		85	x	x	x	3512	18.5	189.84	193.46	11-May	29	C	
												67	18.5	3.62		15-May	30		
A27	289.17	11220	60	117	172.17	122	67,33	33	x	x	x	298	18.5	16.11	16.11	4-May	25	C	
A28	288.89	11240	60	55	233.89	60	53,40	45	x	x	x	3365	18.5	181.89	304.59	3-May	24	C	
								32				2270	18.5	122.70		4-May	25		
A29	287.53	11260	60	49	238.53	54		51	x	x	x	1326	18.5	71.68	71.68	3-May	24	C	
A30	285.11	11280	60	25	260.11	30		25	x	x	x	18	18.5	0.97	0.97	3-May	24	C	
A31	282.71	11300	60	51	231.71	56	56,45	45	x	x	x	695	18.5	37.57	41.41	2-May	23	C	
								25				71	18.5	3.84		3-May	24		
A32	280.95	11320	60	99	181.95	104	25	27	x	x	x	32	18.5	1.73	1.73	2-May	23	C	
A33	279.19	11340	60	52	227.19	57	48, 14	14	x	x	x	45	18.5	2.43	2.43	2-May	23	C	
A34	278.27	11360	60	50	228.27	55	50,8	32	x	x	x	500	18.5	27.03	27.03	1-May	22	C	
A35	278.19	11380	60	38	240.19	43	41, 18	35	x	x	x	1581	18.5	85.46	116.81	27-Apr	21	C	
								32				580	18.5	31.35		1-May	22		
A36	278.11	11400	60	70	208.11	75	56, 24	24	x	x	x	40	18.5	2.16	192.59	27-Apr	21	C	Redrilled
				NA	NA	50		46				6347	33.33	190.43		9-Aug	79		
A37	277.99	11420	60	40	237.99	45	25	24	x	x	x	36	18.5	1.95	1.95	27-Apr	21	C	
A38	277.87	11440	60	100	177.87	105	83		x	x	x	0	29.63	0.00	com w/A39	27-Apr	21	C	
A39	277.95	11460	60	117	160.95	122	64	39	x	x	x	2838	18.5	153.41	251.62	26-Apr	20	C	
								31				1817	18.5	98.22		27-Apr	21		
A40	278.23	11480	60	49	229.23	54	13	23	x	x	x	52	18.5	2.81	2.81	26-Apr	20	C	
A41	278.51	11500	60	61	217.51	66	61	43	x	x	x	2974	24.7	120.40	428.10	24-Apr	18	C	
								39				6588	24.7	266.72		25-Apr	19		
								33				758	18.5	40.97		26-Apr	20		
A42	278.99	11520	60	109	169.99	114	38	17	x	x	x	22	24.7	0.89	0.89	24-Apr	18	C	
A43	279.47	11540	60	66	213.47	71	4	5	x	x	x	13	48.15	0.27	0.27	19-Apr	16	C	
A44	279.48	11560	60	123	156.48	127	75	6	x	x	x	35	48.15	0.73	150.77	19-Apr	16	C	Redrilled
				NA	NA	50		50				5001	33.33	150.05		8-Aug	78		
A45	279.01	11580	60	120	159.01	125	21	12	x	x	x	37	48.15	0.77	13.97	19-Apr	16	C	Redrilled
				NA	NA	50		47				440	33.33	13.20		9-Aug	79		
A46	278.51	11600	60	55	223.51	60	-30		x	x	x	0	29.63	0.00	com			C	

Figure 19 – Sample of Tracking Sheet

COASTAL DRILLING
LMG PRE DRILLING LOG

DEWEY JORDAN SHA PROJECT

HOLE #	12	DATE	2/17/2006
ELEVATION	289.68	GROUT ZONE	12

DETPH FROM-	TO	DESCRIPTION	INTERPETATION
0	1	OVERBURDEN	OVERBURDEN
1	5	WET SAND	
5	9	ROCK	EPIKARST
9	20	WET SAND	
20	27	ROCK SEAMS	
27	28	SOFT ROCK	
28	33	ROCK	COMPETENT RK
		TOTAL DEPTH	33

Figure 20 – Example Drillers Logs with Zones of Interest

(Surfer 8) was used daily to translate, generate and transmit the data to the SHA Engineering Geology Division. This data summarized by the tracking sheets was entered into the Surfer 8 program to develop models of the subsurface by creating grout hole location (grid) maps, and depth to top of rock and grout contour maps for each grout zone. These maps/ models were used daily to evaluate subsurface conditions, grout quantities of primary grout holes, and the need for secondary grout holes, while directing grouting operations. Examples of these maps are presented in the Grouting Results section of this paper.

SECONDARY HOLE SELECTION PROCESS

The secondary hole selection process did not begin until all primary grout holes within a particular zone were grouted. This was done to access the subsurface conditions and corresponding grout takes across the zone for identifying problematic areas. Some of the conditions mentioned below are as a result of primary hole grouting with the dam limits of zone 10 and the secondary hole selection is discussed based on these results. Specific grouting results and subsurface conditions are discussed in the Grouting Results section of this paper. Secondary grout hole locations were determined based on the following:

1. Drill and grout data from the primary grout holes in the form of a rating system,

2. Evaluation of depth to top of rock and grout contour maps for identifying karst features, and

3. Field observations made during grouting operations.

The drill and grout data used for secondary location selection included depth to top of rock, casing depth, grout take, and grout time/order data. Within the Monocacy Boulevard Dam of zone 10, grout communication or shallow casing depths were observed in approximately 36% of the primary grout hole locations. These primary grout hole locations were focus area for secondary holes. In an effort to prioritize these primary grout holes for secondary locations, top of rock depths and grout takes for each grout hole were evaluated/ compared with surrounding grout holes. Three categories were determined based on top of rock depths of primary grout holes that were

1. Deeper than surrounding primary grout holes,
2. Shallower than surrounding primary grout holes, and
3. Intermediate depth compared to surrounding grout holes

Each category was given a rating. Primary grout holes that were non-representative-deeper were considered worst case and given a rating of 5. Primary grout holes that were non-representative- shallower were given a rating of 4. Primary grout holes that had top of rock depths that were in between the surrounding rock depths (intermediate) were given a rating of 3. Also considered in the rating system were primary grout holes that were near large grout takes (>100CY) that were grouted before a particular grout hole and those within an area of noticeable ground heave. These two conditions, next to large takers and ground heave, were considered possible justification for the bottom of grout hole and casing depth differences and were assigned a rating of -2 and -1, respectively.

The locations of field observations (grout communication and ground heave) were commonly associated with primary grout holes with a rating of 3. The primary grout holes with a rating of 5 and 3 with corresponding field observations were considered a priority for a secondary grout hole. The rating system that shows the priority primary grout holes is presented on Table 2. Although communicated grout holes were not specifically factored into the rating system (made a priority, but not assigned a rating number), the location of the secondary grout hole was placed in between primaries and next to a communicated grout hole(s) where applicable. This also served as a verification boring for the communicated areas.

Secondary grout hole locations were also determined by evaluating the Surfer8 generated depth to top of rock contour maps. Predominate troughs and depression features were identified on the top of rock contour maps across the zone. These features typically correspond to areas where the overburden and epikarst zones were the thickest, and are commonly the most sinkhole prone. Therefore, secondary grout holes were added where relatively low grout takes from the primary grout holes that were located within one of these predominate features.

Secondary holes were generally drilled in the middle of four primaries. The secondary grout holes are referenced to the primary grout hole to the southwest and are identified accordingly (e.g. E56-S: middle of E56, D56, E55, D55) to provide a reference location within the existing grid. The termination criteria for drilling of the secondary grout holes was to a depth of three feet of continuous, competent rock, to the average of top of rock depths of surrounding primary grout holes, or until grout was encountered, whichever occurred first. Drilling of the secondary grout holes was visually inspected to verify the termination depth. Frequent correspondence was

Rating System for Secondary Grout Holes

Category	Rating
Non Rep. Deeper	5
Non Rep. Shallower	4
Intermediate	3
Heave	-1
Next to Big Takers	-2

* Takers (>100CY) that were grouted before the referenced boring

Boring	Non Rep. Deeper	Non Rep. Shallower	Intermediate	Heave	Next to Big Takers*	Total Rating	Field Notes	Reference (southwest boring)
C51	5					5		
G49	5					5	VERIFICATION	
E47	5					5		
A44	5					5		REFERENCED TO B44
B42	5					5		REFERENCED TO C43
A39	5					5		REFERENCED TO B39
G32	5					5		
D28	5					5		REFERENCED TO D29
A26	5					5		REFERENCED TO B27
G26	5					5		REFERENCED TO G27
D30	5					5		
F28	5					5		
C47	5			-1		4		
F52	5				-2	3	VERIFICATION	REFERENCED TO F53
D45	5				-2	3		
E56	5				-2	3	VERIFICATION	
G55	5				-2	3		
B53			3			3		
G52	5				-2	3		
D50			3			3		
A49			3			3	VERIFICATION	REFERENCED TO B49
E49			3			3	VERIFICATION	REFERENCED TO E50
A48			3			3		
E45			3			3	VERIFICATION	REFERENCED TO E46
F45			3			3		
F44			3			3		
A43			3			3		
A42	5				-2	3		
D41	5				-2	3		
B34	5				-2	3		
F32			3			3		
B31			3			3		
B28	5				-2	3		
A27	5				-2	3		
E42	5			-1	-2	2		
B54			3	-1		2	VERIFICATION	
C48	5			-1	-2	2		
E48	5			-1	-2	2		
B27		4			-2	2		
E53			3		-2	1		
E52			3		-2	1		
D56			3		-2	1		
F54			3		-2	1		
B26			3		-2	1		
A45			3	-1	-2	0		

Table 2 – Secondary Hole Rating System

conducted with the driller to accurately record conditions of secondary grout holes. The secondary holes locations within the dam limits of zone 10 that were derived based on the rating system, top of rock/grout take contour map interpretations, grout communication and ground heave is presented on schematic plan on the next page (Figure 21).

PROBLEMS ENCOUNTERED DURING LMDG PROGRAM AND MODIFICATIONS IMPLEMENTED

The subsurface karst conditions were more developed than initially anticipated. Thus, the large grout volume was a concern with only approximately 30 percent of zone 10 complete. Some of the drill depths of the primary grout holes were deeper than expected in order to obtain five feet of continuous, competent rock. As a result, the pressure grouting was being conducting at depths that exceeded 100 feet and excessive grout migration was occurring as it crossed multiple fractured and solution erosion zones. It was believed that grouting at these depths (greater than100 ft) with the highly variable top of rock (most less than 100 ft), was less beneficial, because the migration was less uniform across the zone with respect to depth and more difficult to predict. Therefore, the drilling and grouting procedures were modified in an effort to reduce drill depths and grout volumes, but still create the grout cap at a higher elevation. The drill termination criteria was changed from five feet to three feet of continuous, competent rock. Grout hole spacing was also re-evaluated. Twenty percent of the primary grout holes within the dam limits communicated during pressure grouting of adjacent grout holes. This indicated significant lateral migration of grout and suggested that the spacing could be increased. Communication was more common where subsurface conditions were relatively uniform, mainly top of rock. Therefore, to evaluate the subsurface conditions of the remaining sections of zone 10, row D of primary grout holes (the centerline of zone) were drilled first at a 20-foot spacing. In areas where top of rock was relatively uniform or shallow based on the row D profile, a 40-foot by 30-foot grid spacing was used. In areas where top of rock was more variable, the grid spacing remained at 20 feet. Specific locations of grid spacing between stations 107+00 and 112+00, and station 118+00 and 126+20 are discussed following this section.

The modified grout procedure consisted of a 30 cubic yards per foot criteria, in which a grout limit was identified for each grout hole. The 30 cubic yards per foot criteria was derived based on grout volume that would fill an approximate 30-foot square area (void), 1 foot thick. This would provide an overlap of approximately 10 feet between the primary grout holes spaced at 20-foot centers. This would prevent pressure grouting at a same depth for long periods of time when pressure does not build up. The grout limit was determined based on subsurface conditions recorded on the driller's logs as described in the following cases:

1. In grout holes that consisted of only overburden (soil), broken rock, rock (competent), a 30 cubic yard limit was given to be applied at the broken rock/rock interface;

2. In grout holes that consisted of a void between the broken rock and rock, the grout limit was calculated using 30 cubic yards per foot for the entire thickness void;

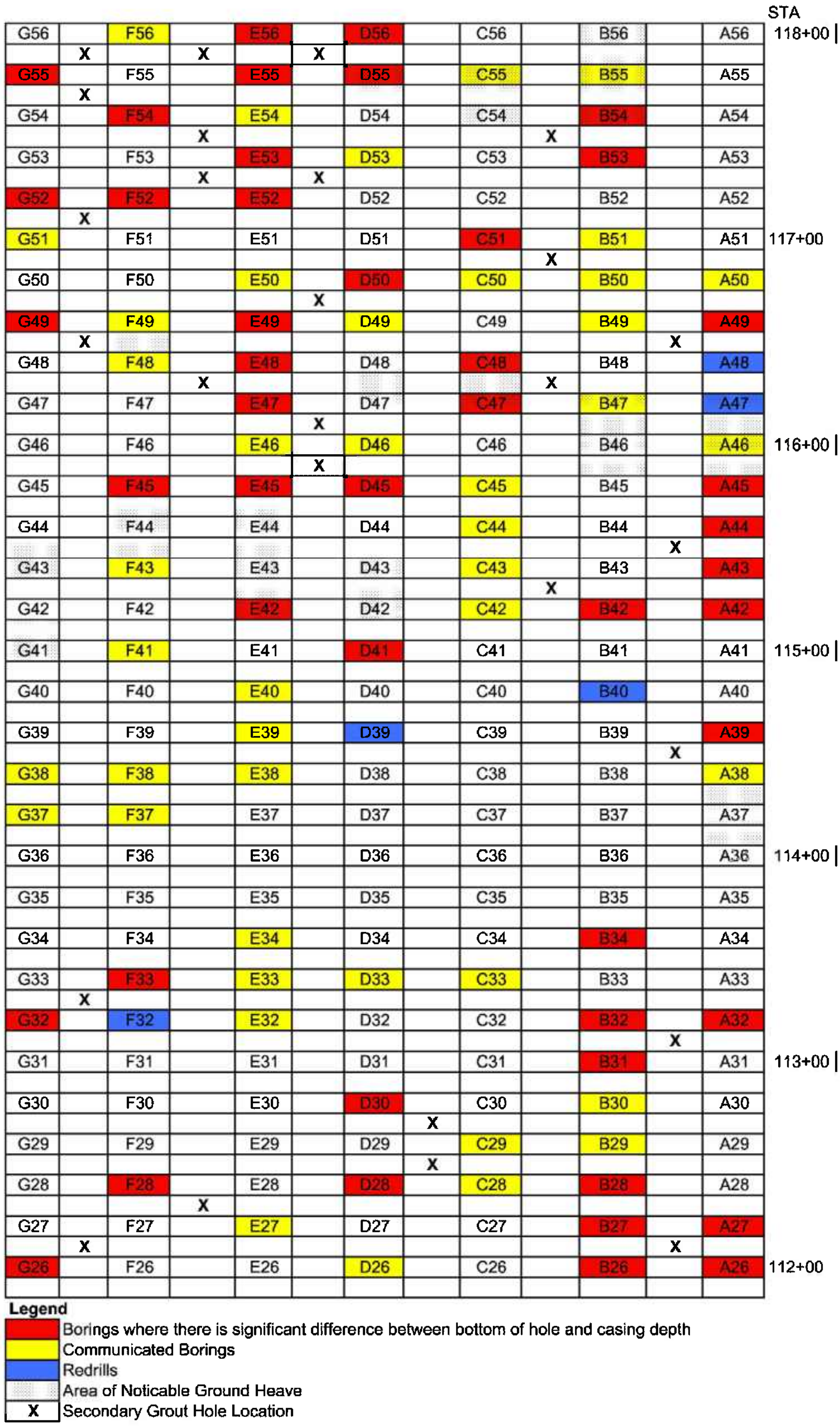


Figure 21 – Schematic Boring Plan Showing Secondary Grout Hole Locations

3. In grout holes where a broken zone was encountered between a void and top of rock, the grout limit was calculated using 30 cubic yards per foot for the entire thickness of the broken zone and void; and

4. In grout holes that contain more than one void, the grout limit was calculated using 30 cubic yards per foot to the top of the bottom void. In all cases, the grout casing was lowered to 1 foot above top of rock.

Example grout hole logs that show casing depth and grout limit are presented on Tables 3a through 3d for each case above on the next page. On grout holes where the calculated grout limit was greater than 400 cubic yards, no more than 400 cubic yards was pumped in one day and grout was allowed to set up overnight. The remainder of the grout volume was pumped the following day. When the grout limit was reached and the grout hole was not complete, the remainder of the grout hole was gravity grouted to the surface. When a particular grout hole continued to take and gravity grouting was unsuccessful after 15 to 20 minutes, the grout hole was terminated and the open depth was noted. The next day that particular grout hole was set up on, casing was lowered to the noted depth, and grouting continued with a 30 cubic yards limit.

The Contractor was provided the grout limits per grout hole in advance and his operations were planned accordingly to minimize mobilization and set up time within the grout zone. Generally, the grout holes that were deeper, contained voids, and had a larger grout limit were grouted prior to the shallower grout holes. This modified grout procedure also was applied to the remaining sections of zone 10 between Stations 107+00 and 112+00, and Stations 118+00 and 126+20.

Table 3a - Case 1 Grout Hole Log

Grout hole # F-2		Casing Depth: 43 ft
Depth (ft)		Grout Limit: 30 CY
From:	To:	Description
0	40	Clay/Sand
40	44	Broken Rock
44	47	Rock

Table 2b - Case 2 Grout Hole Log

Grout hole # E-16		Casing Depth: 46 ft
Depth (ft)		Grout Limit: 90 CY
From:	To:	Description
0	33	Clay/Sand
33	44	Broken Rock
44	47	Void
47	50	Rock

Table 3c - Case 3 Grout Hole Log

Grout hole # C-6		Casing Depth: 40 ft
Depth (ft)		Grout Limit: 330 CY
From:	To:	Description
0	20	Clay/Sand
20	30	Broken Rock
30	36	Void
36	41	Broken Rock
41	44	Rock

Table 3d - Case 4 Grout Hole Log

Grout hole # E-8		Casing Depth: 60 ft
Depth (ft)		Grout Limit: 420 CY
From:	To:	Description
0	23	Clay/Sand
23	29	Void
29	35	Broken Rock
35	36	Void
36	47	Broken Rock
47	50	Void
50	61	Broken Rock
61	64	Rock

As in most projects where a new process is introduced, in this case large scale production grouting, a review showed that the specifications could use some improvements. These shortcomings fell into two general categories. First, the specifications were derived from those used for the maintenance level (smaller scale) contract. The production line work for a construction project is a much larger endeavor than the repair of individual sinkholes. While there were some adjustments made to deal with this, more needs to be done. Specifically, LMGD is too stiff to be pumped great distances. Either the mix needs to be more fluid, which causes the loss of more grout, or the pumping distances need to be limited, which lengthens the schedule. Second, a grouting operation does not lend itself to the rigid scheduling that is found on a highway project. In this case, the grouting was in the critical path, and when it over ran in both the materials and time, both the project budget and completion date were impacted. It was not possible for this project, but whenever possible, grouting projects should be advertised separately from conventional construction.

RESULTS OF GROUTING

The karst conditions and grout results were found to be similar in all grout zones within the LMDG program. For purposes of this paper and to avoid repetition, only the specific results of grout zone 10, Monocacy Boulevard Extension, will be discussed below. Results of the entire LMDG program completed to date are available upon request through the SHA Engineering Geology Division

The grouting of zone 10 occurred in three sections. The station limits of the Monocacy Boulevard Dam are between 112+00 and 118+00. The east and west approach embankments extend from stations 107+00 to 112+00 and 118+00 to 126+20, respectively. Grout zone 10 was located along the Monocacy Boulevard extension and extended 60-feet left and right of centerline. Grout zone 10 consisted of 493 grout holes that were grouted between March 23rd and August 17th, 2006. The total grout volume for the zone was 30,926 cubic yards with an average of 63 cubic yards per grout hole. Thirty seven (36) secondary grout holes were drilled and grouted within the zone.

Two hundred and forty-five (245) primary grout holes were drilled on a 20-foot center-center grid and to a depth of five feet of continuous, competent rock within the dam limits. An additional 256 grout holes were drilled and grouted within the east and west approaches. The grout holes along the centerline of the zone in the approach areas were drilled first to evaluated rock conditions. Top of rock was found to be more variable. Therefore, the primary grout hole spacing consisted of 40-foot (station direction) x 30-foot (+/- offset direction) in areas where top of rock was relatively shallow (mostly less than 50 feet deep) and 20-foot x 20-foot spacing in areas were top of rock was significantly deeper or more variable. The primary grout holes were generally drilled to a depth of three feet of continuous competent rock to reduce drill depths. Top of rock varied from 8 to 160 feet deep across the zone. Due to the highly variable depths, predominate valley-like troughs and depression features were identified on the top of rock contour map (Figure 22).

Within the dam limits, grout holes were pressure grouted from the bottom of hole to the ground surface. The required casing depth for grouting was at or within five feet of top of rock.

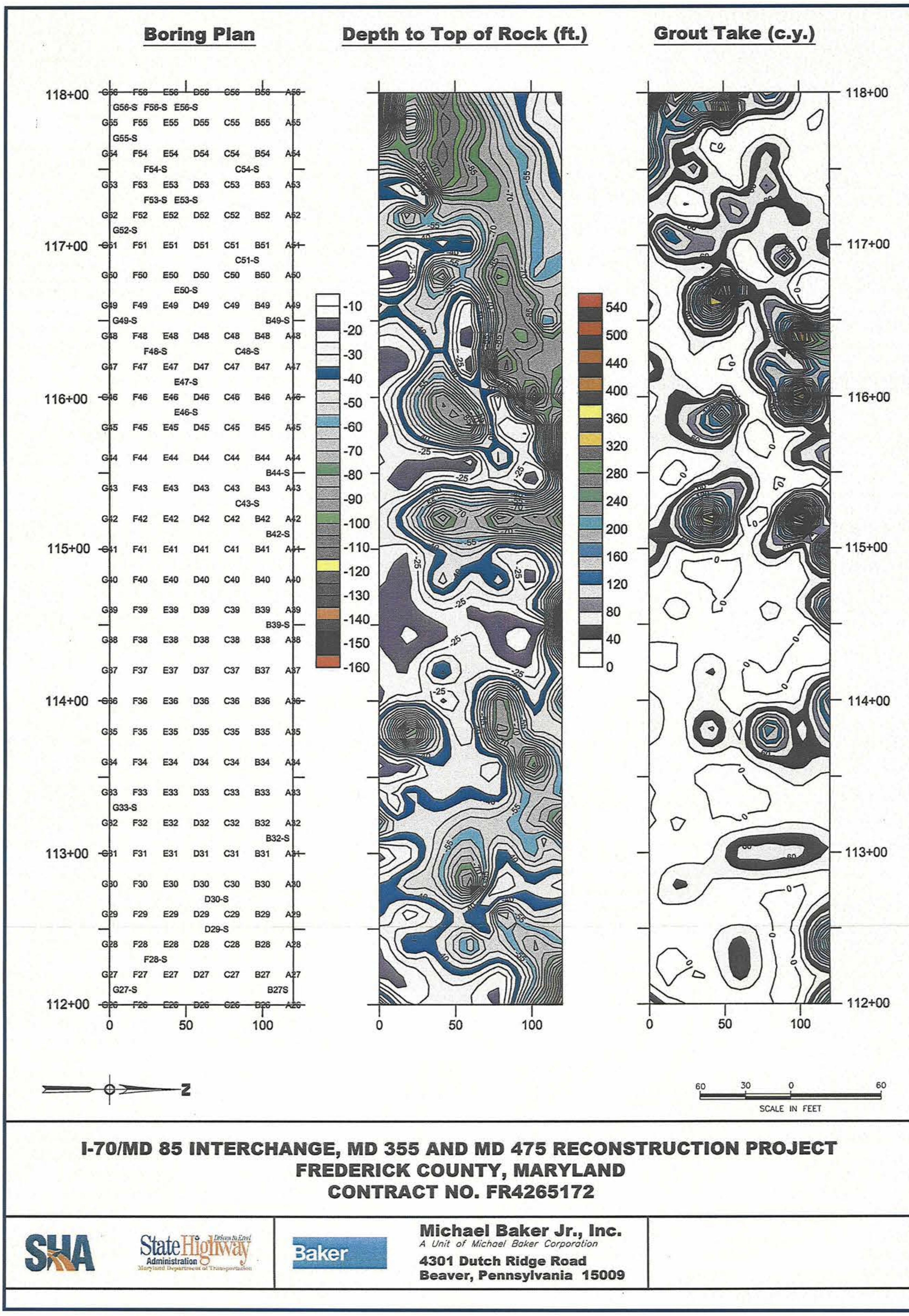


Figure 22 – Top of Rock and Grout Take Contour Maps for the Dam Limits of Grout Zone 10

Modifications to the grout strategy were applied to the east and west approaches of grout zone 10. Grout quantities were assigned to each grout hole based on subsurface conditions. Grout was concentrated at or near the soil overburden/epikarst and competent rock interface. The larger grout takes generally mirrored locations of the predominate valley-like trough and depression features as shown on the grout contour map (Figure 22).

LESSONS LEARNED AND CONCLUSIONS

Site Geology

Upon evaluation of the plethora of subsurface data from available geologic mapping/publications, field reconnaissance, aerial photos, geotechnical investigations, and the LMDG program, the geologic structure seems to have played a significant role in the karst development. Grout zone 10 lies in the saddle area of a double plunging syncline. Based on geophysical and grout take evidence, it is possible that a northwesterly striking shear zone exists in the saddle area. The double plunging syncline has been previously mapped showing a left lateral offset. This would account for the deep epikarst zone found in grout holes and higher then expected grout takes. Many secondary structures and faults are suspected but can not be mapped due to absence of outcrops.

Geophysics

Geophysical studies were incorporated into grout zone location selection and majority of design decisions concerning sinkhole prevention. Geophysics helped identified areas of thicker epikarst or sinkhole prone zones. The 2D correlations between the drill and geophysical data on specific cross-sections were less obvious. However, when the geophysical data was plotted over a larger area and compared with the drainage vector analysis using Surfer 8, predominate karst features were identified that correlated to existing and potential sinkhole prone areas. This was particularly useful in determining subsurface trends outside\between available drill data in an effort to understand the karst system in terms of a big picture (large area) approach.

LMDG program

The evaluation of the 2006 LMDG program is still in progress; however, it appears to have achieved stabilization at the roadway, structure, earth dam, and existing sinkhole locations. In the completed grout zones, no sinkhole activity has been observed. Many lessons were learned with respect to large scale production grouting and highly variable karst conditions throughout the LMDG program. The following is summary of lessons learned. The tracking system implemented during this program was found to be very useful to monitor drill and grout production for field QC, and data analysis and interpretation purposes. Surfer 8 software proved to be essential to expedite presentation of daily data for review, and ultimately directing grouting operations and secondary hole selection. Whenever possible, grouting projects should be advertised separately from conventional construction. Specifications can then be written to provide more individual control of contractor with respect to QC of grout mix, pumping distance, runaway grout, and drilling order/depths without impacts to construction schedule. Modifications can be made more easily to the drilling and grout strategy based on subsurface conditions and grout takes encountered to assure quality of grout cap.

A Look Ahead

Construction of the I-70/ MD 85 Interchange, MD 355 and MD 475 Interchange Reconstruction Project is ongoing where sinkhole preventative designs and treatments are being implemented. Evaluations of the preventative designs and treatments, as well as lessons learned will continue to be developed. For future roadway projects located within a karst environment, multiple designs related to sinkhole prevention such as careful storm water management, drainage liner, surface sinkhole treatments and low mobility displacement grouting are recommended.

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The Competing Needs of Development and Resources in the Karst Terrain of the Lehigh Valley, Eastern Pennsylvania.

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ABSTRACT

A large percentage of the Great Valley (locally the Lehigh Valley) is underlain by thick sequences of deformed Cambro-Ordovician age carbonate bedrock that is a valuable mineral resource. The weathering of these rocks along bedding discontinuities under temperate climatic conditions over long periods of geologic time has produced a karst surface that is generally hidden from view by a variable thickness of glacial and alluvial/colluvial sediments. Karst subsidence features such as sinkholes and surface depressions have propagated up through these sediments, but more often than naught; the land surface can leave very few clues to the casual observer of processes that may be occurring at depth.

The dissolution of carbonate bedrock has resulted in a natural plumbing system of pipes and drains that directly affects the distribution of surface and ground waters. Variations in water volume coincident with seasonal fluctuations in precipitation or by the alteration of drainage patterns through changes in land use can ultimately affect the rate at which sinkholes occur or generate problems of ground-water contamination and localized flooding.

The close proximity of the Lehigh Valley with respect to the New York and Philadelphia metropolitan areas is a major factor in the attraction of people and businesses. Population, households, and employment are all projected to increase by approximately 8% over the next 25 years (Lehigh Valley Planning Commission). On a par with an increase in land development is an increase in utility and transportation infrastructure and increasing demands on water and mineral resources.

Cultural migration can become problematic in karst areas. Oftentimes the application of standard land development practices, developed in non-karstic regions, simply does not apply in karst areas. From the initial concept through the engineering and construction phases of a project, karst areas require special consideration, particularly in infrastruture design and storm water management.

This paper will discuss the general impact of land-use change on karst terrain but will focus on a recent case in the vicinity of Stockertown Borough, Northampton County (see Petrasic, these proceedings). Its story is unique with respect to the relatively short period of time involved and the degree of significant damage to public and private property.

REGIONAL KARST GEOLOGY

Thick sequences of structurally deformed Cambro-Ordovician limestones and dolostones underlie approximately two-thirds of the Great Valley (locally known as the Lehigh Valley) of eastern Pennsylvania (Figure 1). Folds in the carbonate bedrock, often overturned and attenuated, and numerous faults have resulted in extensively fractured bedrock. Weathering of the deformed carbonate units under varying climatic conditions has produced a generally subdued, but deeply developed, karstic landscape (Wilshusen and Kochanov, 1999). Sinkholes and surface depressions characterize the area (Kochanov 1987a, 1987b). Caves are not overly abundant (Stone, 1932; Wheeland, 2007).

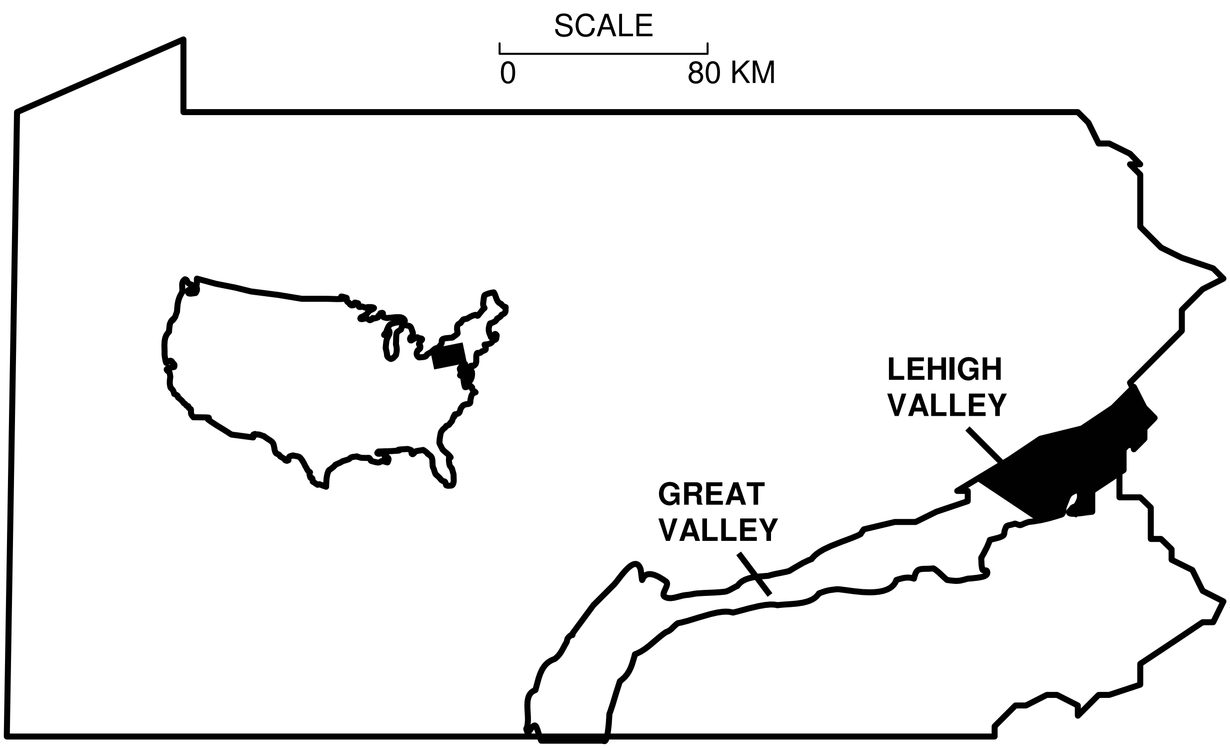

Figure 1. Location map showing the Lehigh Valley.

The topographic relief of the karst land surface is relatively low and can appear "flat" over large areas of the Lehigh Valley in comparison to other karst areas such as the Western Pennyroyal region of Kentucky. In the karstic areas of the Lehigh Valley, surface depressions can vary in size but for the most part are shallow with very little relief. This is due to the covering of bedrock with alluvial, colluvial, and glacially derived sediments of variable thickness. Coupled with land disturbance from urban and rural activities, surficial evidence for these karst subsidence features can be difficult to discern.

The Ordovician Epler Formation along with the Cambrian Allentown Formation account for 85 percent of recorded sinkholes (Wilshusen and Kochanov, 1999) and have the highest number of karst features per unit area in the eastern third of the Great Valley (Kochanov, 1993). This trend is maintained with correlative stratigraphic units further west. The Ordovician Jacksonburg

Formation, on the other hand, shows one of the lowest rates per unit area. This is attributable to its argillaceous composition.

The Epler Formation is highly karstic but the degree of karstification can vary regionally. Density patterns on recent mapping shows that the Epler can exhibit areas with a high density of karst surface features yet have low-density areas or areas lacking observable surface features within the same outcrop belt (Kochanov and Reese, 2003a; Reese and Kochanov, 2003b). Although lithologic composition is an important variable in karst development, structural deformation is the key component in determining the pathways for groundwater movement. It is the orientation of bedrock discontinuities that directs groundwater flow and, coupled with carbonate mineralogy, determines areas of preferential dissolution and the development of karst topography. Dolostones are also a bit more brittle than limestone and where structurally deformed, would be more fractured, allowing more surface area to come into contact with acidic groundwater and in turn, make some dolostones more “karsty” than limestones even though the limestone may be more chemically soluble. The interbedded nature of the Epler, where more soluble limestone is in contact with more fractured dolostone, may also help to direct the carbonate dissolution process. The geographic distribution of these limestone-dolostone sequences could account for the distribution pattern of sinkhole occurrences within the Epler as well as the Allentown Formations (Kochanov, 2006).

Karst Surface Features

In areas underlain by carbonate bedrock, karst surface features cover a somewhat narrow range of morphological types. Through field observations, one can usually distinguish two main features, the hole in the ground (sinkhole) and surface depressions. Sinkholes being differentiated from surface depressions by the rupturing or breaking (collapse) of the ground surface. (Note: The reader is referred to Beck (1984), Jennings (1985), White (1988) and Ford and Williams (1992) to further examine the definitions of the related terms: “sinkhole,” “doline” and “closed depressions.”)

Thick residual soils coupled with differential weathering of the underlying bedrock can be characterized by surface sags and depressions and perched ponds (Parizek and White, 1985). Perched ponds have been observed in carbonate bedrock areas in the Great Valley but also in non-carbonate areas. Along the flanks of South Mountain in Cumberland and Franklin counties, ephemeral ponds are common. Plant material from a core within one such pond had an estimated age range of 14-16,000 BP and had been identified as plants common to tundra areas (Delano and others, 2002). Tundra vegetation has also been recorded in Berks County (adjacent to Lehigh County), some 60 km from the Late Wisconsinan ice border (Watts, 1979).

The establishment of a periglacial setting in conjunction with a karstic surface raises the question as to whether or not identified surface depression features are of karstic or glacial/periglacial origin. Braun (1994, 1996) discusses surface depressions that occur in the slate and shale belt in Lehigh County both inside and outside the glacial limit and suggests they are of periglacial origin. Smaller-scale periglacial depressions have been interpreted by Braun (1996) for similar features observed in the carbonate belt of the Lehigh Valley that are superimposed atop larger-scale karstic dissolution features. These periglacial depressions commonly contain wetlands and perennial ponds and differentiate them from the surrounding karst landscape.

Marine oxygen isotope records and radiometric dating of terrestrial volcanic and glacial deposits have been used as an indicator of alternating cold/warm periods as well as associated glacial advances (Braun, 1999). These records suggest that as many as ten glaciations may have approached the late Wisconsinan terminus and four probably reached beyond that limit in Pennsylvania (Braun, 1989; 1999).

During glacial advances, shallow karst features (e.g., pinnacles, karren, shallow aquifers, bedrock pavement) would have been removed or altered through glacial scour (Ford and Williams, 1989). This would have been followed by periods of renewed karstification during warmer, inter-glacial times and in turn, these karst surfaces would have been destroyed by the advance of later glaciations. However, even though the karst *surface* features may have been removed, it does not preclude the probability that the *subsurface* conduits that were created during those inter-glacial periods, were preserved. The degree of connectivity and extent of the subsurface conduit system would have been extended by the cyclic rise and fall of regional and local base levels due to glaciation and post-glacial drainages (Kochanov, 2006).

The present-day karst surface would, of course, exhibit variations in the degree of karstification based on bedrock mineralogy, structural deformation, and surface and subsurface water drainage patterns. Conduit development would be dependent on the amount of surficial material (mantle) covering bedrock and the degree of in-filling with sediment along bedrock discontinuities. Some of the “pipes” in this natural plumbing system would be filled with sediment, impeding flow and redirecting water flow through more “unclogged” areas.

These conduits are generally unmoving static features as chemical and mechanical weathering and erosional processes continually deepen and widen these zones over long periods of time. As carbonate components are removed, the residual, non-soluble minerals can remain *in situ* or be transported down gradient.

The *in situ* residuum can often be observed as a porous rind of variable thickness following the irregular profile of the bedrock – soil interface. In some instances this residuum is of considerable thickness leaving the framework of the bedrock essentially intact even to the point of retaining the original layering of individual beds. This saprolitic layering in turn can be gradational with the overlying soil or other allogenic sediments (Kochanov, 2005).

Water pathways through the regolith may be initially small, with the infiltrating water filling the interstitial spaces between soil granules and within macropores. Over the course of time they continually change course and evolve into more complex water insurgence systems linking with other pathways that eventually lead to the conduits developed in the soluble bedrock. These macropore insurgences mimic the function of fractures and bedding partings in the carbonate bedrock in that they provide a means for water to flow vertically and laterally (Kochanov, 2005).

Ponors or swallow holes can be considered the end result of this linkage process between surface drainage and the water table. Ponors are dynamic fixtures in the karst plumbing system that varies in size, shape and location within the regolith, changing with the seasonal fluctuations of groundwater and insurgent water. One of the most common forms is the alluvial ponor as discussed by Milanovic (1981) where water is gradually lost along the length of a surface stream through the unconsolidated sediment lining the streambed.

CULTURAL INFLUENCES

The Lehigh Valley has experienced a steady increase in population and subsequent land use changes as a result from residential and commercial development over the last several decades. Central to this development has been an increase of residents from nearby New Jersey and New York moving into portions of the region since the early to mid-1980s. This migration continues and is a particularly important component of population growth in Northampton County (LVPC, 2005).

Population in the region is projected to grow steadily through 2030 with most growth expected to be in the suburban townships on the perimeter of the major urban areas, Allentown, Bethlehem, and Easton. Much of this projected growth will be in suburban townships (i.e., those townships bordering the major cities) (LVPC, 2005).

There are no state laws in Pennsylvania that regulate land development. Each municipal government, in general, develops their own plans to guide land use and zoning. The lack of congruence between municipal ordinances typically leads into urban sprawl. This is particularly apparent in rural townships where scattered subdivisions are commonplace. These islands of residential development are accompanied by a seemingly ill-conceived collage of open space and strip commercial development. In the Lehigh Valley this hodgepodge of development can be further complicated by the occasional presence of surface mines as well as isolated commercial and industrial sites (LVPC, 2005).

It follows that development would increase along major highways. Most of the local roads built in the past 30 years have been built by developers according to local subdivision ordinance requirements. The design of roads within subdivisions focuses on maximizing building opportunities, minimizing improvement costs to the developer, and meeting the technical standards in subdivision ordinances. Rarely do the design characteristics of roads take into account the overall municipal circulation pattern (LVPC, 2005). Interestingly, similar observations were also noted by Willard (1939), "… In that portion of the county (Northampton) underlain by limestones the roads run in every direction…"

Development patterns almost always out pace the capacity to deliver transportation infrastructure. In areas of rapid growth, major highway and transit construction projects developed in the 1970s are inadequate to handle 2005 traffic. There is often no connection between land use and transportation within most local municipal land use plans. Most local plans don't even contain a transportation element. Where a municipality is involved with transportation, is primarily with the regulation of local streets through the subdivision review process (LVPC, 2005).

These practices are not unique to the Lehigh Valley but exist throughout Pennsylvania and across the nation. On a par with an increase in land development is an increasing demand on utility and transportation infrastructure as well as increasing demands on water and mineral resources.

In 2002, the LVPC completed a preliminary assessment report of the Valley's water resources to identify current and future well water users of all types through 2030 and water availability during normal and drought conditions. Types of users include community and central water systems and users with their own individual well such as commercial agriculture production operations, golf courses, residential, commercial/industrial and water bottling operations, among others. From the available data, it was found that well water demand will not exceed groundwater supply during normal and drought conditions through 2030. However, one of the main findings of the assessment was the lack of up-to-date, reliable data on water usage, groundwater recharge and water quality.

Subsidence in carbonate terranes is a water-driven process that is often accelerated by cultural activity. In Lehigh and Northampton counties, 46 of the 62 municipalities are underlain entirely or in part by carbonate rock. These carbonate formations are located in the Lehigh Valley's urban core and as these areas become increasingly developed, the potential for karst-related problems increases. Intensive land use has led to many incidents of subsidence (Knight, 1971; Myers and Perlow 1984, White and others, 1984; Wilshusen, 1979; Wilshusen and Kochanov, 1999).

A number of these activities can account for the majority of sinkhole occurrences. Examples vary from the failure of water-bearing utility lines (Berry, 1986) to groundwater withdrawal from quarries (Foose, 1953). As urban areas expand, storm-water runoff is redirected and concentrated. Sinkholes have been observed in storm-water retention basins and storm drains as a direct result of heavy precipitation and increased storm-water runoff. In some instances, sinkholes have been used as storm-water drains (Wilshusen and Kochanov, 1999). Additionally, sinkhole development can be significantly increased through abnormal precipitation events such as hurricane Agnes in 1972, which dumped approximately 43 cm (17 inches) of rain in the Susquehanna River drainage basin over a three-day period.

Areas that have already undergone development have special problems in redesign and reconstruction. The after-the-fact methods of subsidence repair are often expensive and offer no guarantee from sinkhole reoccurrence. Sinkhole repair for the Vera Cruz Road in Lehigh County cost nearly $800,000 (US) and had a new sinkhole open, just outside of the repair area, within six months (Wilshusen and Kochanov, 1999).

A primary cause for subsidence problems is the failure to be cognizant of karst processes and their impact, prior to land development. In most cases, the local zoning laws are ineffective or nonexistent in regulating land development in potential subsidence areas.

CASE HISTORY: SINKHOLES AT STOCKERTOWN

Since the fall of 2000, increased sinkhole activity south of the Borough of Stockertown has resulted in significant damage to public and private property. Two of three bridges spanning the Bushkill Creek have experienced significant settlement or collapse. Two of the State Route (SR) 33 spans have been razed and rebuilt while the SR 2017 Bridge, serving the local communities of Stockertown and Brookwood, was recently razed (spring 2007).

The Bushkill Creek of Northampton County is located in eastern Pennsylvania. For purposes of this paper, site locations will refer to the Bushkill Creek section south of the Borough of Stockertown (Figure 2). Lying within the Great Valley Section of the Ridge and Valley

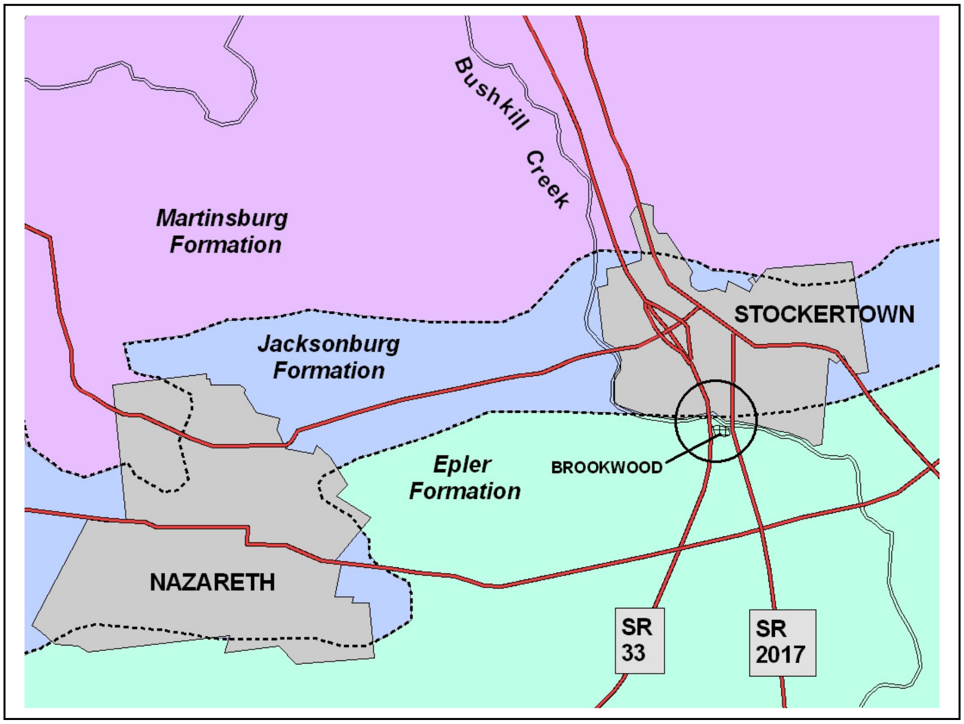

Figure 2. Geologic base map showing Boroughs of Stockertown and Nazareth, the Brookwood community, State Routes (SR) 33 and 2017, and the Bushkill Creek. The circle represents the active sinkhole area. Nazareth 7.5 minute quadrangle; coordinates, 40°44’55.51’’, 75°15’44.43’’

Physiographic Province, the stream has its headwaters to the north at Blue Mountain, a ridge of predominantly siliciclastic Lower Silurian-aged bedrock. As it flows south of the ridge, the creek crosses Ordovician slates and shales of the Martinsburg Formation then across mixed shales, shaley limestone and limestone of the Jacksonburg Formation, and finally across interbedded limestone and dolomite of the Epler Formation. The transition is from non-carbonate bedrock to a karstic carbonate sequence.

Surficial geologic mapping by Braun (1996) indicates pre-Illinoian glacial till and lag deposits over much of Northampton County. The tills tend to lie in the often straight-lined drainages that are oriented with regional bedding and joint patterns. The glacial material is evident in sinkholes in the Stockertown area (Figure 3).

Structural geology in the area is complex. Bedding generally strikes in an east-west direction with variable dips that reflect low to high-angle folding. High-angle joints strike north-south with a variance of 20 to 30 degrees towards the east. One feature, the “Stockertown Fault,” or fault complex as discussed by Epstein (1990), is interpreted as one of a series of imbricate folded thrust faults in this general area.

Based on the east-west strike orientation of faulting observed in the nearby Hercules Quarry and the general straight-lined nature of the Bushkill Creek, it is suggested that the "Stockertown Fault," may extend along the Bushkill on the southern border of Stockertown (Figure 2).

Figure 3. Sinkhole along the north bank of the Bushkill Creek showing coarse glacial till (T) overlain by finer alluvial sediment (A).

Abundant fractures noted in the drill logs for the majority of core borings in the SR 33/2017 stretch (SAIC, 2002) are supportive of this interpretation.

For a more detailed account of the geology see Aaron (1971), Miller (1939), Drake (1967), Sherwood (1964) and Epstein (1990).

Miller (1939) noted the limited occurrence of streams in the limestone regions of Northampton County and that in the area of the Bushkill Creek, and the relative direction of precipitation (i.e., "the rain water all disappears underground.") The Bushkill Creek, at least in part, is an alluvial ponor (swallow hole). It is a losing stream with a significant percentage of the stream waters in a state of divergence through alluvial and glacial sediments that cover a well-developed karstic bedrock surface (Kochanov, 2005). The stream loses approximately 70 percent of its water between the SR 33 bridges and the SR 2017 bridge (D. Zeveney, U.S. Army Corps of Engineers, pers. comm.).

Sinkhole History

Sinkholes began to form along the stream and within the floodplain of the Bushkill Creek in 1999. The main areas of occurrence were at the SR 2017 bridge and along the south bank of the creek. The sinkholes were of the "cover collapse" variety; bedrock was not visible in any of the sinkholes. On average, sinkholes generally ranged from 1-3 meters in diameter, 1-3 m deep but some have been larger and deeper. Sinkholes adjacent to streams do not contain water, implies that sinkhole formation propagates from below. To the present date, over 330 sinkholes have been documented within a 4.2 km (2.6 mi.) radius south of the Bushkill Creek at Stockertown.

In the summer of 1999, sinkhole occurrence began within the channel of the Bushkill Creek followed by sinkholes on the north end of the Brookwood community in the fall of 2000. One residential property in particular had a large sinkhole open in the backyard with over $20,000 in remediation costs. Continued subsidence eventually forced the residents to abandon the house. From there, sinkholes continued to open within the Creek and along the banks of the stream (Figure 4) eventually compromising the Route 2017 bridge (Figure 5) and undermining the approach roadway to the bridge. Over the course of the next few years, individual sinkholes around the 2017 bridge coalesced to make one large subsidence area (Figure 6). Sinkhole activity was not limited to the 2017 area but also upstream and in the medial area of SR 33.

Figure 4. A large sinkhole on the south bank of the Bushkill Creek in 2001.

In April 2001, large sinkholes opened along the south bank of the Bushkill with a portion of the Norfolk Southern railroad bridge being damaged.

During January of 2004, sinkhole activity focused around the abutments of the northbound span of SR 33 bridge. It was during this time that the 33 bridge suffered serious structural damage, was razed and a new span constructed. In a proactive measure, the southbound span was also replaced.

Figure 5. The SR 2017 bridge in October 2002.

Figure 6. Coalescing sinkholes along the SR 2017 bridge in 2002. Photo courtesy of the Brookwood Group.

Concurrent with the damages to the bridges, a small drama was unfolding along the north bank of the Bushkill Creek near SR 33. Two sinkholes opened, coalesced, and created a nick point along the bank allowing the Creek to develop a small meander bend (Figure 7).

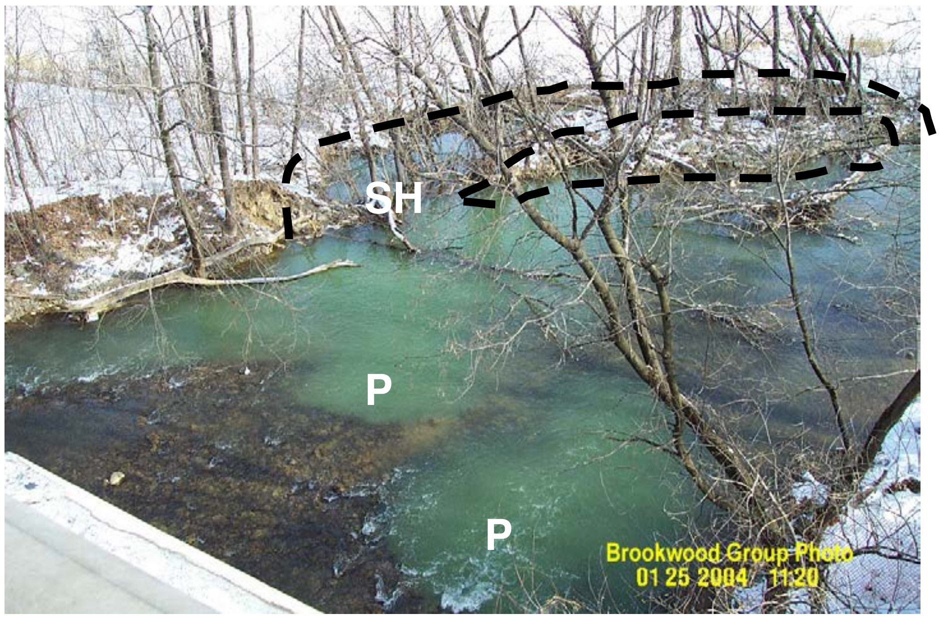

Figure 7. View from the SR33 bridge in January, 2004 showing the location of sinkhole pools (P) in the bed of the Bushkill Creek. A sinkhole (SH) had opened along the bank and has started to divert water from the stream. This diversion helped to form a small meander loop (dashed line). Photo courtesy of the Brookwood Group.

The formation of the small meander loop was intriguing and resulted in much speculation as to the end result of the stream breach. It was hypothesized that the stream might be attempting to revert back to a previous channel configuration. This goes back to the construction of SR 33 (late 1960's through early 1970's). At that time the channel of the Bushkill Creek was split around a mid stream gravel bar (Figure 8a). As part of the construction activity, the southern channel of the Bushkill was filled and the entire stream flow was directed into the northern channel (Figure 8b). The hypothesis was that the stream would play out a game of connect-the-dots, where the stream would link to new and existing sinkholes, starting with those in the field and finally connecting to sinkholes at the SR 2017 bridge. By doing so it would create a new island and have the split channel reform. What really happened, however, was that the stream did connect up with sinkholes but the stream ended up going in a different direction (Kochanov, 2005).

During the summer of 2004 the remnants of Hurricane Ivan dumped approximately 18 cm of rain in the Northampton County area. The resulting flood wiped out the remaining meander “neck” creating a new recessed bank edge (Figure 9a). This recession brought the stream closer to an exis eld (

Figure 8. a.) Aerial photograph from 1939 on the left shows the island bar on the Bushkill Creek (arrow) with the channel split circumventing the island. b.) The 1971 photo on the right shows the location of SR 33 and the modified channel of the Bushkill Creek. Flow of the Bushkill is from left to right, north is up.

In December of 2004 a breach developed connecting the sinkhole in the field and the Bushkill channel (Figure 9b). In the following week, additional sinkholes opened in advance of the prograding stream as the stream was pirated across the field. The stream had nearly advanced across the field and threatened to link up with one recently activated sinkhole near the SR 2017 highway and potentially affect the approach to the SR 2017 bridge (Figure 9c).

A thin finger of land was serving as a partial barrier during these events (Figure 9d). It was plain that if this neck of land failed then the major flow of the Bushkill would have followed the route across the field. This prompted a rapid response to temporarily fill in the new stream segment (Figure 10) to prevent further deterioration of private property and potential damages to SR 2017 (Hill, 2005). This action by the State Department of Environmental Protection, put an end to the progradation of the Bushkill Creek across the field at this time and put a stopper in this interesting case of sinkhole piracy.

The story continued with a new suite of problems that arose after the completion of the rebuilt SR 33 bridges in 2004 and 2005. The northbound 33 pier, located on the north side of the Bushkill Creek, began to move. Motion was detected going in a south and easterly direction. This resulted in an intense geo-technical investigation to further characterize the subsurface geology and hydrology. The primary focus was the around the bridge piers and along the stretch of the Bushkill between SR 33 and SR 2017. See Petrasic (these proceedings) for a more detailed discussion.

DISCUSSION (adapted from Kochanov, 2006)

The overriding thought throughout this case history is what was causing the sinkholes to occur in ... bles ... weight at any o...

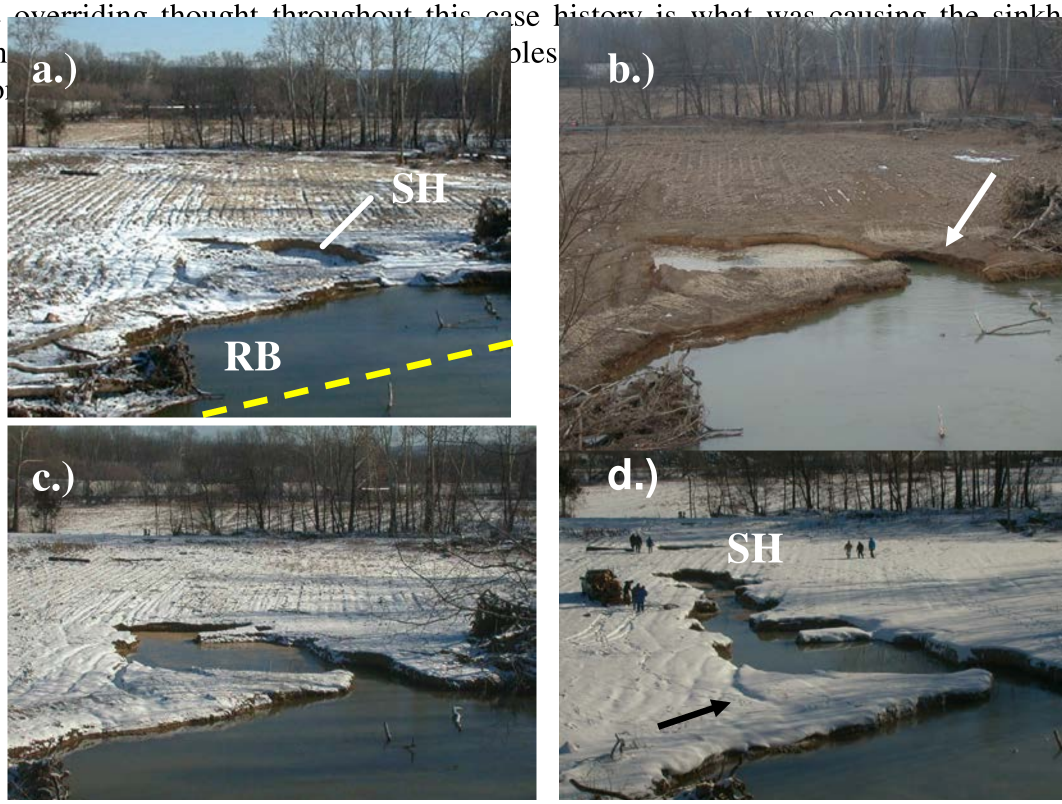

Figure 9. Photographic sequence showing the progradation of the Bushkill Creek across the field. a.) note the recessed bank (RB) and the nearby sinkhole (SH) in the field. b.) shows the breach in the bank (arrow) allowing stream water to enter the sinkhole. c.) shows the beginning of stream progradation across the field. d.) shows the stream almost to another sinkhole (SH) and SR 2017 (line at the top of SH letters). Also note in d.) the finger of stream bank (arrow) serving as a tenuous barrier. The outline of curving tension cracks can also be seen (arrow). All views are looking east.

One version is that:

1. The area is underlain by carbonate bedrock. Sinkholes occur in areas underlain by carbonate bedrock. It was just this areas turn on the big wheel.

Evidence from borings and geophysical surveys indicate that a deeply weathered zone exists in the vicinity of the SR 33 bridge. In addition, borehole data along the Bushkill Creek show depths to bedrock ranging from 2 to 15m indicating a pinnacled bedrock surface. For example, on the northbound segment of the 33 bridge, the depth to bedrock ranges from approximately 25 m on the south side of the creek to over 100 m on the north side of the creek (K. Petrasic, pers. com.). Sinkhole distribution is reflected in the regional attitude of bedding and joints.

Borehole camera views filmed by PA DEP showed a cave invertebrate in monitoring well 3 located on the south side of the Bushkill along SR 33 at depths of approximately 30 m below stream level. Core from a borehole on the north side of SR 33 depths of over 130 m showed rounded limestone gravels similar to pebbles observed in cave streams (pers. obs.).

Add in the fact that the Bushkill is a losing stream between 33 and 2017 it is quite apparent that the area is karstic. But what was the trigger?

Much of Pennsylvania was in the grips of drought during the period September 1995 through November 2002. The drought years were followed by a period of above-normal precipitation. What is noteworthy was that the drought lasted beyond the onset of sinkhole activity along the Bushkill. Additionally, high precipitation events due to Tropical Storms Dennis (5.5 cm/2.2 inches) and Floyd (16 cm/6.3 inches) in 1999 were also coincident with the onset of sinkhole activity.

Drought conditions can exacerbate sinkhole development. As the soil dries out, clays will shrink with desiccation and cracking of the soil would result in the development of more pathways for surficial water to enter the subsurface. In addition, drought conditions would have depressed the water table.

Depending on the cohesive properties of the regolith, infiltrating surface water can play an important role in promoting instability. Residual sediment typically has minimal interstitial cement holding the grains together and would have a low liquid limit. As water comes into contact with such loosely cemented material, cohesion is lost, allowing the regolith to erode at a much easier rate. This is commonly observed during drilling where fluids are often lost at the soil-bedrock interface. In another instance, the rise and fall of a potentiometric surface could also result in repetitive cycles of increasing and decreasing soil pore pressure. This in turn can increase the effective stresses between soil particles causing a decrease in soil cohesion and initiate soil piping which can ultimately create sinkholes.

Over time, dewatering and the lowering of the potentiometric surface (i.e., during a drought) creates a temporary base level as equilibrium is reached. Sinkhole activity is generally low during equilibrium phases. Sudden changes in this equilibrium such as what occurs during extreme swings in precipitation amounts (i.e., dry to wet times) can be the trigger that affects the hydraulic gradient to such a degree that sinkhole activity is high until the next plateau of equilibrium is reached.

or

2. Mining activity was concurrent with the onset of sinkhole activity. The affected area was compromised by the cone of depression developed by pumping during the mining process.

Sinkholes and limestone quarries are as acid mine drainage and coal mining; one often occurs with the other. Connections between quarry operations and sinkhole occurrence have been discussed in the literature (Foose 1953; Knight, 1970; Foose and Humphreville, 1979; Newton, 1987; Kochanov, 1999; Langer, 2001).

Dewatering or the removal of water from sediment commonly occurs during mining. The high pumping rates involved with removing groundwater from a mine can lower the water table and increase the zone of influence for kilometers. Changes in the potentiometric surface from pumping can affect hydrostatic pressure and in turn cause gradual or sudden removal of support for the land surface. As this support is removed, the land surface sags creating a depression and increasing potential for collapse.

A nearby quarry (approximately 1850 m west of southbound SR 33) has been in operation since 1919 primarily mining limestone and argillaceous limestone of the Jacksonburg Formation. On an average they pump 20-25 million gallons per day (mgd) ranging to 32 mgd during periods of high precipitation out of the active pit and return it to the Bushkill Creek (DEP, 2000). Hydrographs of the pumpage rates during the period of increased sinkhole activity indicate that there has been a steady increase in pumping over time (DEP, 2000). However, one would think that sediment-laden water would be observed in the water being pumped out of the quarry with each sinkhole collapse. This was not always the case and leaves room to speculate that sediment could be traveling in some other direction or simply that the finer sediment never made its way back to the quarry. The observation of cave invertebrates, cave-type of sediment observed in deep core returns and significant water loss between 33/2017 leaves the investigator with some degree of certainty that a subsurface conduit system of some undefined extent is present in the Bushkill-Brookwood-SR33 area.

Two other quarries (~ 2.7 km SW of SR 33 bridge) were also operating during the same time period. It was felt that the cone of depression for the three quarries overlapped at some point but the precise location of the combined cone of depression was indeterminate (S. Hill, DEP, pers. comm.).

Although no direct hydrologic connection has been determined quantitatively by means of a dye trace, it is generally assumed that there has been significant impact from the nearby mining activity and that the water table has been lowered significantly in the vicinity of the 33/2017 stretch. A brine tracer study was conducted in 2006 with some connectivity established to the quarry. However, the testing was not designed to provide a more regional perspective of groundwater flow.

Dewatering and the lowering of the potentiometric surface with an increase (or decrease) in pumping rates could create a temporary base level and somewhat artificial groundwater equilibrium. A disruption of this equilibrium by increased pumping could remove the hydraulic support of the land surface and cause sinkholes to occur.

or

3. The construction of Route 33 set the stage for sinkhole development at the 33 and 2017 bridges as well as the sinkholes between 33 and 2017.

It is interesting to note that the onset of sinkhole activity began in the area where the Bushkill channel had been modified through the construction of SR 33. It would appear that the change of the Bushkill channel had some influence on stream processes.

Straight banks are not the norm for any significant distance but can contain many of the channel features common to meandering streams (Ritter, 1986; Leopold and others, 1964). Typically straight reaches contain sediment that accumulates along alternating sides of the stream as alternate bars with the thalweg or deepest part of the channel migrating back and forth (Ritter, 1986). The sediment within the Bushkill channel is generally a poorly sorted mixture of sand and cobbles and the channel is reflective of the dynamics of the stream flow rate and volume. The base of the channel alternates with a series of shallow riffles and deep pools basically directing the flow of water from side to side of the channel as it flows towards the Delaware River (Kochanov, 2005).

Pools in the Bushkill appear to be directly linked to sinkhole development within the stream channel. Keller (1971) observed that that as discharge increases, the velocity in the pool approaches that of the riffle and from this he suggests that in bankfull discharge conditions, the velocity in the pool will exceed that of the riffle. During periods of high flow, the pools are scoured, with sediment being deposited on “high” reaches of the stream. These topographic highs correspond to places where bedrock is closer to the surface. As stream energy dissipates during the waning of a flood event, the coarser sediment falls out while the finer sediment is transported and deposited to the next pool.

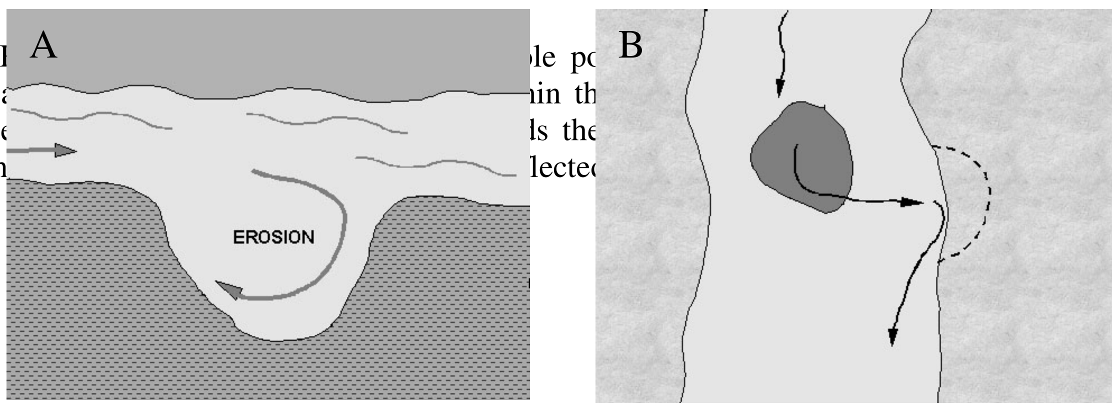

Figure 8. A sinkhole can serve as a deflector, forcing the water downward within the sinkhole pool (A) or directing water laterally towards the banks (B).

be more focused. Changes in sediment pore sizes, such as what would be encountered with the poorly sorted glacial sediment, can result in more turbulent groundwater flow and enhance erosion. As the banks are undercut, connections are made to voids and other subsurface drainageways and promote new sinkhole development ahead of the prograding stream (Figure 9).

Within the channel pool, the downward deflection of water may increase the size of the sinkhole or shift its location through erosion. In the case of the Bushkill reach between SR 33 and 2017, sinkholes within the channel appear to approach a certain maximum size, roughly 3 m. Sinkholes within the stream channel go through cycles of opening and filling in with sediment over time (Kochanov, 2005).

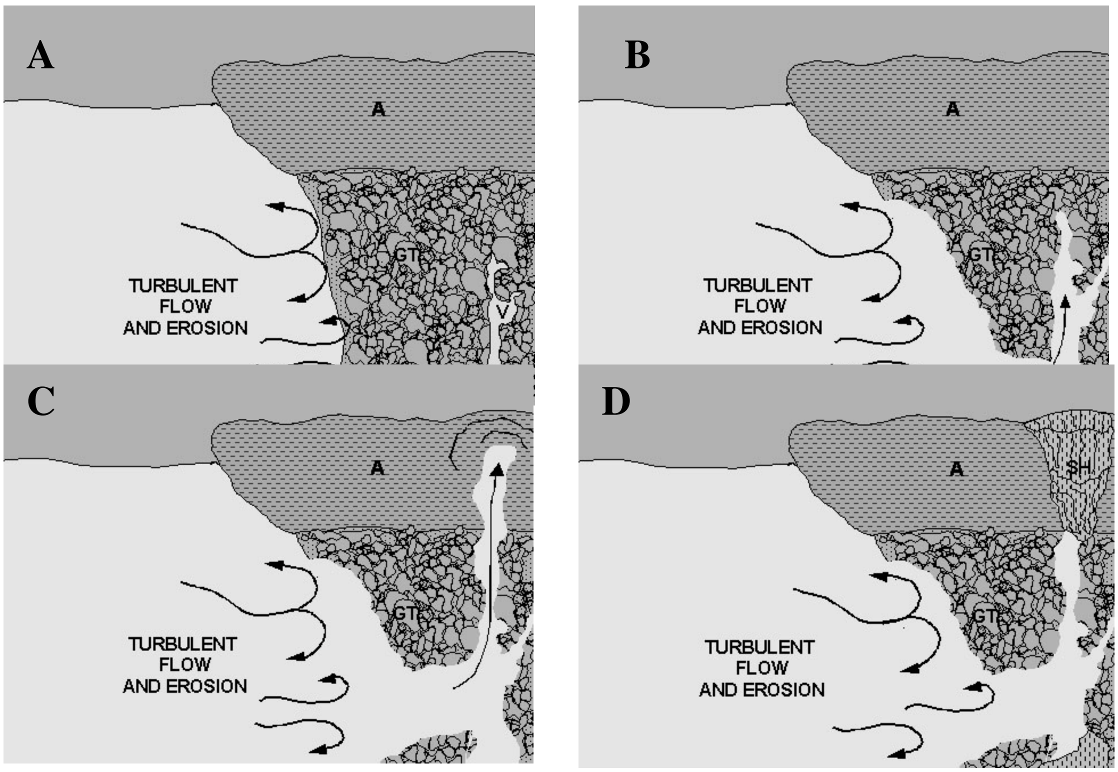

Figure 9. As stream water is deflected towards the bank and encounters poorly sorted glacial sediment (GT), more turbulent groundwater flow and enhanced erosion can occur. As the banks are undercut, connections are made to voids (V). Increased hydrostatic pressures (arrow)within pore spaces and within the voids can flush out sediment more easily and promote new sinkhole development.

Or

4. Nearby land development.

Changes in land use can often have a significant impact on sinkhole development. Based upon property records for Northampton County, there was concurrent residential development south of the SR 33 and 2017 bridges during the early 2000s. It is interesting to note that there were no sinkholes identified in housing developments outside of the Brookwood community during this period. Houses in Brookwood (Figure 10) were built during the late 1970s through the 1980s, well after the construction of SR 33 and its realignment of the Bushkill Creek. (see previous section and Figure 8).

Conversations with the residents of the Brookwood community stated that there were no sinkhole problems until the onset of sinkhole activity during the 1999-2006 period.

Figure 10. Location map showing the Brookwood Community, SR 33 and 2017, and the Bushkill Creek. North is up.

Sinkholes can be exacerbated by the failure of water-bearing utility lines. As a matter of record, water and sewer lines in the Brookwood community were compromised during June and October of 2006 (Figure 11). It was not determined whether these lines failed by some flaw in the composition or installation of the pipes or if they were affected by a longer ranging and more complex subsidence process. The occurrence of sinkholes within the stream channel and along the banks of the stream suggest that the stream was one major source of water to flush out the karstic drains and create sinkholes.

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Figure 11. Sinkholes along water and sewer lines in the Brookwood community.

geologists, industry, and all the while addressing the concerns of safety, private and public property, environmental impact, and economics.

One may ask why was there such a long period of time between the varied types of land use activity and sinkhole occurrence. The Stockertown case is not alone in this respect. The Borough of Macungie, Lehigh County, experienced a catastrophic collapse in 1987 where a large diameter sinkhole opened in a residential area. Aerial photographs were used to trace the history of the site back to the late 1940s (Figure 12). Photos indicated that a large sinkhole had been filled and was reactivated some 25 years later on the same site (Figure 13) (Kochanov, 1987c). This case established the fact that the subsidence process can take a long period of time to basically "set the stage" for the final collapse.

This case demonstrates the many variables that need to be considered when investigating subsidence problems. Conceptually, the karst system can best be related to a natural plumbing system where drains and pipes go through continual cycles of development and destruction over time.

Oftentimes the application of standard land development practices, developed in non-karstic regions, simply does not apply in karst areas. From the initial concept through the engineering and construction phases of a project, karst areas require special consideration, particularly in infrastruture design and utilization of natural resources.

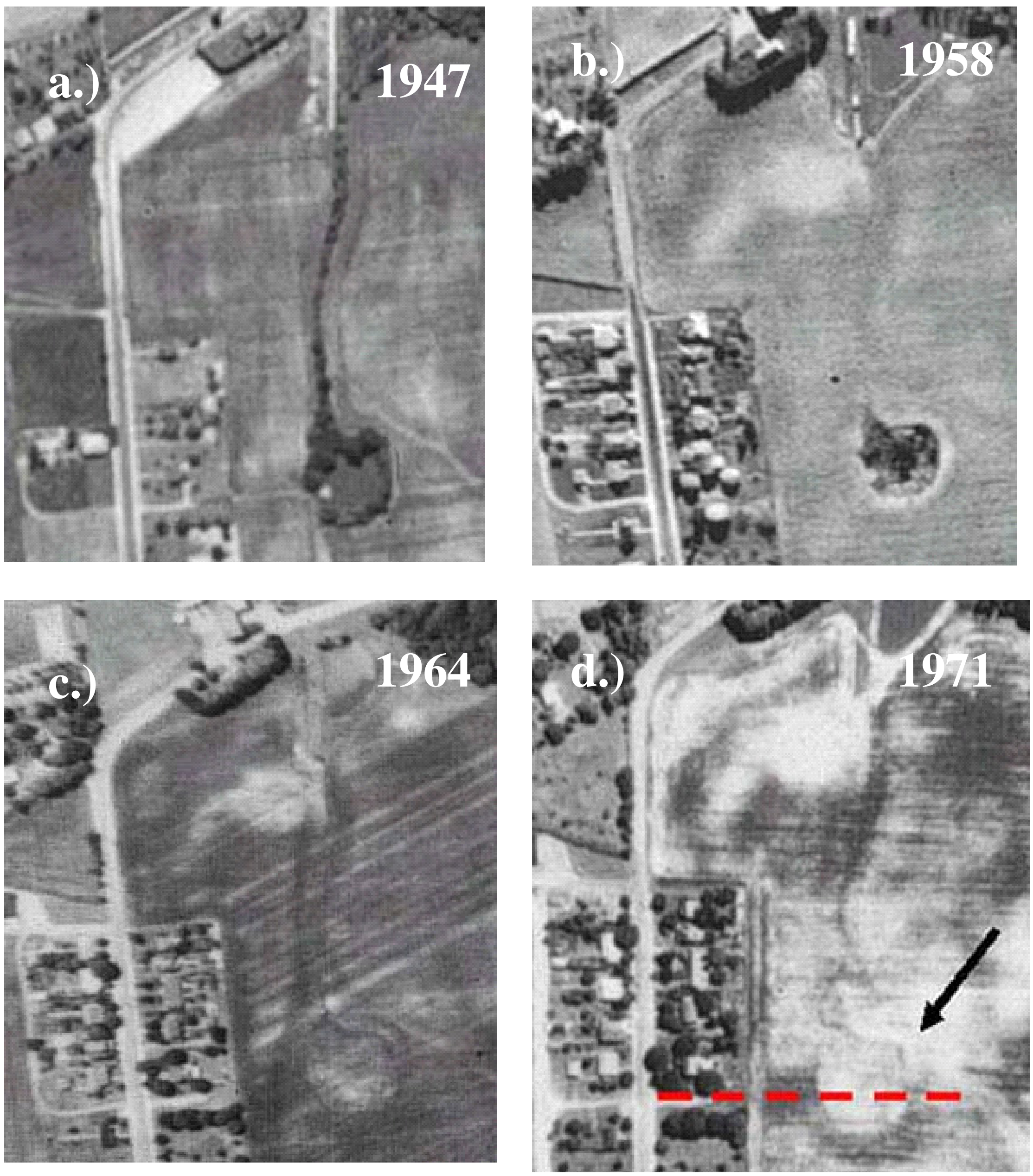

Figure 13. Macungie sinkhole, 1987.

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Stabilization of a Rock Cut with Ore-grade Uranium Mineralization

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Disclaimer

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ABSTRACT

During 1988, the New Hampshire Department of Transportation (NHDOT) remediated a 100-foot high rock slope located on Interstate 89 near Lake Sunapee in New London, New Hampshire (N.H.). The stabilization work was part of a larger roadway project and consisted of removing 100,000 cubic yards of potentially unstable rock, which was dangerously close to the northbound lane. This site was unique in that the granite rock contained anomalous concentrations of uranium and thorium.

In 1968-69, high concentrations of secondary uranium minerals were found in the rock during the initial excavation for this roadway cut. The uranium minerals generally occurred as fracture filings and coatings along east-west trending, steeply dipping joint surfaces. After construction, the highly soluble uranium minerals in the granite rock migrated downward when the water table dropped and were re-deposited at lower elevations in the rock cut.

Due to the naturally occurring radioactive minerals in the rock, a Health and Safety Plan was implemented to protect the construction workers and the general public. The Plan included sampling, testing, on-site monitoring and training of personnel involved in the construction phase of the project. This is the first time in the United States that health and safety precautions were implemented on an engineering project for the removal of uranium bearing rock.

The challenges were formidable which included presplitting the entire height of the rock slope in a single lift, locating and designing a disposal site for the uranium rich rock, and protecting the workers and public.

INTRODUCTION

Stabilization of an existing rock cut was completed in 1988-89 as part of a larger safety improvement project on Interstate 89 (I-89) near Lake Sunapee in New London, New Hampshire (Figure 1). The remedial rockwork included removal and disposal of 100,000 cubic yards of granite containing anomalous concentrations of uranium and thorium minerals. The site is a through cut measuring approximately 1600 feet in length and reaching a maximum height of 100 feet. The rock slope along the northbound barrel contained unstable blocks with weathered zones throughout. There was a narrow median strip of unstable rock, 30 to 40 feet in height, between the northbound and southbound barrels. The southbound barrel was lower in elevation than the northbound barrel. The ditch along the toe of the rock slope was narrow in width and not adequate for catching rock fall. Remediation consisted of removal of the median rock and cutting the northbound rock slope further back from the roadway to provide a rock fall catchment area. The challenges were formidable to include presplitting the one hundred (100) foot high northbound rock slope in a single lift, locating and designing a disposal site for the uranium rich rock, safely removing the rock while maintaining traffic, providing a long-term solution that minimizes the rock fall hazard, and protecting the workers and public. This site is unique in that it is the first time in the United States that health and safety precautions were implemented on an engineering project for the removal of uranium bearing rock.

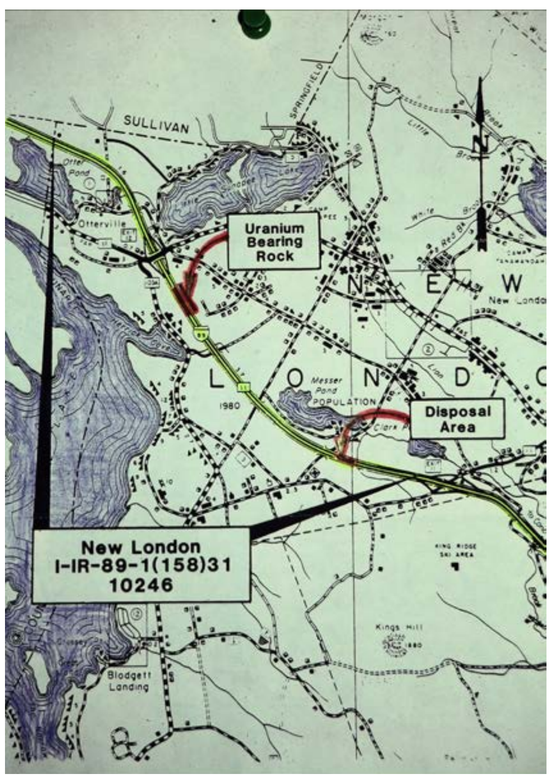

Figure 1 – Location Map

SITE CONDITIONS AND HISTORY

This rock cut was constructed in 1968-69 using production blasting techniques. The rock slope was ragged in appearance and had a blocky structure caused by numerous sets of intersecting joints. There was heavy seepage of water from the rock slope and surface water flowed continuously over the rock face in several areas (Figure 2). During the winter, much of the exposed rock slope along the east side of the northbound barrel was covered with a layer of ice. There were overhangs and unstable blocks throughout the rock cut. Rapid deterioration of the rock in some sections had resulted in weathered zones, which were undermining large blocks. Extensive over blasting during the original construction had shattered the rock face and opened existing planes of weakness (fractures, joints, etc.). This made the rock slope more susceptible to the infiltration of water, causing increased weathering and freeze-thaw action. This site had a history of rock fall, particularly along the northbound barrel.

The rock in the median area was criss-crossed by joints with variable spacing and fractures caused by the blasting during the original construction. Large blocks (3 - 10 feet in diameter) were precariously stacked on top of each other like a large stone wall. In 1982, selected scaling had removed several unstable blocks from the median area.

Figure 2 – Irregular Rock Slope with Jointing, Blocky Structure and Heavy Seepage of Water

STRUCTURAL GEOLOGY AND POTENTIAL INSTABILITIES

The rock is primarily a medium grained, massive to slightly foliated, two mica granite with intrusions of basalt. The granite has a pronounced tendency toward facial shattering and rapid superficial weathering upon exposure. A fault, striking northwest and dipping southwest, cuts across the median strip on the north end of the rock cut. A basalt dike, which has been cut by the fault, has been offset several feet. The jointing in the rock is well developed with most of the joints steeply dipping and a few nearly horizontal in orientation. Mapping identified several conjugate joint sets trending north-south/east-west and northwest-southeast/northeast-southwest. Most of the joints are discontinuous and short in length. Only a few joint surfaces are oriented within 20 degrees of the existing rock slope and dip toward the road. Therefore, the chance of a major plane failure along an existing joint surface is low. The rock fall hazards and instability at this site consist of unstable blocks, overhangs and fractured rock caused by a combination of intersecting joints and fractures, differential weathering and frost action. These hazards were further exacerbated by heavy seepage of water and the irregular slope profile, which provided launching pads for falling rocks.

RECOMMENDATIONS FOR REMEDIATION OF THE ROCK SLOPE

The rock on the east side of the northbound barrel was set back an additional 25 feet further from the road to provide an adequate rock fall catchment area. To minimize the quantity of rock excavation, to make effective use of the limited space available and to limit the overall impact of the proposed work, it was recommended that the rock slope be presplit on a one horizontal: eight vertical angle in a single lift (maximum height 100 feet). The NHDOT Standard Specification requires rock cuts deeper than 35 feet to be presplit in lifts with no lift less than ten feet in depth. Special drill bits and couplings were specified to minimize hole deviation and wander of drill rods. Unloaded relief holes, located between loaded presplit holes, were drilled to help promote fracturing along the proposed slope. A maximum deviation of two percent of the total drill hole depth in any direction was specified in the contract documents. The contractor utilized a laser transit to produce cross sections of the existing rock face accurate to within one foot. The information was used by the blaster in determining how much explosives to load in each hole and in tailoring the blast design for each shot. It was recommended that the strip of unstable rock in the median be completely removed.

Due to the limited work space, the large quantity of rock excavation and the high risk to the traveling public, it was recommended that the northbound barrel be closed for the duration of the rock removal operation. The southbound barrel would be divided in half with concrete barriers placed between the two travel lanes. Temporary crossovers would be constructed between the northbound and southbound barrels with the northbound traffic detoured along a three-mile long section of the southbound barrel.

URANIUM MINERALS IN THE GRANITE

Uranium ore was identified at this site in 1969, during the initial excavation of the rock cut. A rock with a lemon-yellow and grass green mineralization was found at the rock cut by a mineral collector who took the sample to the Geology Department at Dartmouth College for identification (Figure 3). A visit to the construction site by Dartmouth Professors Dr. Lyons and Dr. Boudette, confirmed that the granite contained uranium ore. Deposits of uranium minerals, 1.5 – 2.0 inches thick, had become concentrated and backfilled in cracks in the granite rock. Most of the excavated uranium bearing granite had already been buried in nearby roadway embankment fills.

Figure 3 – Lemon-yellow Uranium Mineralization

At the time, it was estimated that a small fortune in uranium ore may have been used in the construction of the interstate. Ten years later, Dr. Bothner, Professor at the University of New Hampshire, identified secondary uranium minerals as fracture fillings and coatings along steeply dipping joint surfaces during an investigation of the I-89 rock cut. The investigation was part of a study being conducted on uranium and thorium occurrences in New Hampshire *(1)*.

The granite at the site has been intruded by lamprophyre (basalt) dikes, which were exposed by the construction of the interstate roadway (Figure 4). These dikes are steeply dipping and cut diagonally across the roadway alignment with exposures on both sides of the rock cut and in the median strip. Dr. Boudette believes that the basalt dikes may have acted as a natural barrier to migration of uranium minerals dissolved in the groundwater moving through existing joints and fractures in the rock. He predicted that concentrated deposits of uranium ore would be found in the vicinity of the basalt dike with the highest concentrations on the north side of the dike *(2)*. The naturally occurring uranium minerals in the granite are highly soluble. The water table in the immediate area of the rock cut dropped 85 to 90 feet when the roadway cut was opened. The theory is that as the water table in the vicinity of the rock cut was lowered much of the uranium minerals in the granite rock adjacent to the exposed cut slopes mobilized, migrated downward and were re-deposited as secondary uranium minerals at lower elevations in the cut. Since, the proposed stabilization work would expose fresh granite surfaces along the northbound barrel and would remove the median rock, there was a chance that additional concentrations of uranium minerals could be encountered. Although, not expected to be a radiological hazard, there was concern for the dust that would be generated by drilling, blasting and excavation activities. Therefore, the NHDOT decided it would be prudent to undertake health and safety precautions for the construction workers and general public.

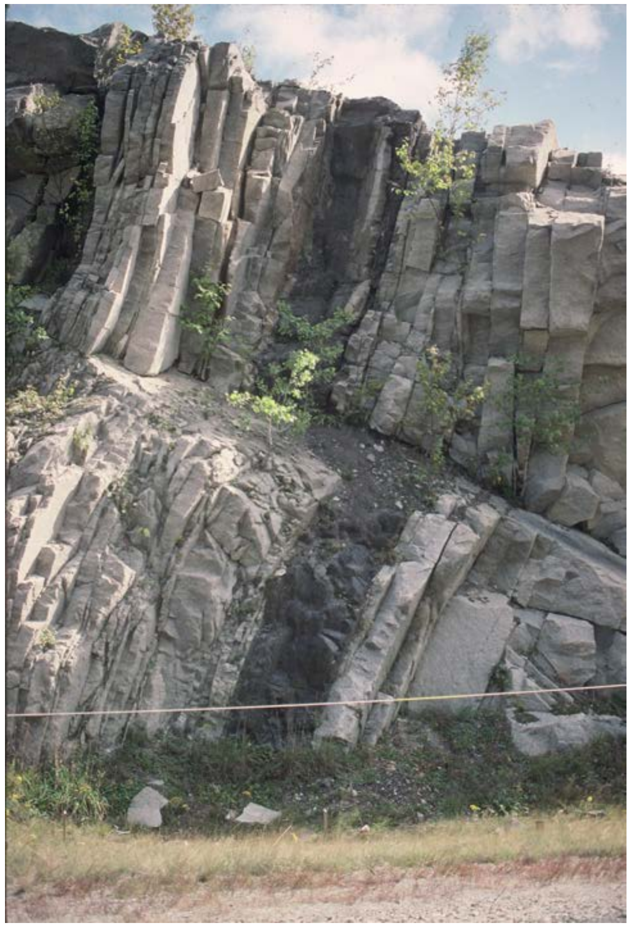

Figure 4 - Lamprophyre (basalt) Dike Exposed in Median Strip

Most of the surface drainage in the area empties into nearby bogs with organic deposits (peat) from the Holocene Epoch (Figure 5). Past studies indicate that organics have an affinity for absorbing dissolved uranium minerals. The peat deposits appear to have an unlimited capacity for filtering out the uranium minerals and depositing them as a coating on the organic fibers. Analysis of core samples from selected peat deposits in Vermont and New Hampshire have confirmed high concentrations of uranium minerals. It is speculated that dissolved uranium minerals from the surrounding granite rock have been accumulating naturally in the local organic deposits for thousands of years. As part of a study conducted in 1986 by the U.S. Geological Survey and NH Geological Survey, organic samples taken from a bog located adjacent to Messer Pond and approximately one mile southeast of the rock cut showed high concentrations of uranium *(3)*.

Figure 5 – Nearby Peat Bog

DESIGN OF THE DISPOSAL SITE FOR THE URANIUM RICH ROCK

As a precaution the excavated granite was placed at a site located two miles south of the rock cut. The following characteristics made this a favorable disposal site for isolation of the uranium rich rock:

- Close proximity to the rock cut
- State owned property between existing embankment fills for the I-89 northbound and southbound barrels
- Underlying soil is a dense glacial till
- Surface water drains into a nearby peat bog, filtering out and trapping any dissolved uranium minerals in the organics.

All vegetation to include stumps and topsoil were removed in the disposal area. Fill was placed over the disposal site to an elevation of at least five feet above the seasonal high water table. The rock excavation was placed in four foot thick layers and chinked with smaller rock sizes to eliminate large voids. The area was capped with an impervious soil to keep surface water from contacting the buried rock fill. A 12-inch thick drainage layer of sand was placed between the imperious cap and a layer of topsoil to expedite lateral movement of the surface water (Figures 6, 7 & 8). Only grass was allowed to grow at the disposal site to eliminate potential roots from bushes or trees penetrating the imperious soil cap.

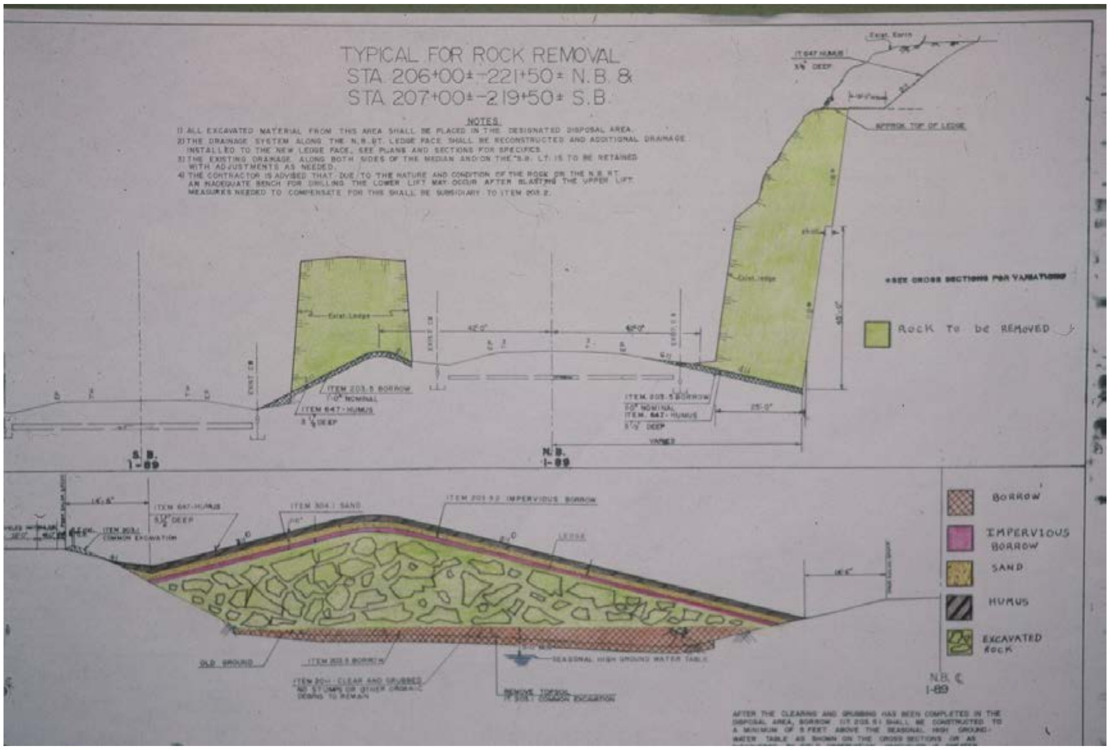

Figure 6 – Cross Section of Disposal Site

Figure 7 – Construction of Disposal Site

Figure 8 – Completed Disposal Site

Three monitoring wells were installed between the rock disposal site and a network of organic bogs to the northwest. Groundwater samples from the wells were tested before, during and after the construction phase to determine if uranium minerals were migrating from the disposal site. The wells extended to varying depths with the well screen in each monitoring well installed within a different sampling zone. One of the wells sampled water from the overlying glacial till, another from the bedrock and the third from a transition zone within both the rock and till. Initial groundwater samples with baseline readings were conducted at each well prior to the utilization of the disposal site. The recovered water samples were tested for screen alpha, uranium, radium 226, radon gas, PH and specific conductance. The range of test results from the groundwater samples collected from the wells over a two year period are listed in Table 1 below.

Table 1 – Results from testing water samples

Type of Test	Well #1 (Initial Reading)	Well #2 (Initial reading)	Well #3 (Initial reading)	Well #4 (Initial reading)
Screen Alpha (pCi/L)	20 (20)	4 – 45 (16)	1 – 19 (7)	1 – 17 (2)
Uranium (pCi/L)	9 (9)	4 – 8 (2)	3 – 18 (deleted)	1 – 2 (deleted)
Radium (pCi/L)	5.7 (5.7)	3.5 - 10.2 (3.8)	0.4 – 1.8 (1.8)	0.5 – 4.4 (deleted)
Radon Gas (pCi/L)	23,800 (23,800)	2,400 – 4,200 (2,800)	1,400 – 8,400 (1,700)	2,100 – 10, 200 (9,300)
PH	6.1 (6.1)	4.3 – 5.8 (5.1)	5.3 – 6.1 (5.8)	4.8 – 5.5 (5.5)
Specific Conductance (UNMHOs)	385 (385)	142 – 1430 (546)	50 – 1460 (1460)	51 – 797 (163.7)

Table Notes: Well #1 was destroyed during construction and replaced with well #4
Well #1 and Well #4 (Five foot screen in bedrock)
Well #2 (Five foot screen in glacial till)
Well #3 (Ten foot screen, five feet in bedrock,five 5 feet in glacial till)

Definition of terms and units of measurement:

- Alpha is a particle that is emitted from the nucleus of certain radioactive isotopes in the process of decay.
- Uranium is a heavy, silvery white, metallic element, which is radioactive and toxic. The isotope U 238 is the most abundant in nature *(4) (5).*
- Radium occurs naturally in the environment as a decay product of uranium and thorium *(6).* It is soluble in water and commonly found in low levels in nearly all rock, soil and water. Radium-226, its most stable isotope, has a half-life of 1602 years and decays into radon gas *(7).*
- Radon is a radioactive gas produced during the natural decay process of uranium. It is a dense, colorless, chemically unreactive inert gas, which you can not see, smell or taste *(8).* In water, radon is measured in picocuries per liter (pCi/L). Ten thousand pCi/L in water is equivalent to approximately 1 pCi/L in air. The U.S. Environmental Protection Agency (EPA) considers airborne radon levels of 4 pCi/L or higher to be a concern. Although at present there are no EPA standards for radon in water, a maximum contaminant level of 300 pCi/L has been suggested *(9).*
- Pico Curie per liter (pCi/L) is a unit of radioactivity corresponding to one decay every 27 seconds in a volume of one liter, or 0.037 decays per second in every liter of air *(10).*
- Specific Conductance is a measure of how well water can conduct an electrical current and can be used as an indirect measure of the presence of dissolved solids. UNMHOs is a measure of the resistance in units of ohms *(11).*

Concentrations of radon in groundwater from areas in the New England region with similar bedrock types typically range between 50,000 and 100,000 pCi/L. Initial readings from bedrock well #1 showed radon levels of 23,000 pCi/L. Other than the higher radon gas levels in the bedrock, the other test results showed no discernible trends or significantly increased levels of contaminants that would raise concerns.

PRECAUTIONS DURING CONSTRUCTION

The Division of Public Health Services of the Department of Health and Human Services (DPHS) regulates radioactive materials in the state of New Hampshire. The uranium ore in the granite was being excavated and disposed of as a raw material in an unchanged state. Therefore, the rockwork activities were exempt from permit or regulation. Due to the past history of this rock cut, the New Hampshire Department of Transportation (NHDOT) decided it would be prudent to take precautions during the construction phase of the project. Special Provisions were written by the NHDOT, the DPHS and the FHWA (Federal Highway Administration) outlining requirements to be followed by the contractor during the excavation and disposal of uranium rich rock. The NHDOT required the contractor to develop and implement a "Health and Safety Plan" to protect the public and workers at the construction site during all phases of the project *(12)*. The plan was to include the following:

- Methods for sampling, testing and monitoring of the rock and air during the rock removal disposal operations
- Methods to measure and track exposure rates, to assess risk, to protect the workers and the public
- Radioactivity training for the workers
- Decontamination procedures for personnel and equipment

The procedures to accomplish the requirements outlined in the Health and Safety Plan included the following:

- All workers (Contractor, NHDOT and other government officials) would attend a mandatory training session on radiation and the use of a respirator. Each attendee was required to pass a written examination.
- All workers were required to undergo an initial physical exam to include periodic analysis of urine samples for radionuclides.
- The Contractor would maintain controlled access to the construction site with security guards and an access control book. Radation caution signs would be posted throughout the project area.
- All workers would be issued radiation work permits, which specified the minimum radiological safety requirements for their particular job duties.
- All drill rigs would be equipped with dust suppression equipment consisting of vacuums and water mist/detergent foam systems.
- Protective clothing and respirators would be available (Figure 9).
- The radiation exposure of workers was monitored and tracked with individual monitoring devices (Thermal luminescent dosimeter badges and pocket dosimeters) to be worn on the person at all times while in a radiologically controlled area. The individual sampling devices were worn by the loader operators and the drillers. An exposure history file was maintained on all workers. The workers would be periodically frisked with a probe to detect contamination on clothing.

Figure 9 – Drillers Wearing Protective Clothing

A baseline survey was conducted prior to beginning the work to establish background radiation levels. Dr. Bothner's 1978 data was used as a reference point in establishing the baseline. The initial survey measured radiation levels at the rock surface with a gamma scintillator, sampled the air at the excavation/disposal sites and tested sediments collected from existing drainage catch basins. The initial data was later supplemented with in-situ testing conducted during the construction phase. Testing during the construction work consisted of the following:

- Continuous sampling and testing of dust during the drilling & blasting operation and the handling of the excavated rock at the disposal site
- Drill hole surveys were conducted with a scintillation detector lowered into the drill hole to scan levels of radioactivity. Every presplit hole and one production hole within a 16-foot square grid were surveyed. Readings were taken at one foot intervals along the entire length of the drill hole. The results were used to determine the protective equipment and level of monitoring to be used when that section of the rock cut was excavated and the rock was placed in the disposal site.
- Smear tests of the drill tailings off the drill bit and from dust collected by the vacuums on the drill rigs
- Pancake probes to monitor the blasted rock rubble and to frisk workers at the site
- Visual mineral inspections, air sampling, rubble surveys for uranium concentrations were conducted after each blast, before allowing workers to return to the blast site.

A variety of instruments were used to conduct the monitoring and testing of loose contaminants, to measure airborne radioactive contaminants and to track exposure of workers and dose rates. The instruments and their purpose are listed below:

- Thermal luminescent dosimeter badges and pocket dosimeters (Figure 10)
- Loose and fixed contaminants – Ludlum 177 with thin window gm pancake probe (Figure 11)
- Loose and fixed alpha contamination – Ludlum 177 with 59 cm² ZnS scintillation probe
- Drill hole monitoring – NMC GA-6 well logging scintillator (Figure 12)
- Smear tests and air filters – Ludlum 2000 scales with ZnS scintillator and thin window pancake gm detector (Figure 13)
- Low volume air sampling (within 50 feet of the drill rig) – Eberline RAS-1 air sampler and Gillian/Dupont personal lapel air samplers
- High volume air sampling (within one foot of the drill rig) – Radeco/Staplex air sampler (Figure 14)
- Beta exposure – Eberline RO-3C air ionization chamber (Figure 15)
- High level dose rates – Ludlum 14-C with hot dog gm probe
- Low level dose rates – Eberline PRM-6 with thin crystal sodium iodide (NaI) detector
- Dose rate – Automess teletector

Figure 10 – Individual Monitoring Devices

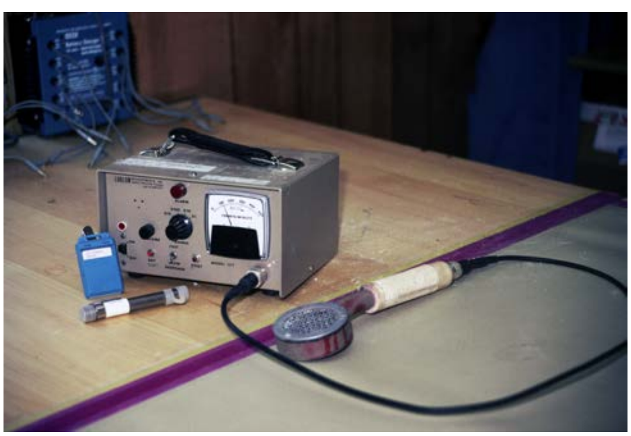

Figure 11 – Ludlum Pancake Probe

Figure 12 - Well Logging Scintillator

Figure 13 - Ludlum 2000 scales with ZnS Scintillator and ThinWindow Pancake gm Detector

Figure 14 – Air Sampler

Figure 15 - Eberline RO-3C Air Ionization Chamber

NATURAL RADIOACTIVITY VERSES MAN MADE SOURCES

The subject of radioactivity evokes a sense of foreboding and concern in most people. The reality is that radioactivity occurs naturally throughout our environment. Radionuclides (radioactive elements or isotopes) are found in the air, water, soil and even in the human body. Radoactivity is common in the rocks and soil that make up our planet, in the water and oceans, and in the building materials used to construct our homes *(13)*. Therefore, it is important to understand the radiation levels to which the average person is exposed from both natural and manmade sources. These levels of exposure can be used as a benchmark to compare with exposure rates measured at the project site. Experts in the field estimate that the average person in the United States is exposed to about 300 to 360 milli-rem per year from natural background sources and approximately 50 milli-rem per year from "artificially produced" sources such as medical x-rays. Background radiation exposure can vary significantly from location to location depending on elevation, soil, rock and latitude. For example radiation exposure from cosmic rays increases with altitude, approximately doubling every 6,000 feet *(14) (15)*. The following are estimated average radiation doses from common activities (Table 2):

Table 2 - Estimated average radiation doses from common activities	
Activity	**Typical Dose**
Smoking	280 milli-rem/year *(16)*
Dental x-ray	10 milli-rem per x-ray *(16)*
Chest x-ray	8 milli-rem per x-ray *(16)*
Cross country round trip by air	5 milli-rem per trip *(16)*
Convential Coronary Angiogram	560 milli-rem per test *(15)*
Ct Body Scan (Chest/abdomen/pelvis)	1,300 – 1,800 mill-rem per test *(15)*

Radon is a radioactive, odorless, colorless gas that results from the radioactive decay of uranium. Studies have shown that rocks with high uranium content often contribute to high levels of radon. Uranium oxide minerals typically found in deposits of two-mica granite, similar to the bedrock at the New London rock cut, are easily leached and readily transported by ground water. Transport of these soluble uranium oxides can led to a secondary enrichment of uranium which can contribute to local high levels of radon. Uranium deposits in New England are known to occur along fissures, unconformities, faults and shatter zones. Dr. Bother's 1978 study reported secondary uranium minerals as fracture fillings and coatings along East-West trending, near vertical surfaces at the site of the New London rock cut. The most abundant secondary uranium mineralization detected at the rock cut was within a fracture zone of closely spaced joints that was intruded by two basalt (lamprophyre) dikes.

The terms and units of measurement utilized in describing radioactivity and exposure rates can be confusing. The disintegration of an unstable atom results in particles being emitted (radioactivity). There are three types of radiation to include alpha, beta and gamma particles. Alpha particles can only travel a few centimeters in the air and can't penetrate the outer layer of skin. Beta particles can travel up to approximately 35 feet in the air and penetrate a few millimeters into living tissue. Gamma particles have a range of one mile in the air and can penetrate deep into living tissue. All three particles can be a problem if ingested or inhaled *(17).* Roentgen Equivalent Man (Rem) is a unit of radiation dose and how it affects living tissue. Dosages are commonly measured as millirem (mrem), which is one thousandth of a rem. Rad (Radiation Absorbed Dose) is a measurement of gamma and beta exposure, which is commonly measured as millirad (mrad) or one thousandth of a rad *(18).* A term typically used in reporting and measuring radiation contamination over an area is disintegrations per minute (DPM) *(19).* The maximum allowed concentration or level of exposure for a particular substance, which has been set by a regulatory agency is expressed as maximum permissible content (MPC).

MONITORING RESULTS

Extensive monitoring was conducted during the construction phase to include testing of loose contaminants, logging of selected drill holes, measuring airborne radioactive contaminants and tracking exposure of workers to radioactivity. In addition, initial background radiation readings were taken prior to any rock work at the site. These readings ranged from 0.02 - 0 .03 mRem/hr as compared with Dr. Bother's 1978 readings of 0.55 mRem/hr in contact with the rock. Air samples collected at the construction site and in areas occupied by the traveling public showed no detectable radioactive airborne contamination. Testing of loose contaminants in the drill tailings and the drill hole logging showed no detectable radioactivity. Smear samples taken directly from yellow cake deposits of uranium minerals on the rock surface resulted in loose contamination readings for combined alpha and beta activity of 20,000 DPM/100cm^2. The measured external exposure rates for the project are listed below:

- Areas of known contamination - 0.05 mRem/hr
- Disposal Site - 003 to 0.05 mRem/hr
- Top of rock cut - 0.012 to 0.020 mRem/hr
- Equipment Operators - 0.015 to 0.02 mRem/hr
- Traveling Public -0.01 to 0.015 mRem/hr
- Highest readings of contact exposure on pieces of contaminated rock – 0.1 to 0.5 mRem/hr of gamma exposure and 10 - 20 mRad/hr of beta exposure

The Nuclear Regulatory Commission has set a limit for whole body exposure of 1.25 Rem (1,250 mRem) for a three month period and for internal exposure (intake of radionuclides) of 520 MPC for a three month period. Based on Dr. Bothner's 1978 original readings a person working within two feet of the rock over a period of one week at a 0.02 mRem/hr. ambient dose rate would receive a whole body exposure of 1.5 mRem. Therefore, the total exposure per worker at the project site over a four month period (time to complete the rockwork) is estimated to be 24 mRem (1.5 mRem per week X 16 weeks). Continuous direct contact with the rock during the rock excavation phase would result in a maximum exposure of only 660 mRem. These exposure rates are well below the limits set by the Nuclear Regulatory Commission.

SUMMARY

The stabilization work at the rock cut was accomplished without negative environmental impacts, without any hazard to the traveling public, with measured radiation exposure to the workers well below established NRC limits and with minor disruption to traffic. Air samples taken throughout the project site during all phases of the work showed no detectible amounts of radioactive airborne contaminants. Loose contaminants readings and external exposure rates were very low. It is suspected that most of the original deposits of uranium minerals in the vicinity of the rock cut went into solution, migrated downward below the bottom of the cut shortly after the original excavation or were carried away by the flow of both surface and groundwater. The uranium minerals may have leached from the granite over time resulting in lower than expected concentrations when fresh rock surfaces were exposed during the stabilization work.

The newly constructed rock slope along the east side of the northbound barrel was uniform, stable and without overhangs (Figures 16 and 17). The half casts from the presplit drill holes were evenly spaced throughout the rock slope with the deviation in the holes

Figure 16 – Stable Slope

Figure 17 – Presplit Holes Evenly Spaced with Less than 2 Percent Deviation

maintained at less than two percent of the total drill hole length. The new widened ditch along the toe of the eastern rock face on the northbound barrel provided a sufficient catchment area for future rock fall. The overall condition of the stabilized northbound rock slope was excellent.

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US Route 15 in New York and Pennsylvania:
A Joint Effort in Rockslope Construction

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Disclaimer

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ABSTRACT

The U. S. Route 15 Improvement Project in Tioga County, PA and Steuben County, NY is a joint effort between the New York and Pennsylvania Department of Transportations (DOT's) to upgrade the highway to a four-lane limited-access highway. The highway will be on a new alignment along the western side of the Tioga River Valley, approximately 12 miles from the interchange with PA Route 287, and will connect to the existing four-lane section of Route 15 in Presho, New York.

Approximately 750,000 cubic yards (CY) of fill is needed for the construction of a new bridge over the Cowanesque River near the Pennsylvania-New York border. A large new side hill through-cut in New York will provide 370,000 CY's of the material needed for the new bridge. New York State DOT (NYSDOT) designed the rock cuts and Pennsylvania DOT (PennDOT), incorporated the rockslopes into their construction project. This paper will describe some of the issues with construction of a rock slope in New York using NYSDOT specifications within a PennDOT construction contract. The States will share responsibilities for inspection of slope construction with NYSDOT responsible for the final acceptance and maintenance of the rockslopes.

BACKGROUND

In 1991, ISTEA, the Intermodal Surface Transportation Efficiency Act, was enacted. One of the “high priotity transportation corridors” it listed was “Corridor 9”. It ran along US 220, from Bedford to Williamsport PA, and then north on US 15 to Corning, NY. In 1995, the National Highway Designation Act designated Corridor 9 as Interstate Route I-99, also known as the Bud Shuster Highway or Appalachian Thruway. The section of U.S. Route 15 between the Pennsylvania state line and Presho, New York, will be upgraded to interstate standards and link I-99 with I-86 in Painted Post, New York.*(1)* The interchange of U.S. 15 and I-86 is currently under construction and being upgraded to meet interstate standards. *(2)*

The Route 15 corridor has been recognized by New York and Pennsylvania to be critical to the NAFTA (North American Free Trade Agreement) associated traffic. Since 1995, PENNDOT and NYSDOT have worked with the engineering consultant Dewberry Inc. (formerly Goodkind and O’day) to examine alternative roadway alignments and designs to meet the area's transportation needs, while avoiding, minimizing or mitigating impacts on important natural and cultural resources. Working together for seven years, the two states developed a Final Environmental Impact Statement (FEIS), which was released for public review in April 2002. Following a public comment and review period, the FHWA accepted the preferred alignment (Alternative C-3-C) leading to the Record of Decision in August, 2002. The selected alternative includes an interchange with PA Route 49, west of Lawrenceville, and a Tourist Information and Rest Area on the northbound New York side of Route 15. Following the approval of the Federal Highway Administration (FHWA), the two state agencies worked separately and in coordination with each other to develop their independent roadway designs.

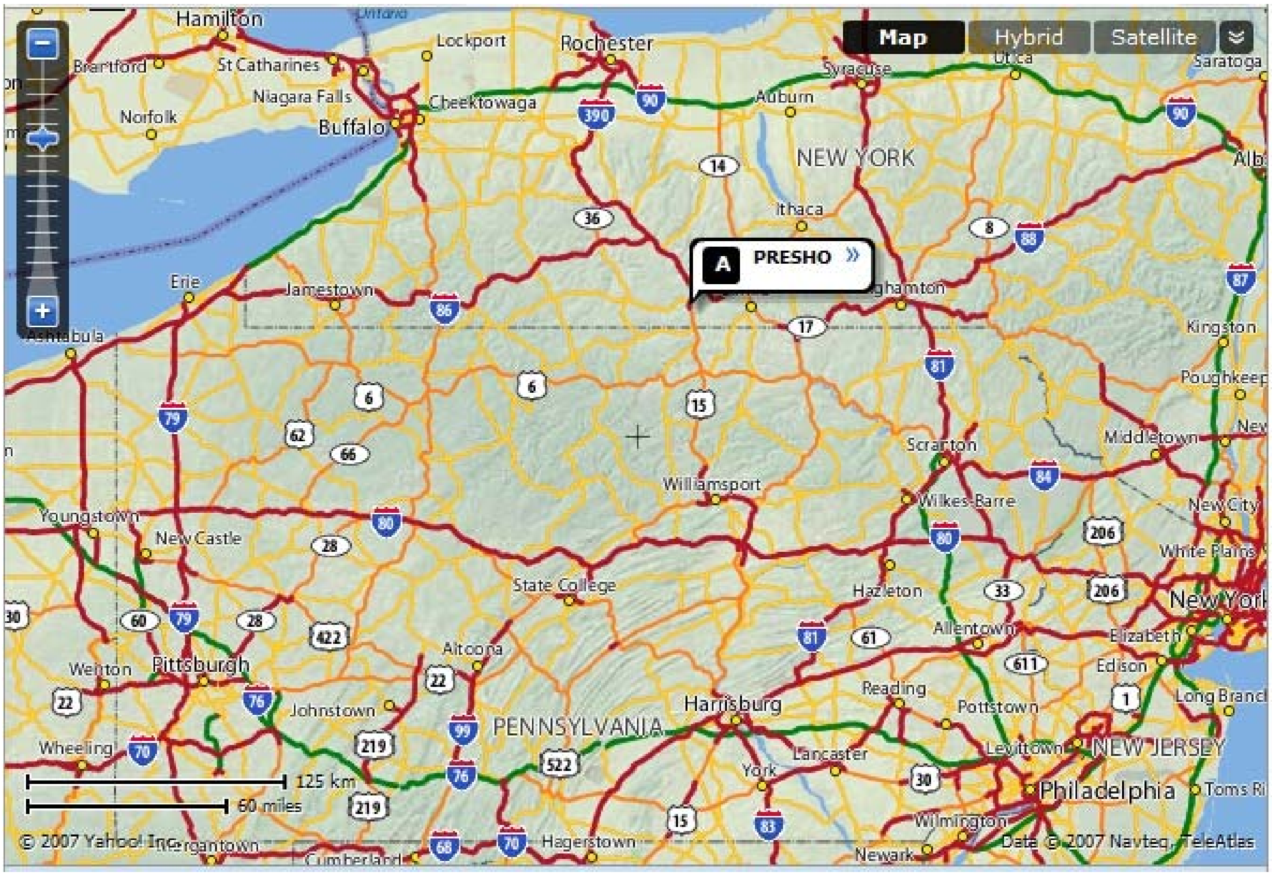

Figure 1- Map of PA and NY showing existing I-99 and Route 15 to Presho, NY. Future I-99 will pass through State College and Williamsport and connect with I-390 to Rochester, NY. *(3)*

ROUTE 15 - AGREEMENT (No. 032146)

The Agreement stated that: *"each State will independently design and build its own projects, but of necessity will periodically encroach into the other state during their construction and maintenance activities"…PENNSYLVANIA will remove enough material from NEW YORK to construct approximately 370,000 cubic yards of completed embankment from the northern edge of the Cowanesque River, northward… to limits agreed upon at NY metric station 10 +800, approximately 305 meters (1000-feet) from the State border. NEW YORK will provide PENNSYLVANIA with details that will allow PENNSYLVANIA to design a set of construction plans for the area north of the State border within the grading limits. "To the extent applicable, during the performance of this Agreement each part will require its contractor(s) to comply with Standard Clauses for all New York State Contracts… for all work performed in NEW YORK."* This agreement was signed by the Transportation Commissioner, Chief Counsel, Attorney General, and Governor of each State by October 20, 2004.*(4)*

As was stated in the agreement New York will provide Pennsylvania with the necessary details that will be used to design a set of construction plans. The Pennsylvania project plans would use NYSDOT specifications and pay items for the excavation in New York. One obvious problem, as can be deduced from the agreement, was that New York contracts uses the metric system (SI) and Pennsylvania uses US customary units in their projects. (New York is currently preparing to switch back to US customary). Other major issues in providing Pennsylvania a set of usable plans are that the Specifications and the pay items are different.

GEOLOGY

The new alignment for Route 15 will cross the Cowanesque River which is on the Pennsylvania side of the border. In glacial times, the area was covered by Glacial Lake Cowanesque and the subsequent release of rushing waters created the nearby Pennsylvania Grand Canyon in Wellsboro and the periodic glacial advances created broad river valleys. The hill on the New York side of the Route 15 project, which the new alignment cuts through, had been glacially sculpted and deposited with up to 50 feet of till. Bedrock is mapped as the Wiscoy member of the Java Group and consists of horizontally bedded Devonian sandstones, siltstones and shales.

SUBSURFACE INVESTIGATION

The New York State Department of Transportation’s Geotechnical Engineering Bureau (GEB) drilled the first exploratory drill holes in 1998 and continued drilling through 2001. A total of 89 drill holes were drilled to an average depth of 39 feet over the entire Route 15 relocation project limits. Wire line NQ core barrel was drilled with CME 45 drill rigs. An automatic hammer was used to progress some of the holes. A total of 35 holes were drilled on the hillside and 19 holes went into bedrock with excellent rock core recovery. However, drilling conditions on the slope proved difficult due to steep slopes and the bouldery till overburden. During the drilling an Engineering Geologist from the GEB assisted the Regional drill crews in determining top of rock. These drill holes were logged and entered into BLAP, which is the GEB’s Boring Log Automation Project database. To supplement the drill holes a total of 131 seismic points were shot by the Engineering Geology Section of the GEB between November 1998 and July 2003. As you can see from the map, one alignment alternative was proposed along the other side of the Tioga River Valley.

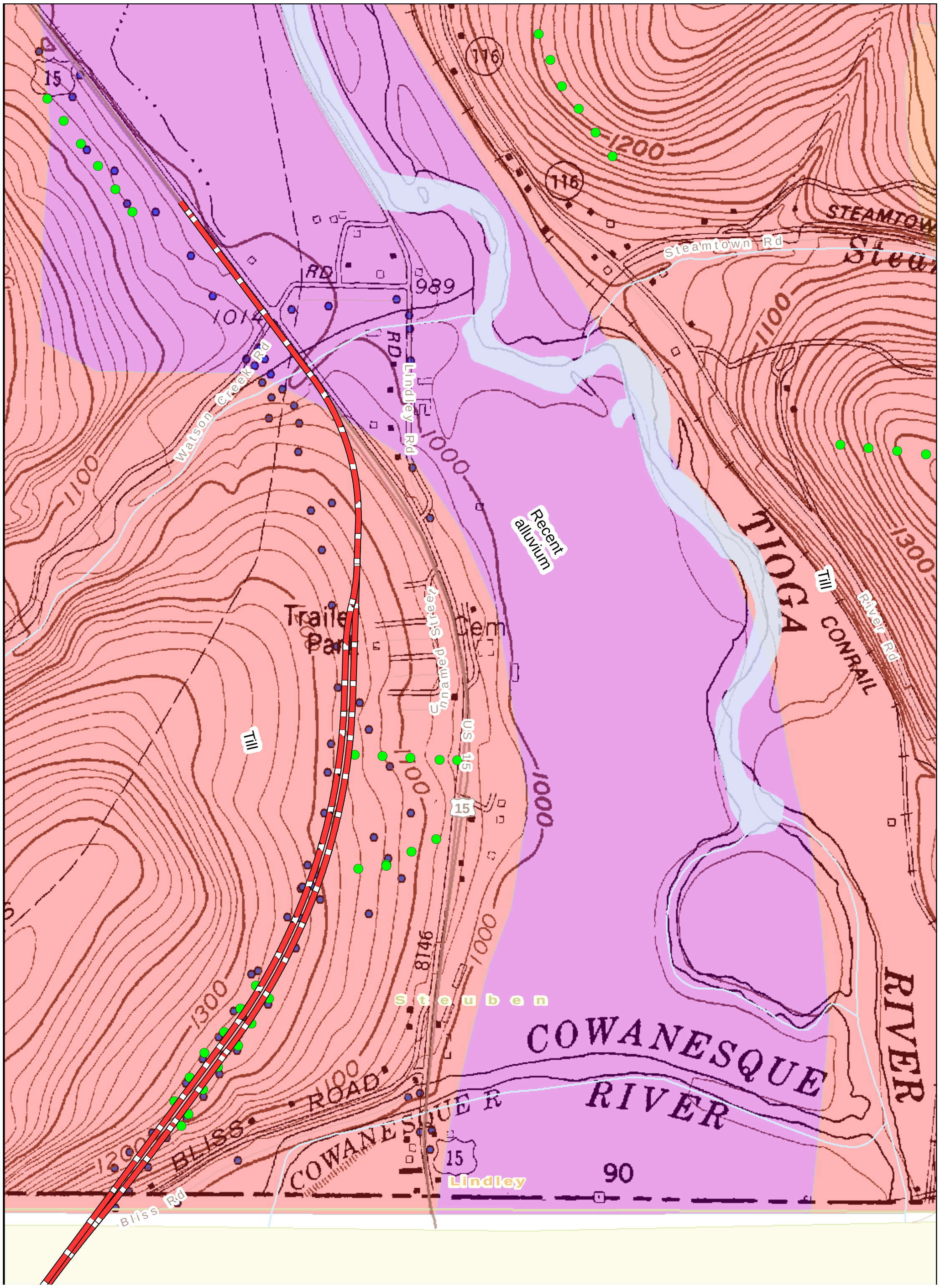

Figure 2 - Map of new alignment (Red). Drill holes are in blue and seismic points are green. Total length of the realignment shown is 1.6 miles. Pennsylvania will remove rock along the first 1000 linear feet. (Adjacent to the 1300 foot contour label).

PRELIMINARY DESIGN

The first request for rockslope designs for the throughcut were received by the Engineering Geology Section of GEB in October, 2001. The original rockslope design recommendations were based upon examination of nearby rock cuts and the evaluation of rock cores, including one 30 meter deep boring taken at the expected maximum slope height. The rock cores showed a weathered top of rock surface, some clay seams and steeply dipping fractures. A 3 vertical on 2 horizontal presplit rockslope was designed for both rockslopes in the throughcut. “Ritchie Ditch” criteria were used to establish the ditch design width of 6.7m (22 ft.) wide x 2.4m (8 ft.) depth for the estimated final rock slope height of 40 meters (131 feet) southbound slope. The Northbound slope estimated slope height of 9 meters required a 4.5 m (15 ft.) width and a 1.5m (5 ft.) depth. *(5)*

FINAL ROCKSLOPE DESIGN RECOMMENDATIONS

Due to the delay in letting the project and the completion of the FHWA’s Rockfall Catchment Area Design Guide Final Report SPR-3(032) in November 2001, revised ditch recommendations were supplied by Engineering Geology in accordance with the Report’s guidelines. *(6)* This consisted of a 9m (30 ft.) southbound and a 3.5m (11.5 ft.) northbound ditch width with a 1 vertical on 4 horizontal ditch slope.

The quantity of rock to satisfy the PennDOT bridge fill requirements of 375,000 CY’s was estimated to be attained over a length of 1000 linear feet of roadway and slope excavation. This is approximately one third of the way through the rockcut. The contract plans show a vertical payline at the north end of the slopes. The remainders of the rockslopes are to be finished in a separate NYSDOT contract that is now under construction. Geologists were brought on late into the design phase of the project after the project limits had already been established. The preferred design would have been construction of the entire first lift of the rockcut to a depth determined by the quantity of fill required. This would leave a flat continuous working bench to finish the rockcuts in the next construction contract.

Figure 3 - View from the south end of the southbound rock cut looking toward Pennsylvania. The new Cowanesque River Bridge is being built with fill from the rockcut.

CONSTRUCTION

The PennDOT construction project (SR 6015 Job#6895) began in Pennsylvania in June 2005. Months prior to beginning any work in New York, the Project Engineer-in-Charge (EIC) and contractor reviewed the specifications and plans for the rockcuts. He e-mailed his questions to the Regional Design Engineer in Hornell, New York. Geotechnical questions were forwarded to the Regional Geotechnical Engineer and geologic questions were forwarded to the Engineering Geology Section of GEB in Albany. Geologists responded by e-mail and copied all interested parties. This method worked reasonably well, keeping all informed, creating a paper trail of responses, and getting timely answers to the project. After the contractor began working in New York geologists visited the project and communicated directly with the project personnel.

A meeting was held on July15, 2006 to review the New York State Department of Transportation's blasting specifications. Present at the meeting were representatives from PENNDOT, FHWA, NYSDOT, the Prime Contractor H.R.I., and the blasting subcontractor, Douglas Explosives Inc. Also present was the EIC for the Route 15–I-86 interchange Project in Corning who was assigned to oversee this project. At the meeting the geologists for New York described the requirements for a pre-blasting meeting that must be held before any explosives can be brought onto the Right of Way. The meeting is run by an Engineering Geologist from the GEB and must include: the Project EIC, the Contractor's superintendent, and Project Blaster(s). Invitees include: emergency services, local authorities, and utilities in the area. The Project Blaster must have a valid NYS Explosives License and Certificate of Competence. This proved to be a hardship for the blasting contractor since they had not worked in New York and did not have a NYS licensed blaster on board. Fortunately, we had given the project a heads up and told the contractor of the next New York State Blasting exam and his blaster was able to pass the exam. Then the blaster had to wait several weeks until the license arrived before he could transport explosives into the State.

Prior to scheduling a pre-blasting meeting, the blaster must submit his blast plan in accordance with the GEB's blasting manual, GEM-22.*(7)* Seismograph certifications and copies of Explosives Licenses and Certificates of Competence are submitted to the Engineering Geology department. Included in the requirements is the need for construction of a presplitting test section of 50-100 linear feet that is exposed and cleaned completely to the bottom of ditch. This section is typically at the beginning of the cut extending to a minimum slope height of 10-15 feet so that a good representation of the drilling accuracy and slope stability can be assessed by a geologist. At this time the slope location is surveyed and any discrepancies from the design slope location are addressed. Following approval of the test section, the contractor may blast without removing material until the completion of the lift. Prior to drilling on a new lift, the first lift must be satisfactorily scaled of any loose material.

NYSDOT specifications include a maximum bench height of 60 feet for presplit rockslopes. The maximum bench width is one foot for slopes steeper than a 1 vertical on 1horizontal and three feet for a 1 vertical on 1 horizontal slope. The designed rock cut on this project has a maximum height of 131 feet and would require a minimum of three lifts. PennDOT's blasting specifications are similar to NYSDOT's and in many ways are more stringent. PennDOT has a separate pay item for presplit holes drilled within a tolerance of one foot in any direction. However, according to the project, this item is hardly ever paid, since the contractor bids low on it and the drillers do not spend extra time and effort trying to be within tolerance. PennDOT's maximum bench height is 15 feet with a 1.5 foot bench width. The PennDOT test section requires the end of the rockslope to be cut at three different angles and at different spacings. Following inspection of the resultant composite slope, a slope angle and drill spacing is chosen for the rest of the cut. NYSDOT specifies the rockslope angle and requires three foot spacing of the presplit holes.

The revelation that NYSDOT would allow up to 60 foot high lifts and require a minimum bench width of 1 foot caused the blasting contractor to rethink his original blast plan. There was much discussion over the ability of his drill rigs to set up on a one foot bench. The drilling contractor's rigs were large self-contained hydraulic drill rigs which could not be laid back easily against a slope. His contention was that any other type of rig would not be as safe for his drillers, but he left the meeting agreeing to see what other drilling equipment was available that would require less clearance and could achieve the minimum bench width.

PRE-BLASTING MEETING

The pre-blasting meeting was held on August 31, 2006. By this time, the blaster had passed his New York State Blasting exam and received his Own and Possess License and Certificate of Competence. The contractor had submitted a blast plan in accordance with the NYSDOT specifications. The meeting went very smoothly since there were no utilities in the area and no surprises for the contractor and blasting subcontractor.

Following the preblast meeting it was agreed as to how the inspection of the rock slope construction would be handled. In this respect NYSDOT has more stringent requirements in place. Following approval of the test section, PennDOT leaves the construction of the rockslope up to the contractor and only periodically inspects the slope. The blaster is responsible for providing daily blasting reports, complete with seismograph readings. Following completion of the blasting the project inspectors or EIC directs the final cleaning of the slope.

In contrast, NYSDOT assigns Engineering Geologists to rockslope projects. The Engineering Geologists provide training and oversight of the blasting. They are responsible for the approval of the test section and periodically visit the project to make sure the blasting and final slope is progressing well. The project is responsible for providing a full time blasting inspector to shadow the drillers and blaster and fill out the NYSDOT blasting form. Engineering Geologists are on the project to supervise the final cleaning and scaling of all lifts on the slope. No drilling of the lower lifts can begin until the final scaling of the upper lift is approved by the Geologist. This is for the protection of the drillers and to assess the stability of the slope. This is the easiest time to perform a thorough cleaning of the slope. The Project did provide a full time blasting inspector who was trained and assisted by NYSDOT Geologists.

TEST BLAST

Following removal of the overburden at the south end of the cut and construction of the haul road on the slope, the contractor realized that beginning to blast at the south end of the cut would cause problems for the removal of material. The blasting and removal of rock would destroy his haul road so he proposed having a test blast at the top of the cut. This would be at the center and highest part of the cut and then the blasting could continue in each direction. This was not the best location for the test shot since any problems encountered would be at the worst possible location on the slope. The north end of the cut terminates one third of the way into the slope so this was not any better. The Geologists agreed to the test section at the top of cut but made the contractor aware that in the worst case he would have to reshoot the cut behind the original top of slope if the test section failed. This could cause some major over excavation and redesigning of the final slope.

The test blast was on September 8, 2006. A total of 27 presplit holes were shot to a depth of 60 feet. Production blasting and cleaning of the slope took an additional two days. The presplit drill traces measured ranged from 55 to 59 degrees varying slightly from the designed 56.3 degrees (3V:2H). Approval was given to proceed with the presplitting. The contractor drilled and blasted presplit and production holes on the top lift. The blaster shot the presplit days ahead of the production holes in front of it. Production holes were 6 inches in diameter and the depths were only 25 feet long. Bulk loading trucks were used to load a maximum of 50 pounds of ANFO into each hole.

Figure 4 - Cleaning of the test section.

FIRST LIFT

The blasting for the first lift was completed in three weeks. Four large “Euclid” dump trucks were continuously loaded from the top of the cut and deposited the rock and soil 0.5 miles away on the north side of the new bridge. Scaling and cleaning of the rockslope began under the supervision of a NYSDOT Engineering Geologist and a PennDOT blasting inspector. The contractor used a backhoe with a long reach to remove large blocks of rock as he was removing the top lift and again as he removed the remaining rock from the lift. A final cleaning using a high pressure hose and water did an excellent job on the rock face.

Following the scaling the first lift was examined and surveyed to determine the location of the toe of slope. The overall slope face was stable and the competency of the rock was improving with depth. There was a 300 foot length of weathered rock extending to a depth of 25 feet. This weathered rock was regraded by mechanical means to flatter than a one vertical on one horizontal slope.

Figure 5 – Loading of the second lift

SECOND LIFT

The contractor began the second bench by drilling the presplit holes to a depth of 60 feet and the production holes to a depth of 25 feet. The drillers attempted to lay the drill rigs up tight against the rock face and were able to achieve a working bench that varied from one to two feet in width. This was acceptable to NYSDOT as every effort was made to minimize the bench width. During excavation of the middle lift the backhoe operator was able to effectively remove this bench.

Figure 6 - Drilling the second lift

Blasting and excavation of the second lift continued in the same manner as the upper lift. The rock had tightened up and the rockslope construction seemed to be progressing well. However, during the excavation of the top of the second lift, it was apparent that the drill traces varied by up to 20 degrees, from the design three vertical on two horizontal (56 degrees), steepening to a four vertical on one horizontal slope angle (76 degrees). A NYSDOT Geologist remained on the project for several days to observe the drilling operation at the south end of the second lift, and following excavation, the drill traces were found to be within two degrees of design.

This steepening of the drill traces set the bottom of the second bench back eight to ten feet behind the design slope location in places. A meeting was held on the slope to discuss possible design changes. The rockslope was looking very clean and stable even at this steeper slope angle. At the NYSDOT Geologist's request, the driller set up his drill rig against the rockslope. To continue on the same slope angle, the drillholes would have to be collared eight feet in front of the toe of slope. One option discussed was to steepen the bottom lift to a four vertical on one horizontal slope which would keep the bench width smaller, but greatly increase the excavation quantities and ditch width. The decision was made to start the third lift at the original design location, based upon practical drilling considerations, slope stability concerns, and the necessary transition to the adjacent New York State contract that will complete the construction of the rockslope.

Figure 7 - Drill set up on third lift. Notice how the rig cannot be laid up against the slope and drill at the design slope angle.

THIRD LIFT

The final lift was scheduled to begin this July, however as of mid-August there has been no construction work done on the rockslopes under the PennDOT contract. Blasting has begun at the south end of the NYSDOT Contract No. D260389 which begins at the north end of the PennDOT contract. There is a nearly seamless continuation of the rockslope along this top lift, but it remains to be seen how the rest of the rockslope will come out.

CONCLUSION

This is the first time that NYSDOT and PennDOT have worked together on construction of a large rockslope. Many obstacles such as different specifications, terminology, methods of measurement, regulations, contract pay items and inspection methodology were encountered, however good communication and partnering has moved the project along smoothly. Geologists and trained blasting inspectors and project personnel were able to detect and correct problems as soon as they occured. Hopefully, following the completion of both construction projects, a review of both State DOT's specifications, contracts, and construction inspection practices will be implemented, that will lead to improvements in rockslope and highway construction.

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Replacement of the Amelia Earhart Memorial Bridge
US-59 Highway Over the Missouri River
Atchison, Kansas

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Disclaimer

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Abstract

The Kansas Department of Transportation and Missouri Department of Transportation are currently working on the design phase of the replacement of the US-59 Bridge over the Missouri River. At this time there are two structural designs: one a Tied Arch and the other is an Overhead Truss. The design and construction of this structure presents several Geotechnical Dilemmas.

The foundation material for the structures both new and existing is the Weston Shale Formation. The Weston is problematic shale that contains several Paleosols and deposits of kaolinitic clays. Thick discontinuous near shore sandstone deposits are also common with minor inclusions of limestone and coal seams. The Weston Shale does not become a better foundation material with depth! Even the most competent of layers (Qu values of 100 tsf) can be underlain by material the strength of weak clay.

The Bridge Foundation Geotech investigation for this structure will consist of core-holes and power auger soundings of which a minimum of 10 will have to be conducted from a barge set up. A Geophysical Study of the proposed river piers will also be done. This study will entail: seismic reflection, sub-bottom profiling, side-scan sonar and a single beam echo-sounder. Also included in this investigation was the first ever 3 cell Osterburg Test. This test was done to mimic the bedrock conditions we will encounter with the proposed drilled shaft group foundation elements.

There will still be several problems facing the geotech section of KDOT during the design, and construction phase of the project. The most problematic will be the design and construction of drilled shafts in an ancient glacial river environment.

INTRODUCTION

The existing Amelia Earhart Memorial Bridge was constructed in 1938 and carries US-59 across the Missouri River. This bridge serves as a major arterial for Missouri and Kansas as the next closest river crossings are approximately 30 miles north and/or south of this location. The memorial bridge is a very unique structure utilizing 4 span types in 6 separate segments. The span types are: Steel Beam, Steel Girder, Steel Deck Truss, and Steel Overhead Truss. (See Photos)

Photo #1 Looking North at the Overhead Truss Section and UPRR Bridge

Photo #2, Looking west, Steel Deck Truss and Overhead Truss Sections

Photo #3 Looking east at the Steel Beam Section

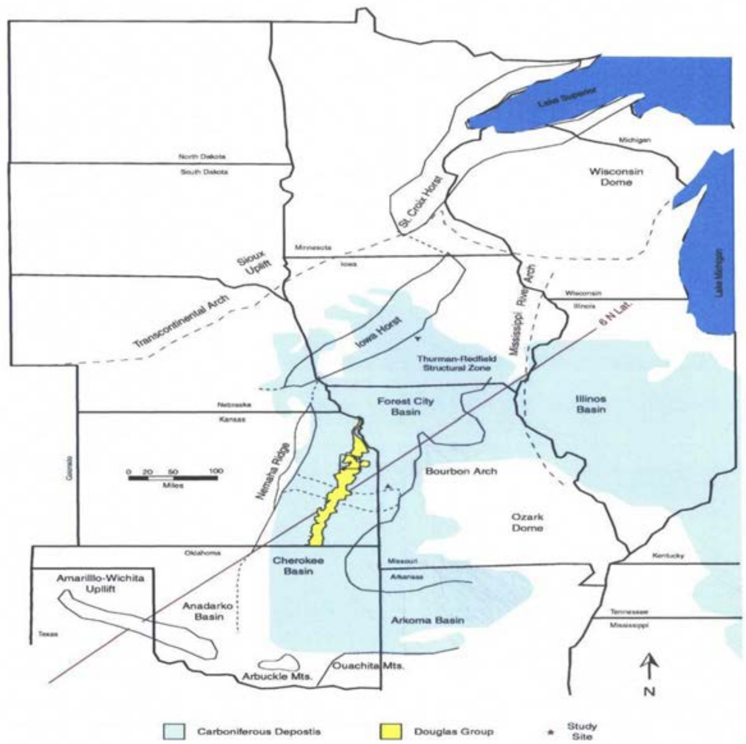
The railroad structure to the north of the US-59 Bridge is an electric turnstile bridge. As barge traffic approaches the bridge an operator rotates the bridge to parallel the river channel. This structure was built in the 1860's and originally was operated by man-power.

Photo #4, Railroad Bridge in Operation

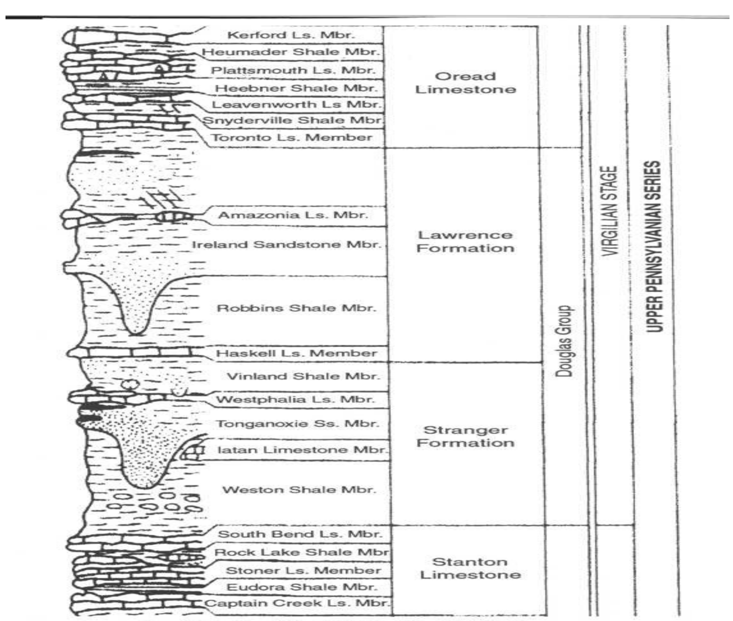
GEOLOGIC SETTING

Deposition of the Weston Formation of the Douglas Group occurred in the Carboniferous Period 354 to 290 million years ago. At that time Kansas was part of the super-continent Laurussia. Collisions with another super-continent (Gondwanaland) created the Ouachita, Appalachian and Ancestral Rockies Orogenies. In areas surrounding this portion of Kansas, uplift and subsidence were simultaneous. This up and down movement provided sediment sources as well as depositional basins. The Weston Formation was deposited into the Forest City Basin as portion of a series of cyclothems. (See Figure #1)

Figure 1: Tectonic Framework Surrounding Kansas during the Carboniferous



(Tectonic framework modified and adapted from Jewett, M., 1951, and Runkel, 2002, Rutan, D., 1975, and Steeples, D., et. al., 1979; the carboniferous deposits were adapted from http://www.paleoportal.org/time_space/period_map.php?period_ id=12; and Douglas Group was adapted from KGS Map M-23)



(Hagen and Hensiek, 1968)

Figure #2 Stratigraphic Column

These cyclothems on a grand scale are made up of a pattern of limestone, shales and sandstones. However when individual deposits are intensely studied a much more detailed pattern develops. Within the Weston Shale at this site we have identified the following facies:

Table 1: Depositional Processes and Interpretations of the Weston Shale Facies

Facies	Depositional Process	Depositional Interpretation
Churned Shale	Disruption or reformation of the primary sedimentary bedding due to bioturbation	Marginal marine (bivalves); continental (root whisps and plant fossils)
Shale	Low energy, sediment fall out	open marine environment below wave base
Clayshale	Suspension fall out	Ponded water (lagoons or abandoned tidal creeks and channels)
Coal	Peat accumulation and coalification	Wetland
Paleosol	Weathering and pedogenesis of pre-existing facies in reducing environments	Waterlogged soil
Carbonate-rich Shale	Both marine and pedogenic processes	Subaerially exposed limestone with overlying soil development
Lenticular Shale	Alternating periods of high energy (turbulent water) and low energy (slack water)	Subtidal, intertidal, or mid-mud flat
Sandy/Silty Shale	High energy with rapid sedimentation	Near-shore, fluvial-deltaic to estuarine
Weathered Silty/Sandy Shale	High energy with rapid sedimentation; churned and altered by modern pedogeneisis	Near-shore, fluvial-deltaic to estuarine; altered by modern weathering and pedogenesis

Table 1 by Dr. Archer, Kansas State University for the Kansas Dept of Transportation

The Weston Shale was found to be in excess of 150 feet thick and colored various hues of gray. The most complete stratigraphic section is located on the west side of the river.

Overlying the Weston Shale is a thick sequence of Alluvial Deposits. These recent deposits are predominately silts, sands and cobbles with the occasional Quartzite Boulder. The silt deposits are gray to light gray in color and extremely well compacted. The sands are made of quartz with minor amounts of chert, limestone, and feldspar. The isolated quartzite boulders are up to 14 feet in diameter which raises many concerns for pile driving and shaft construction.

PROPOSED BRIDGE INFORMATION

The proposed new structure is to be constructed with two main bridge types: Steel Girder and Tied Arch. Beginning on the Kansas side a steel girder section will be used up to the rivers edge followed by a 525 foot Tied Arch spanning the navigation channel of the Missouri, ending with another steel girder section. As of this writing, the bridge will be founded on 14 piers with only one pier within the Missouri River. The Abutments and Piers 8-14 will be set on driven H-pile and Piers 1-7 are utilizing Drilled Shafts foundation elements.

GEOLOGIC INVESTIGATION

The Geotechnical Section of KDOT was requested to provide all of the geological parameters for the design of the proposed structure. Included in this investigation would be the constructability of piers within the river. The investigation included the following: Core Drilling, Power Auger Soundings, Air Hammer Drives, Osterberg Cells and Offshore Geophysical Survey.

Power Auger Soundings

At each land based foundation element a minimum of one power auger sounding per element was conducted. These soundings were made with 3 inch continuous flight augers. The objectives were to not only find the mantle-bedrock contact but also search for the massive quartzite boulders. Boulders were found in one of the borings and its actual size was not determined. An additional objective was to ground truth the Air Hammer drives, when zones or layers of resistant material were encountered the Power Auger sounding gave us the ability to classify that material. To date 23 power auger soundings have been drilled.

Core Drilling

Core drilling was conducted at several of the piers on land and also from a barge within the river. The landside drilling was conducted utilizing KDOT's drill rigs and the barge work was contracted to Terracon. A minimum of 30 feet of rock core was obtained from each hole, with a maximum 110 feet of rock core taken at 2 river pier locations. All of the cores were logged including RQD, RMR and % Recovery calculations. Selected Samples were subjected to Unconfined Compression Testing and Slake Durability.

The coring on the river proved to be extremely difficult, mainly do to the extended periods of rain. The drilling operations were conducted at flows 10 feet higher than normal and over 8 mph. Removing debris from around the barge was a constant battle. Spuds on the barge were design with normal flows depths in mind, thereafter several holes had to be abandoned due to the depth of water. At one point the river was so high it spilled out over the levee's The barge was tied off but the river eventually won and the barge with the drill rig still aboard floated over a mile south of the project.

Constructability of the river piers were a much contested topic. The original bridge design had 2 piers within the water. The first was directly down stream of the existing Railroad and US 59 piers. The existing piers are constructed by the excavated caisson method and set approximately 3 feet into the Weston Shale. This location offered several issues: Scour, during construction and after and constructability in a turbid flow. The cores indicated that the foundation material was very strong in compression but was very weak in the lateral direction. Most of the Qu values for the shale were over 50 tsf but some cores broke laterally with as little as 3 psi! This weak lateral strength is due to very thin layers of organic rich material within the shale. Utilizing the proposed size of coffer dam it was determined that scour would completely remove any soil mantle material under normal flow conditions. Based on these findings the Tied Arch section was lengthened to 525 feet thus eliminating one river pier.

Photo #5 Core Drilling at High Flow!

Photo #6 Barge and Rig are Found!

Air Hammer Drives

The Kansas Dept of Transportation has developed an investigative tool for deep sediment situations. This tool is an Air Hammer. The Air Hammer drives a 2 inch washer into the soil at a known pressure. Much like a pile drive the blows and depths are recorded as well as the time. These are plotted and the depth of penetration and resistance give a very good indication as to the final depth of the pile.

Photo #7 Air Hammer

Air Hammer drives were conducted on Piers 8-14. All drives penetrated into the Weston Shale. The penetration was from 1.4 feet to just over 6 feet.

Osterberg Cell Testing

Due to the variable nature of the shale and Drilled Shafts being the preferred foundation element in the river a Loadtest (Osterberg Cell Test) was performed. We had originally planned on conducting 2 tests, one either side of the river to check the allowable skin friction for multiple layers of shale. However, this plan was changed when we could not get the equipment into the eastern site. We than changed to conducting one very deep test with 3 O-cells installed into the shaft.

Picture #8 O-cells

Picture # 9 Load Frame with 2 cells installed.

To closely mimic the proposed drilled shafts a 60 inch test shaft was constructed. The test shaft was drilled through 61 feet of fill and alluvium then advanced 100 feet into the Weston Shale. To a model a worst case scenario, the shaft was flooded and left standing for 4 days. After 4 days the load frame was placed into the shaft as it was being adjusted the middle O-cell broke free from the welds and causing it and everything else to fall to the bottom of the shaft! The O-cells and frame were removed from the shaft and reconstruction of the load frame and replacement of all hydraulic lines was required. Concrete was placed into the shaft to the top of the bedrock, representing a 100 foot rock socket.

The O-cell was systematically pressurized and relaxed to obtain the allowable skin friction for each of the shale layers. Three levels of maximum net shear were determined varying from 18.3 ksf to 24.4 ksf with a top loading of 35,000 kips. The settlement with this loading was 3.57 inches.

The O-cell testing results gave a much greater skin value than previously calculated (6 to 14 ksf) but the settlement is above our tolerance so a skin value between our calculated value and the Loadtest value will most likely be utilized in the design of the shafts.

Offshore Geophysical Survey

An Offshore Geophysical Survey was conducted to try and determine: depth of scour in the vicinity of the existing piers, determine the depth to bedrock and locate large Quartzite boulders near the proposed pier locations that would adversely affect the construction of the proposed piers. Kansas Dept of Transportation contracted with Golder Associates to perform this survey. They utilized seismic reflection, sub-bottom profiling, side scan sonar and a single beam echo-sounder.

Once again the river was not cooperating; it rose over 12 feet in just over 14 hours. This made the boat travel extremely treacherous especially around the piers. KDOT's bridge inspection team operating the boat did an outstanding job of not loosing Golder's 30,000 dollars worth of equipment!


Picture #11 Boat Loaded with equipment. Photo #12 Still surveying after a 12 foot raise!

The geophysical survey had mixed results. Due to the compacted silt layers, penetration and resolution was limited. The top of bedrock could be determined but not much more. No boulders where found near the proposed pier location. Given the flood event occurring during our survey we were able to map the scour hole and its migration between the existing piers. This scour hole was in excess of 52 feet deep. In hind sight this operation should have been done in conjunction with a ground truthing drilling operation rather than a stand alone investigative tool.

CONCLUSIONS

The replacement of the Amelia Earhart Memorial Bridge set to be let in August of 2008. The estimated cost for this new bridge and approach reconstruction is approximately 59 million dollars. The Kansas Dept of Transportation has used Drilling, Geophysics, Air Hammer Drives, O-cell testing and Power Auger Soundings to set the foundation design parameters in the Weston Shale. The Weston Shale is known to be problematic in numerous locations; we expected the same on this project. The varying material strengths were evident however the higher than expected Unconfined Compression Tests are a welcome addition. Even though these techniques were employed during some highest flows ever recorded on the Missouri River the true test will be the actual construction.

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Evaluation of Landslide Impacts to the Green River Bridge
A Preliminary Geotechnical Assessment

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ABSTRACT

On October 7, 2006 an earthquake occurred on Mount Rainier, Washington prompting inspection of all nearby bridges by the Washington Department of Transportation. Two days later, an inspection of the SR 169 Bridge over the Green River Gorge revealed that the natural soils and fill near the southeast corner of the bridge abutment had slid, displacing a small crib wall and exposing a significant portion of the abutment and pier. A second investigation was conducted a few days later to determine if the observed slope movement was part of a larger landslide complex called the Green River Landslide. Detailed investigations revealed that the observed slope movement was due to a shallow depth landslide covering approximately 1.75 acres and was not directly related to the larger landslide complex. The lateral scarp of this small landslide exposed a significant portion of the bridge pier, creating concern over the stability of the bridge and the integrity of the pier footing. As-built plans from 1932 revealed that the bridge pier was supported on a spread footing, but it was unclear as to whether the footing was founded on soil or on bedrock. To determine if the bridge was moving, the nature of the foundation material, and the extent and stability of the landslide a geotechnical investigation was initiated. Access difficulties complicated the investigation and a unique approach including geologic mapping, field-developed cross sections, hand auger probing, ground penetrating radar, seismic refraction surveys, pile integrity testing, and the installation of biaxial tilt meters on the bridge piers was used to investigate the problem. Findings to date indicate that the bedrock elevation is approximately 3 feet below the bottom elevation of the bridge footing, the landslide slip plane appears to be at the interface between the overburden soils and the underlying bedrock, movement within the bridge pier has been negligible, and the landslide was likely initiated by surface water runoff traveling down the side of the bridge abutment.

INTRODUCTION

The Dan Evans Bridge, built in 1932, is a steel truss bridge that spans the Green River Gorge. It is located approximately 2.5 miles south of the town of Black Diamond on SR169 at MP 5.3 (Figure 1). In 1995, erosion was observed on the eastern side of the southern abutment which led to the construction of a small crib wall to maintain the integrity of the bridge abutment (Figure 2). On October 7, 2006 an earthquake occurred on Mount Rainier prompting inspection

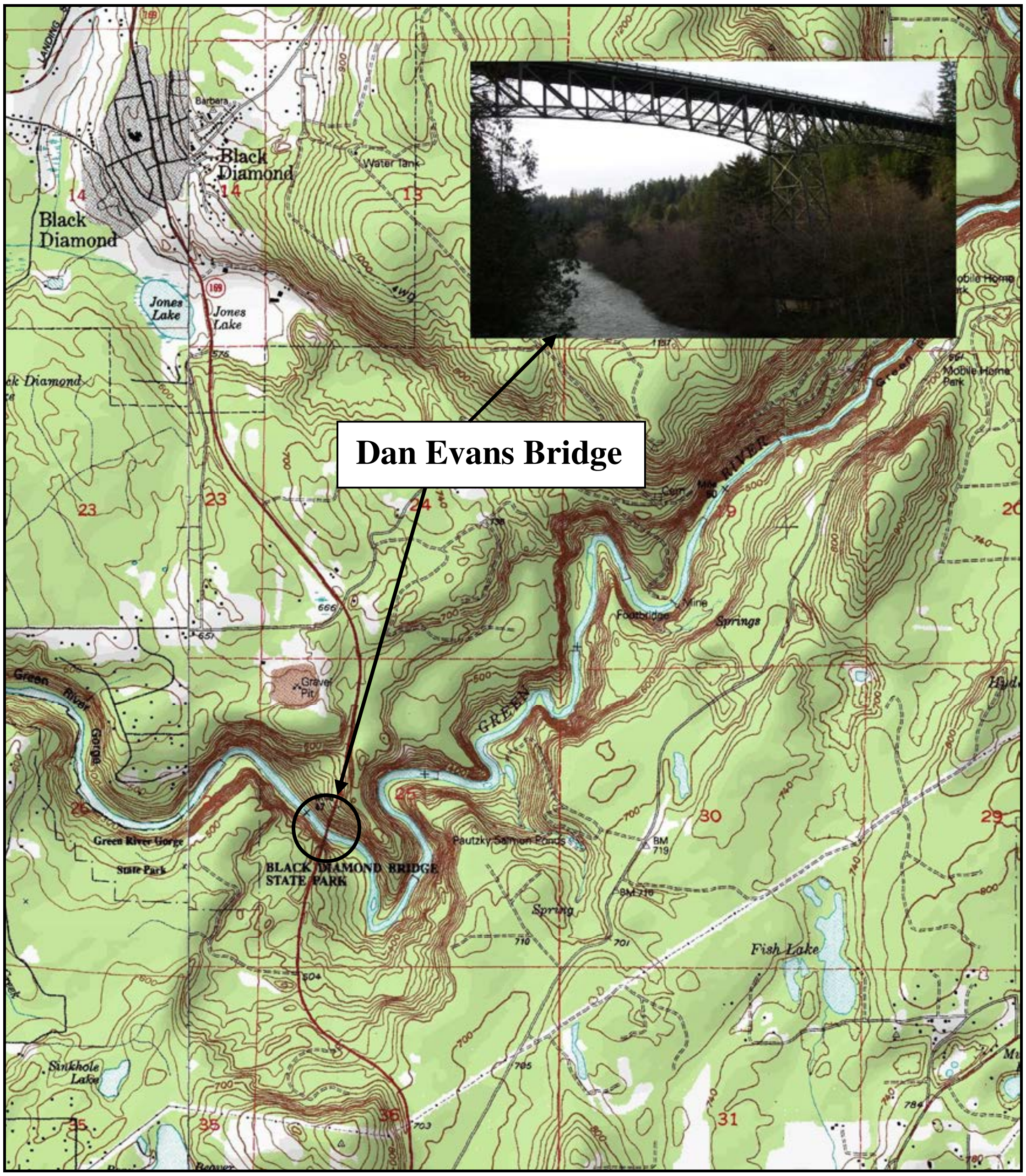

Figure 1: Dan Evans Bridge 169/08, 2.5 miles south of the town of Black Diamond.

Figure 2: Crib wall on southeast abutment constructed in 1995

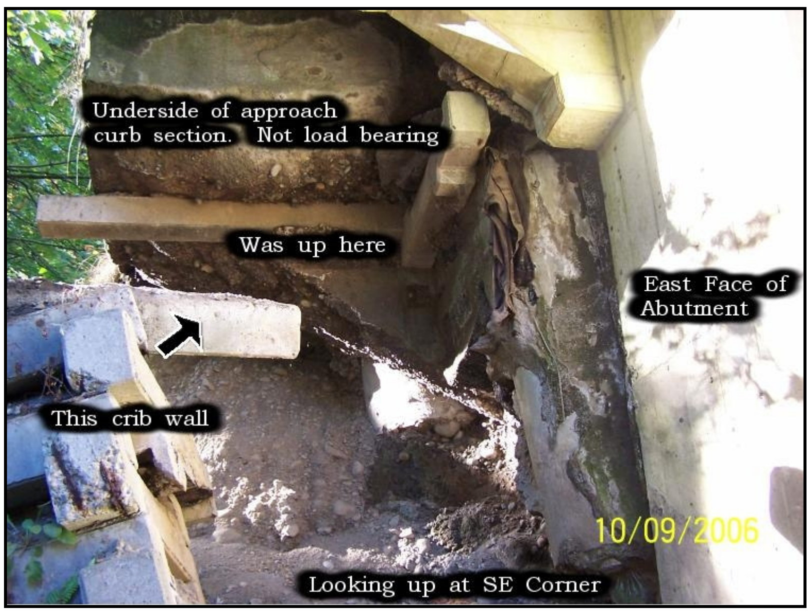

Figure 3: Displaced crib wall as of October 9, 2006.

of all nearby bridges by the Washington Department of Transportation. Two days later, an inspection of the Dan Evans Bridge over the Green River Gorge revealed that the natural soils near the southeast corner of the bridge had failed, displacing the small crib wall and exposing a significant portion of the southern abutment and pier 1 (Figure 3). It was evident from this inspection that the landslide was not a result of the earthquake but occurred sometime after a previous inspection on December 13, 2005. In the western foothills of the Cascades normal precipitation levels are at their highest during the earlier months of the year and it is likely that the slide occurred during the winter or spring of 2006. As a result of the observed landslide movement a geotechnical investigation was initiated.

This paper presents the results of a preliminary geotechnical investigation for the active landslide located immediately adjacent to the Dan Evans Bridge over the Green River Gorge. This active landslide has the potential to impact both pier 1 and pier 2 of the bridge structure.

GEOTECHNICAL INVESTIGATION

This landslide is located immediately adjacent to the southeastern side of the Green River Bridge. Landslide scarps (Figure 4), bent Douglas fir and Cedar trees, and a landslide toe (Figure 5) wasting over the cliff down to the Green River below were observed during the initial site reconnaissance.

Figure 4: Landslide lateral scarp against southern bridge pier.

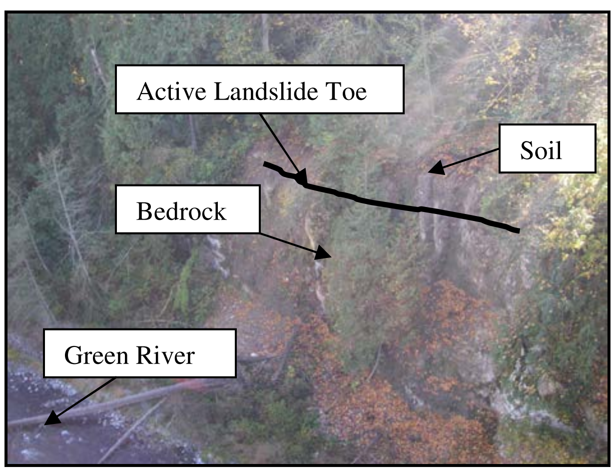

Figure 5: Toe of landslide advancing over face of cliff on October 31, 2006.

Geologic Mapping

On October 26, 2006, geologic mapping was conducted within the vicinity of the active landslide located immediately adjacent to the southeast corner of the bridge. The mapping, utilizing a global positioning system (GPS), sought to identify major geologic, geomorphic, and hydrologic features as they relate to the landslide (i.e. scarps, bedrock, and slope morphology).

The identified features were plotted onto a digital terrain model (DTM). The landslide is estimated to be approximately 1/2 acre in size (Figure 6). The geologic mapping revealed that the landslide is composed of glacial outwash deposits, consisting of silty sands and gravel that overlie sandstone bedrock. Two field developed cross-sections were completed through the landslide mass, A-A' and B-B'.

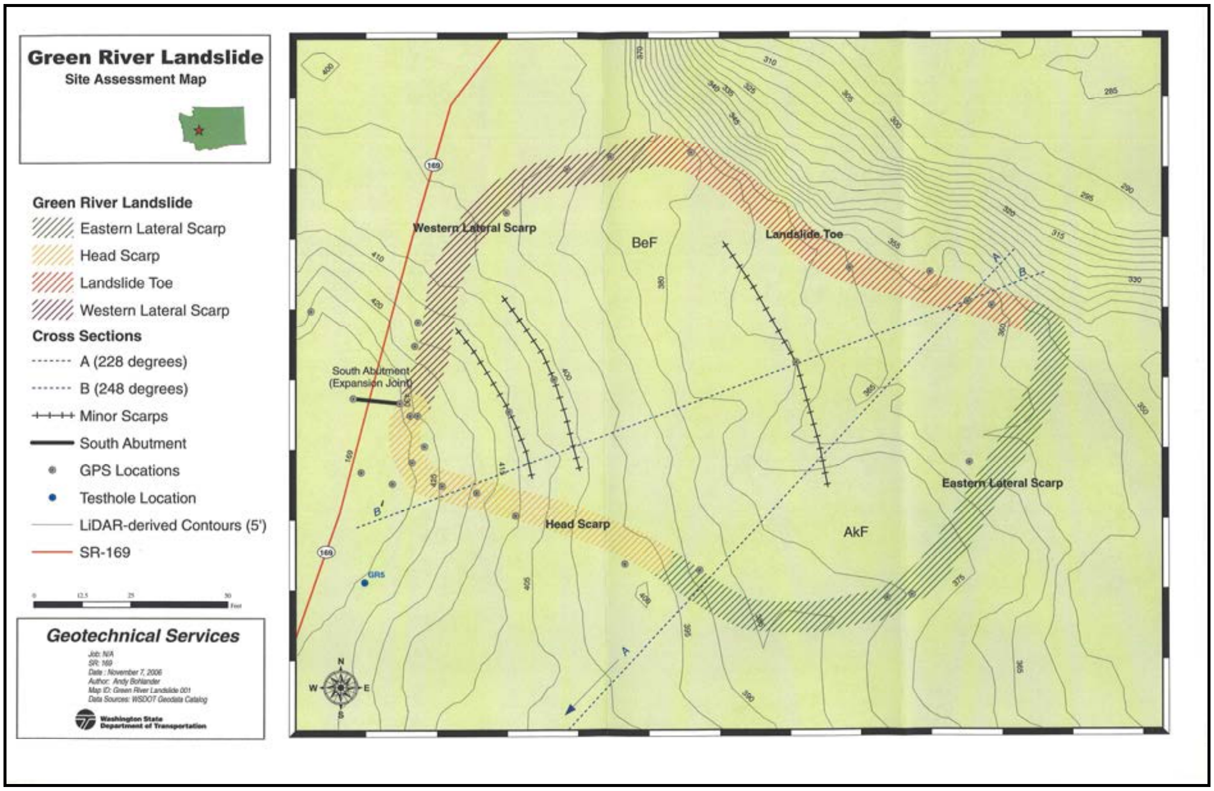

Figure 6: Mapped features of the landslide located adjacent to the Dan Evans Bridge.

The landslide is approximately 150 feet wide by 90 feet long. The landslide appears to be failing within the overburden soils and is toeing out in the face of the cliff and delivering landslide debris to the Green River below. In October of 2006, the head scarp of the landslide was estimated to be approximately 5 feet in height and was located adjacent to the southeastern abutment (Figure 7).

As-Built Plans Review

Review of the King County, 1932 as-built plans for the bridge over Green River Gorge reveals that piers 1 and 2 are on spread footings. The base of the footing at pier 1 is at an elevation of 395.31 feet and the base of the footing at pier 2 is at an elevation of 377.33 feet. The as-built plans for pier 1 state that the spread footing is 2 feet thick and the ground surface is 8 feet 5 inches above the top of the footing. The original ground surface is visible from staining on the side of pier 1. In October of 2006 the soil was approximately 5 feet 3 inches above the top of the spread footing (Figure 8). It is not clear from the as-built plans whether the spread footings for piers 1 and 2 are founded on sandstone bedrock or sand and gravel.

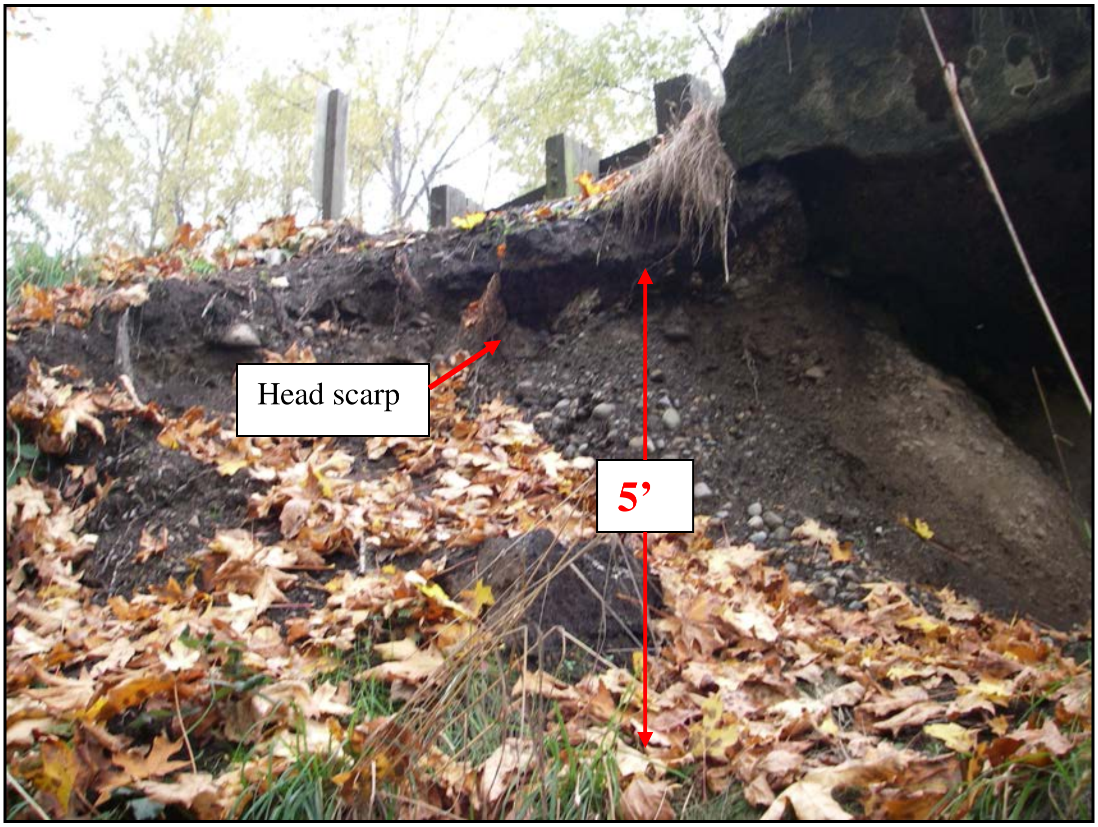

Figure 7: Landslide head scarp showing five feet of displacement on October 26, 2006

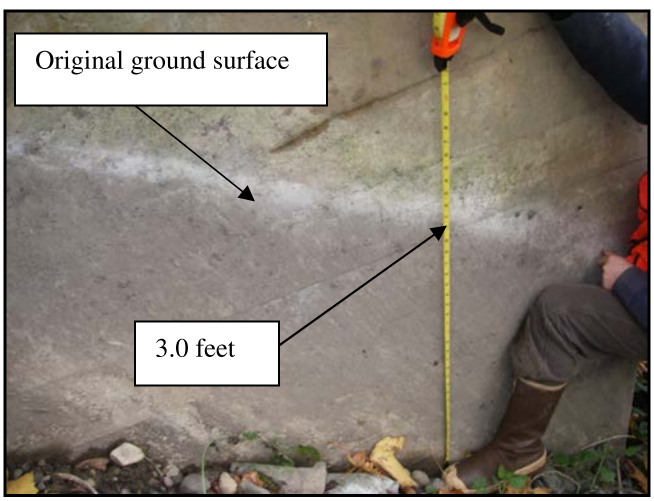

Figure 8: New ground surface on pier 1, October 26, 2006

Slope Inclinometer Data

Recent readings of an existing slope inclinometer were obtained. This slope inclinometer is approximately 30 feet southwest of the recently developed active landslide head scarp. The inclinometer readings indicated that 4 inches of movement had occurred at an approximate depth of 7 feet sometime between the dates of September 24, 2003 and December 1, 2005. This movement was before the recently observed slide activity. Subsequent readings of this inclinometer have indicated no movement has occurred since December of 2005.

Surface Water

In the longitudinal direction, the bridge deck slopes to the southeastern corner of the bridge. Bridge deck scuppers are present along the bridge deck. Foam and debris were observed in some of the bridge scuppers during a routine bridge inspection on December 13, 2005. The debris was apparently left over from a modified concrete overlay that had been placed on the bridge deck during the 2000/2001 construction season. On October 31, 2006 the scuppers were cleaned and a Cold Patch asphalt curb was created along the edge of SR169 to channel the surface water runoff away from the southeastern abutment of the bridge.

Field Exploration

To estimate the depth to bedrock an initial seismic refraction survey through the middle of the landslide along cross-section B-B' was conducted on October 31, 2006 (Figure 9). Based upon the as-built plans and the seismic refraction results, it was estimated that the pier 1 spread footing was founded near the bedrock surface and that several feet of soil still covered the base of the spread footing at pier 1.

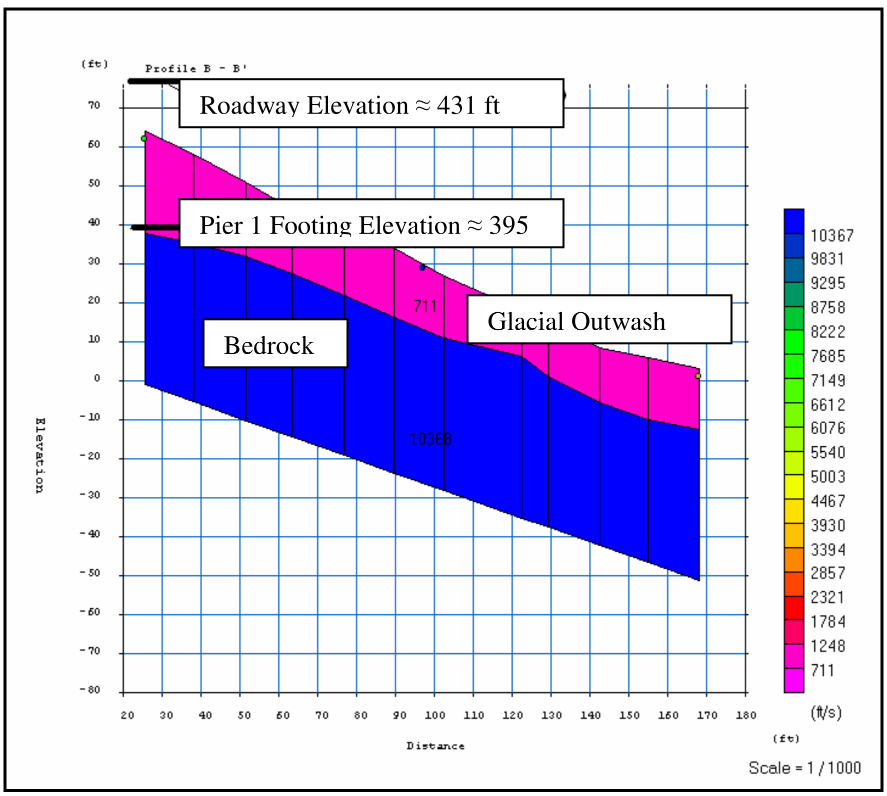

Figure 9: Initial seismic refraction results along cross section B-B'

LANDSLIDE ANALYSIS

Based on the initial field investigation, two engineering units have been identified, which are described below:

Glacial Outwash: This soil unit consists of a moist to wet, brown, silty sand with gravel. This soil unit overlies the sandstone bedrock.

Sandstone Bedrock: This rock unit is exposed on the cliff face of the Green River Gorge. It consists of alternating layers of sandstone, siltstone, and coal.

The above engineering units were used to develop a subsurface stability model. The stability model was analyzed with limit equilibrium methods. Based on back-analysis of the landslide model and utilizing the geophysical data and existing soils data from previous investigations, the following engineering unit properties were used for analysis.

Table 1: Engineering Unit Properties

Type of Deposit	Unit Weight	Shear Strength Cohesion	Shear Strength Friction Angle
Glacial Outwash	115 pcf	0 psf	34^o
Sandstone Bedrock	175 pcf	0 psf	40^o

The slope geometries were obtained through the field developed cross section and the depth to bedrock was determined from the initial seismic refraction survey. Three groundwater cases were analyzed. The results indicated that the slope failure is likely a circular type failure, the overlying glacial soils are moving on top of the sandstone bedrock, and as water is introduced into the slope its factor-of-safety drops from 1.63 as a dry slope, to 1.02 as a partially saturated slope, to 0.75 as a fully saturated slope (Figure 10).

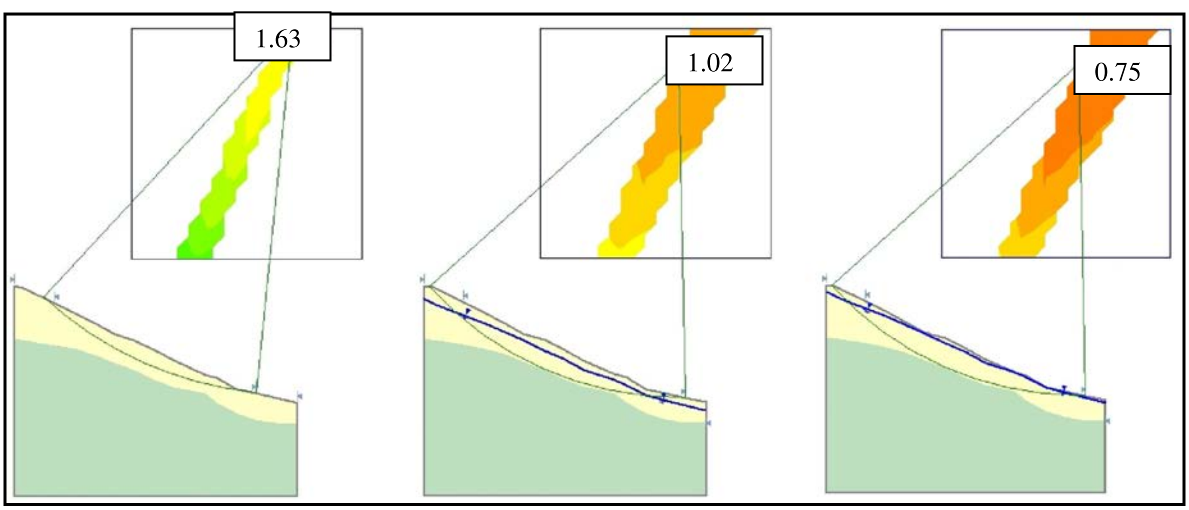

Figure 10: Landslide stability analysis for a dry, a partially saturated and a saturated slope.

Based upon our field investigations and analysis it appears that the contributing factors that most likely influenced the movement of the landslide include: 1) elevated groundwater conditions resulting from surface water running off the Green River Bridge deck down the southeastern side of the abutment, and 2) unconsolidated silts, sands and gravel overlying an adversely oriented bedrock surface.

Additional Field Investigations

To obtain additional subsurface data, within the foot print area of the southern abutment and piers 1 and 2, high resolution seismic refraction, ground penetrating radar, pile integrity testing on pier 1, and hand auger probing near pier 1 were conducted in mid-December 2006.

High Resolution Seismic Refraction

High resolution seismic refraction data were acquired with one line parallel to the bridge immediately below the roadway between pier 1 and pier 2 (Figure 11) and another line perpendicular to the bridge, 6 feet north of pier 1. The line between the piers was 80 feet long and the line perpendicular to the road was 94 feet long, both with 5 foot geophone spacing.

Figure 11: Seismic refraction layout - Line 1

Seismic data was processed using OYO Corporation's PickWin and PlotRefA programs, both part of the SeisImager 2D software package as well as Optim LLC's SeisOpt @2D V4.0. Simple two-layer and tomographic seismic velocity models were generated for both lines. Figure 12 presents the tomographic velocity models for seismic line 1 and interpreted material. Though the models vary in the calculated depth to bedrock, in all models the interpreted bedrock surface is calculated to be deeper than the footing elevation of pier 1 (395.31 feet).

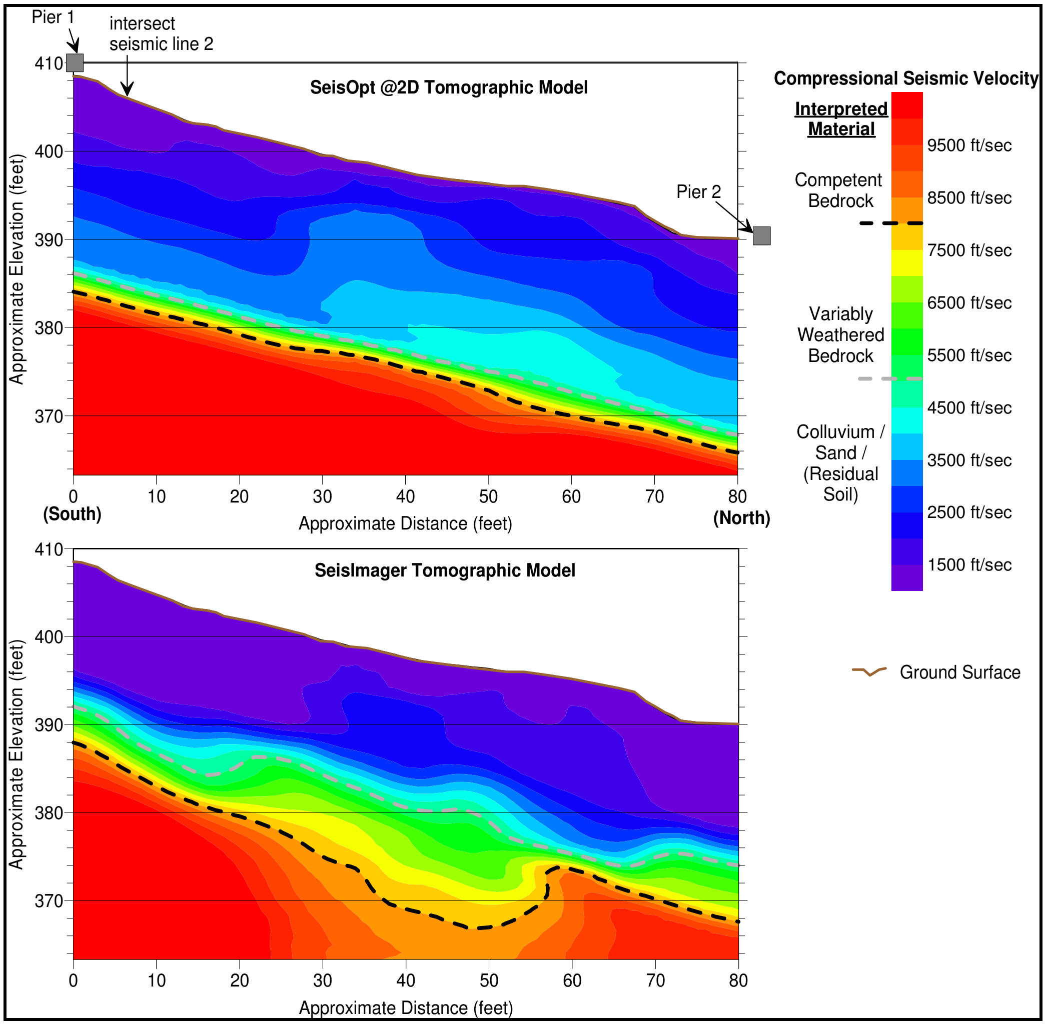

Figure 12: Tomographic model seismic refraction results – Line 1

The tomographic velocity models for seismic line 2 are presented in Figure 13. The SeisOpt @2D tomographic model (top panel of Figure 13) shows an undulating gradational pattern representing velocities from about 5,000 to 8,000 ft/s. This gradation is interpreted to define the boundary between unconsolidated material and the top of the sandstone bedrock. The irregular pattern is possibly the result of variations in the structure in the overlying landslide material on the eastern half of seismic line 2. The SeisImager tomographic model (bottom panel of Figure 13) shows a significantly sharper, less undulating contrast in velocity with depth. Again, all models suggest the bedrock surface is deeper than the footing elevation of pier 1.

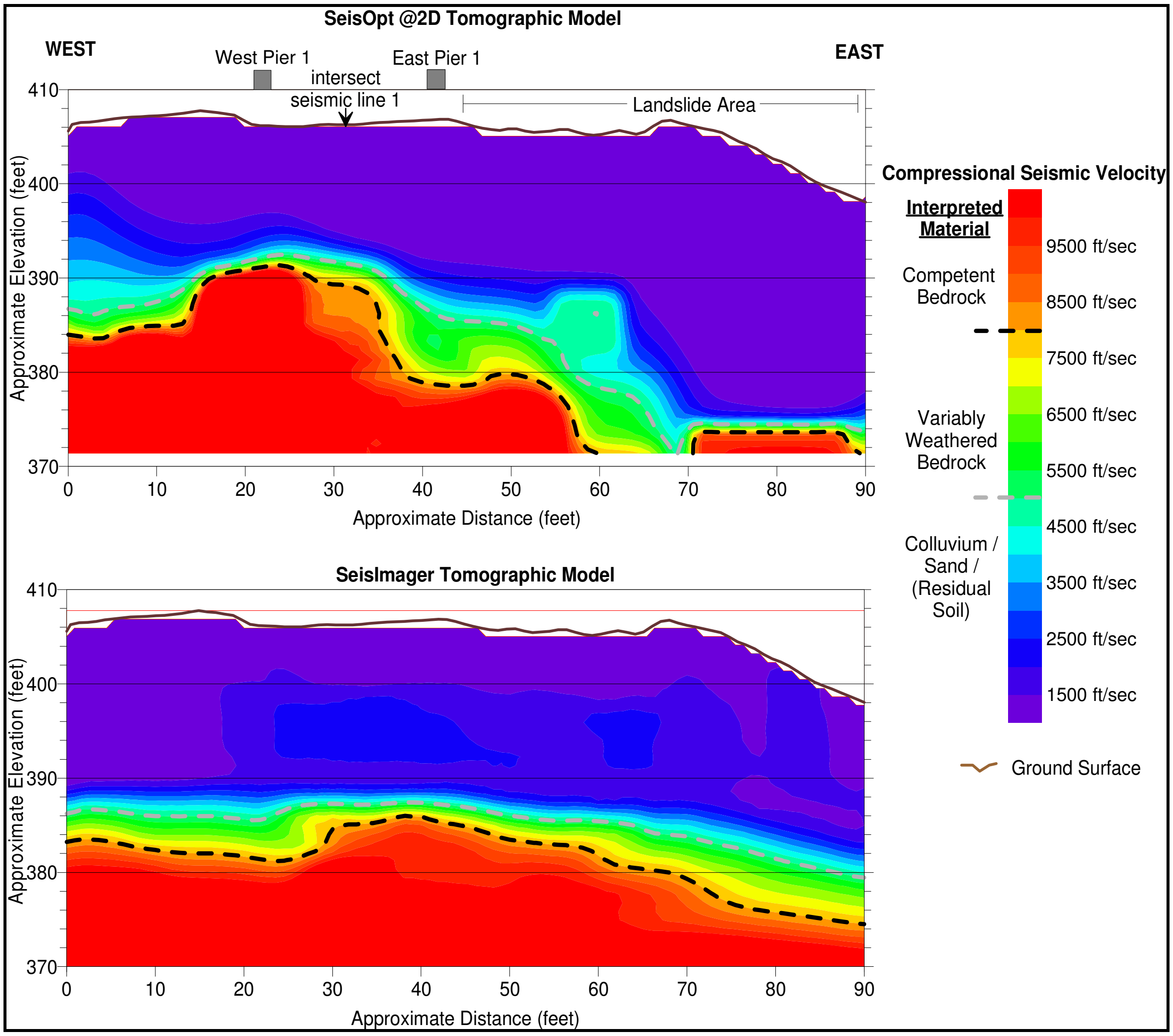

Figure 13: Modeled seismic refraction results – Line 2

Ground Penetrating Radar (GPR)

GPR records were collected along several transects covering the landslide area and areas adjacent to the bridge piers. A nearly continuous reflector appears in each of the records and is interpreted to be the top of bedrock. It could not be determined from the GPR data or the hand-augured data if the reflector represents the base of unconsolidated material and top of variably weathered bedrock blocks or the top of competent bedrock. GPR records shown in Figures 14 and 15 are scaled to show surface topography and approximate elevations. Both records indicate the elevation of the continuous reflector to be 1 to 3 feet below the spread footing elevation of pier 1.

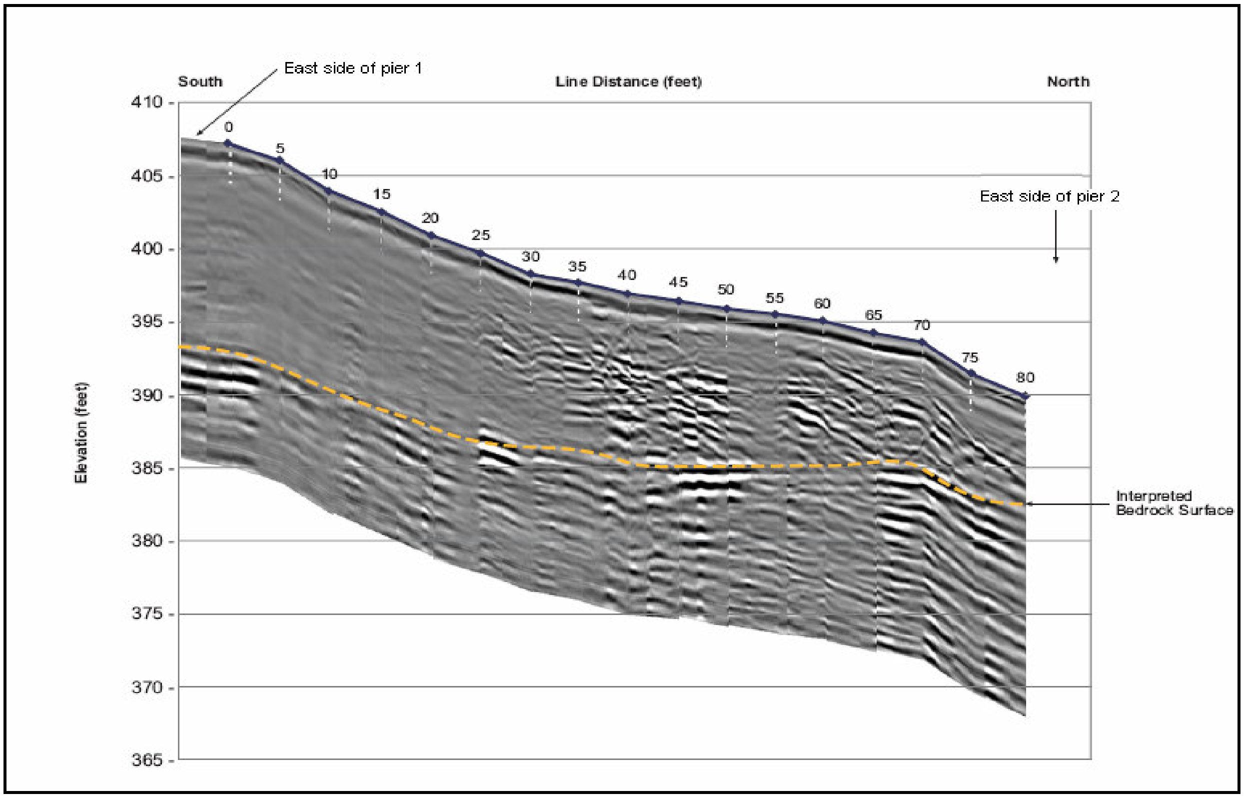

Figure 14: GPR record between pier 1 and pier 2 – east side

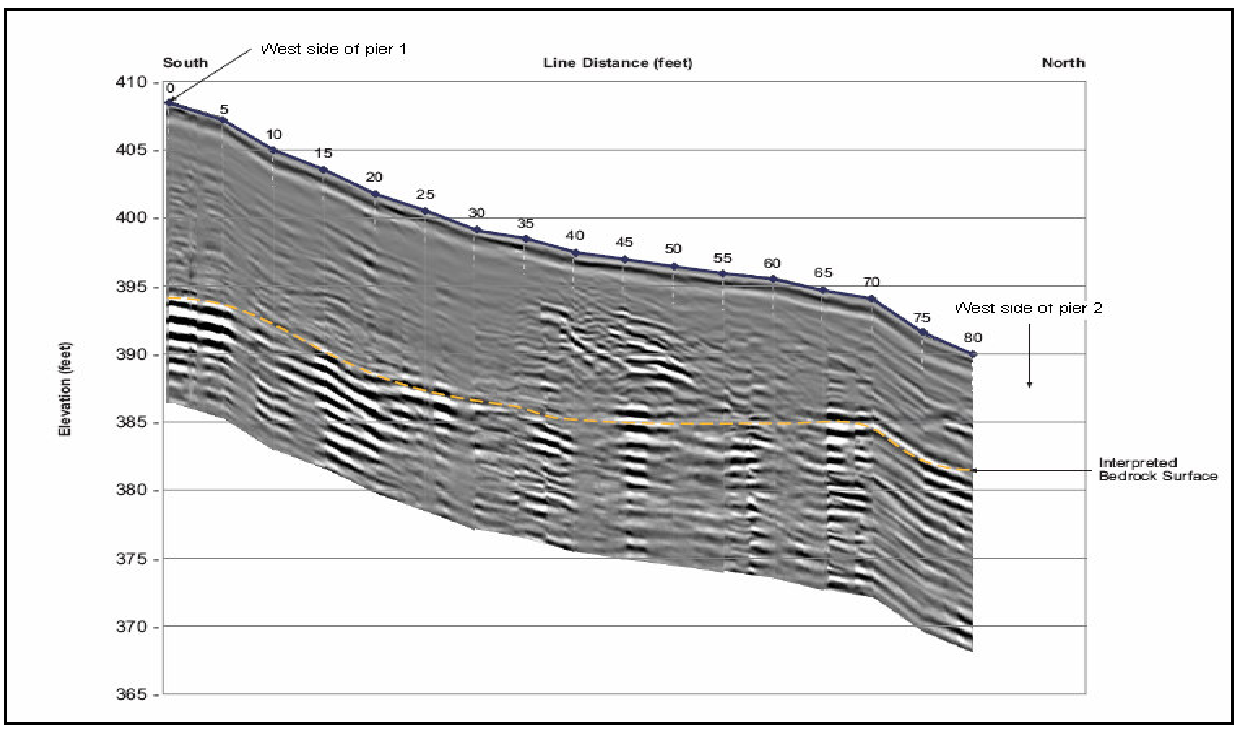

Figure 15: GPR record between pier 1 and pier 2 – west side

<u>Pile Integrity Testing</u>

Results from the pile integrity testing indicated that the elastic properties of pier 1 are similar to the elastic properties of the bedrock and a reflection at the contact between the two was not observed. Direct ultrasonic velocity measurements of the concrete at the surface of pier 1 were 8,650 ft/sec to 9,900 ft/sec while modeled compressional seismic velocities from the seismic refraction models indicated sandstone bedrock velocities of 8,000 to 10,000 ft/sec.

<u>Hand auger</u>

A hand auger probe three feet east of the eastern side of pier 1 was also completed during the geophysical investigation. The hand auger probe indicated sands and gravel exist to a depth of 13.5 feet below ground surface (elevation 392.76 feet) before refusal (Figure 16). The refusal elevation correlates to be approximately 2.5 feet below the base of the spread footing but it could not be determined whether refusal was on an unconsolidated material, the top of weathered bedrock, or on the top of competent bedrock.

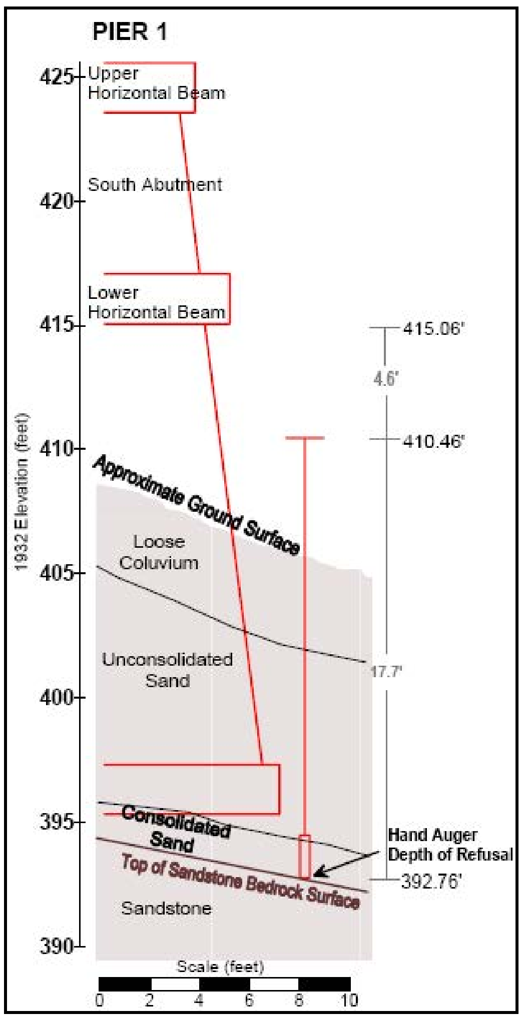

Figure 16 – Hand Auger Elevation Profile

Additional Landslide Movement

Sometime after completing the more detailed geophysical investigation on December 19, 2006 and the installation of the tilt meters on January 31, 2007 additional movement occurred within the slide mass. An April 18, 2007 site visit revealed that the entire slope had moved exposing a larger landslide toe (Figure 17), a larger head scarp (Figures 18 & 19), and larger lateral scarps (Figure 20). Upslope of the original head scarp location, additional landslide features were not observed. The Cold Patch asphalt curb that was built on top of the southeastern abutment was still in place, allowing surface water to drain from the bridge deck away from the landslide mass. It was not clear if the surface water draining off of the bridge deck was somehow finding a path into the southeastern abutment and contributing to the observed landslide movement.

In April of 2007, the head scarp was measured to be approximately 8 feet in height. The head scarp had encroached further up towards the bridge abutment. The active western lateral scarp of this slide was further pronounced and runs adjacent to the eastern side of pier. At this time, the ground surface was 4 feet 2 inches lower than the original ground surface (Figure 21). Based upon the as-built plans, approximately 4 feet 3 inches of soil remain above the top of the spread footing at pier 1.

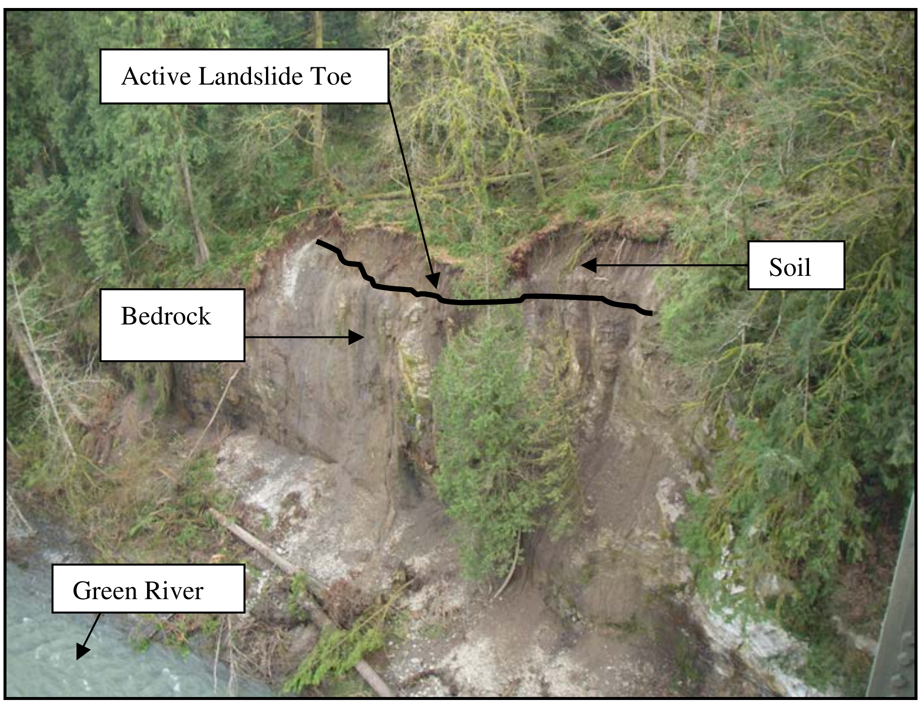

Figure 17: Toe of landslide advancing over face of cliff on April, 2007.

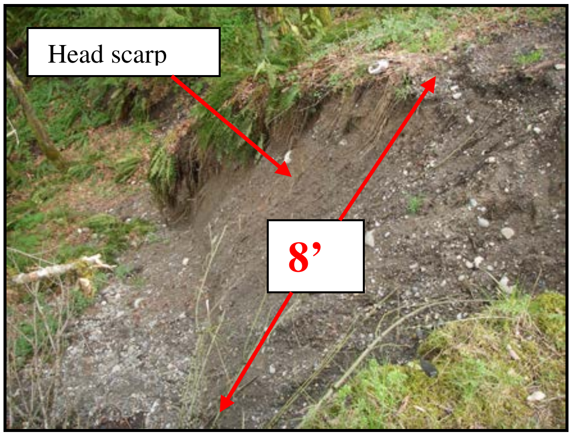

Figure 18: Head scarp with eight feet of displacement, April, 2007.

Figure 19: The head scarp encroaching upon bridge abutment, April, 2007.

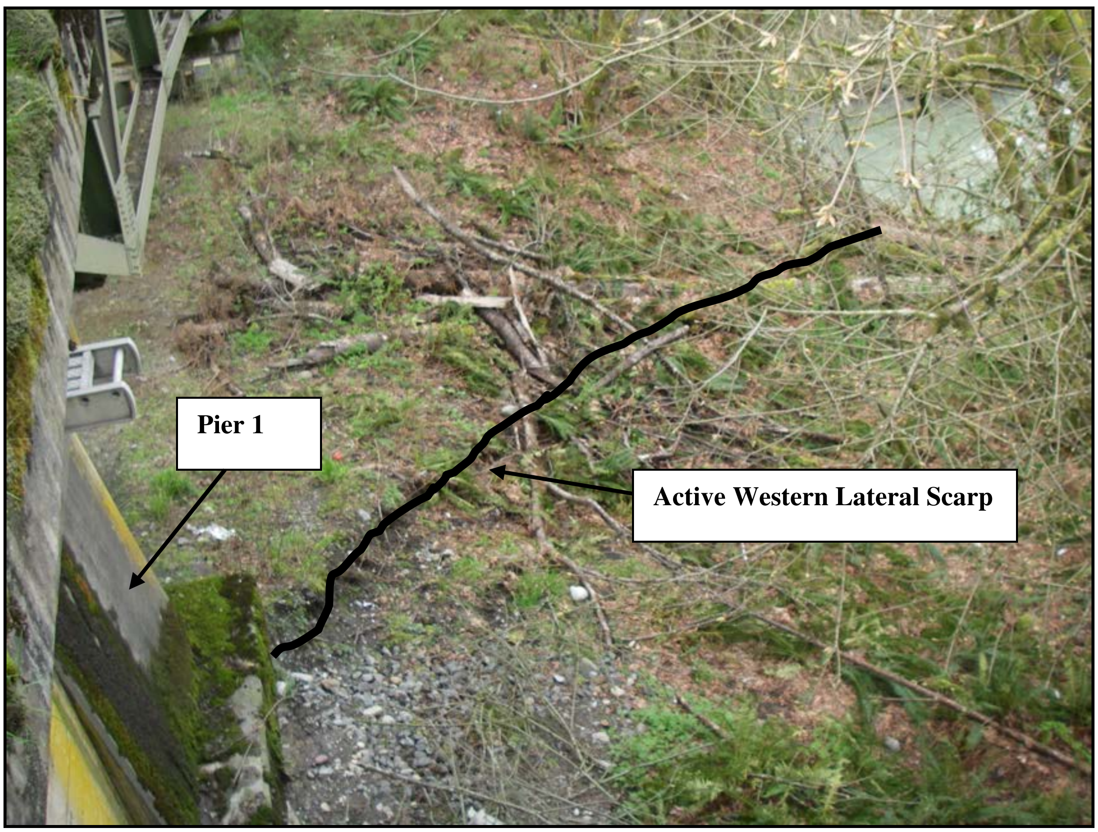

Figure 20: Lateral scarp running adjacent to the eastern side of pier 1 on April, 2007.

Figure 21: Ground surface on eastern side of pier 1 in April 2007. Original ground surface was 4.0 feet higher.

Additional Instrumentation

On January 31, 2007 bi-axial tilt meters with data recorders were installed on the eastern and western sides of pier 1 and on the eastern side of pier 2 (Figure 22). They record millimeters per meter of movement in two directions 90 degrees from one another. In July of 2007, tilt meter 3 had moved approximately 1.0 mm/m in the A direction, while the remainder of the tilt meters had shown negligible movement (Figure 23). Tilt meter 3 is on pier 2 and the A direction is parallel to the bridge with a positive direction towards pier 1.

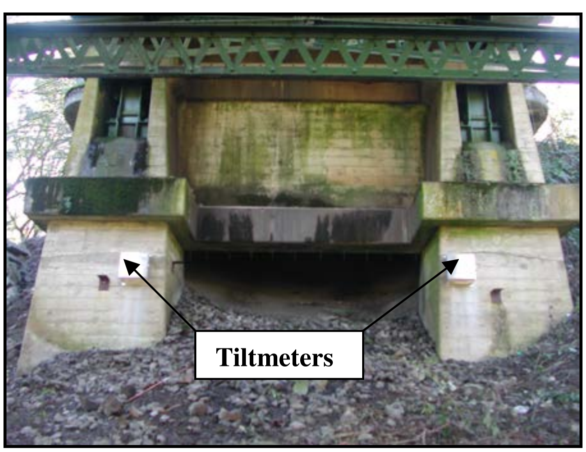

Figure 22: Tiltmeter installations

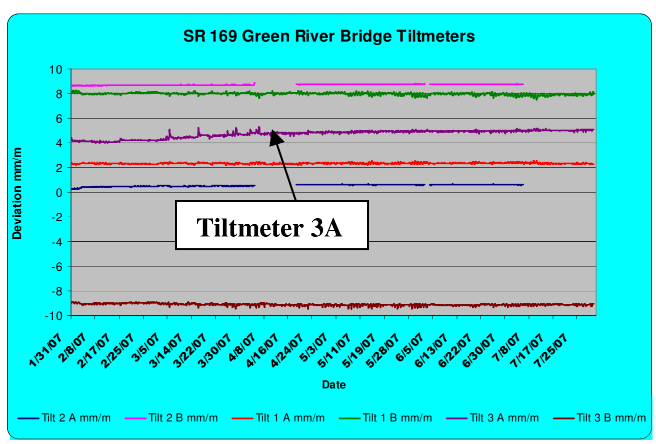

Figure 23: Tiltmeter 3A has shown some movement

SUMMARY AND CONCLUSIONS

Based upon published geological literature, previous geotechnical investigations, and exposed bedrock within the Green River Gorge the bedrock dips towards the west and plunges north towards the Green River below. The first seismic refraction line conducted along cross-section B-B' estimated bedrock to be approximately 1 to 2 feet below the base of the spread footing at pier 1. This analysis was not conducted immediately adjacent to pier 1 but was offset approximately 30 feet to the east and assumed a constant bedrock elevation going towards pier 1.

A more detailed geophysical investigation between piers 1 and 2 utilizing high resolution seismic refraction, ground penetrating radar, and pile integrity testing was conducted in mid-December 2006. The high resolution seismic refraction results indicated that bedrock is approximately 3 feet below the base of the spread footing on the eastern side of pier 1 and is approximately 2.5 feet below the base of the spread footing on the eastern side of pier 2. The ground penetrating radar results indicated that a reflector at eastern pier 1 is at the base of the spread footing and at 7 feet south of the eastern side of pier 2 it is approximately 5.5 feet above the base of the spread footing. It could not be determined if this reflector represented the base of an unconsolidated material, the top of weathered bedrock, or the top of competent bedrock. Results from the pile integrity testing indicated that the elastic properties of pier 1 are similar to the elastic properties of the bedrock and a reflection at the contact between the two was not observed.

A hand auger probe a three feet east of the eastern side of pier 1 was also conducted on December 18, 2006. The hand auger probe indicated sands and gravel exist to a depth of 13.5 feet below ground surface (elevation 392.76 feet) before refusal. The refusal elevation correlates to be approximately 2.5 feet below the base of the spread footing but it could not be determined whether refusal was on an unconsolidated material, the top of weathered bedrock, or on the top of competent bedrock.

Based upon this preliminary geotechnical investigation it is likely that the active landslide adjacent to the eastern side of pier 1 happened sometime during the winter or spring of 2006. Since that time, additional movement has continued to occur, putting the spread footings on piers 1 and 2 of the Green River Bridge in potential jeopardy. The geophysical investigations and hand auger probe indicate that the spread footings of piers 1 and 2 are most likely founded on 2 to 3 feet of sand and gravel and not on bedrock. The investigation to date indicates that this slide is not part of the much larger Green River Slide complex but is adjacent to it and is much smaller in nature. If the observed landslide movement continues, the spread footing on pier 1 and possibly pier 2 could be potentially undermined as the active landslide retrogresses towards the southwest.

FUTURE WORK AND RECOMMENDATIONS

It has been recommend that additional test borings be drilled through the southern end of the bridge deck, through the spread footing of pier 1 and into the underlying bedrock. These borings will ground truth the geophysical results for the western and eastern sections of pier 1 and confirm the characteristics of the material which lie directly beneath the spread footing.

Based upon the results of these test borings a detailed analysis of piers 1 and 2 may be required to determine any consequences associated with their movement or degradation. In addition, a full geotechnical analysis of the active landslide has been recommended to determine the landslide characteristics and to develop an appropriate landslide mitigation design.

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ABSTRACT

The Federal Highway Administration (FHWA) in conjunction with the Ohio Department of Transportation formed a work group comprised of 11 State DOTs, United Kingdom Highway Agency, USGS, USEPA, US Army Corps of Engineers, FHWA Ohio Division, and FHWA Office of Federal Lands Highway to oversee the development of data dictionaries and data formats for geotechnical management systems through Transportation Pooled Fund (TPF) project TPF-5(111) "Development of Standards for Geotechnical Management Systems". One of the products being produced through the pooled fund project is a geotechnical and geoenvironmental data exchange standard called Data Interchange for Geotechnical and Geoenvironmental Specialists (DIGGS). The first version of DIGGS is being released in 2008 and will include standards for borehole, laboratory test, deep foundation, and borehole geophysics data.

DIGGS provides a standardized means of geotechnical and geoenvironmental data exchange between disparate databases. There are several significant advantages to the user of DIGGS including: ability to exchange data between databases within an organization and with external organizations, ability to efficiently incorporate data from consultants into any database, ability to perform software-automated data checks, ability to exchange data between compatible software packages, and the ability to merge databases and incorporate software into an integrated geotechnical management system. DIGGS facilitates the seamless flow of geotechnical and geoenvironmental data from point of generation, through project usage, to storage, and then reuse.

Several DIGGS compatible tools will be available at the time of the release of DIGGS version 1.0. These tools include: a database with GIS interface for state transportation agencies, software for subsurface data reporting, a virtual data center that enables data exchange across organizational boundaries, and the United Kingdom Highway Agency geotechnical management system. Several geotechnical and geoenvironmental software vendors have already included DIGGS translators in their software.

INTRODUCTION

Many agencies, companies, and organizations are struggling with managing increasing volumes of geotechnical and geoenvironmental data, geo-structural assets, and geo-hazards. The ultimate answer to addressing many of the associated frustrations is a comprehensive, electronic geotechnical management system (GMS) capable of managing, evaluating, and manipulating data sets. The development of such a system requires a significant investment of time and money, along with specialists with expertise in software and database development, geotechnical and geoenvironmental knowledge, and geographic information systems (GIS). Currently, many state transportation agencies are in various preliminary stages of developing geotechnical management systems. These systems will provide the means for efficient data storage, retrieval, and utilization for enhanced decision making.

Geotechnical activities and features are typically on the critical path during the design and construction phases of projects. These activities and features represent a notable percentage of the design and construction costs as well. Consequently, geotechnical information represents an essential risk management aspect during design and construction.

Five transportation agencies have evaluated the impacts of a GMS on their operations, and have estimated their savings. Examples of estimated savings include:

- The United Kingdom Highway Agency has a Geotechnical Data Management System (GDMS) for management of subsurface exploration data and management of slopes and other geotechnical features. They estimate that by using the management system to initiate proactive maintenance, they save 80% of the cost of slope repair system-wide.

- The Ohio Department of Transportation (DOT) estimates that they can save $12 million to $24 million per year by using previously collected subsurface exploration information. In the past, 90% or $52 million worth of subsurface exploration data was discarded annually because it was not in an electronic format and there was no effective means to store it.

- The Florida DOT estimates that they will save $250,000 to $500,000 for subsurface investigation of a project for widening and reconstruction of I-595 by using previously collected exploration data recently made available in an electronic database. They also saved several hundred thousand dollars on a project for widening a bridge on I-75 by using historical boring information.

- Missouri DOT estimates an annual savings of $81,000 in preparing boring logs by electronic entry of data in the field by 4 crews. In addition, they estimate an annual savings of $20,000 by reducing boring needs by 10% to 15% for 10 to 15 structures per year by using historic boring data.

- Minnesota DOT estimates that by using its electronic database, in lieu of hard copy information, they save $20,000 per year in personnel time alone to look up boring information.

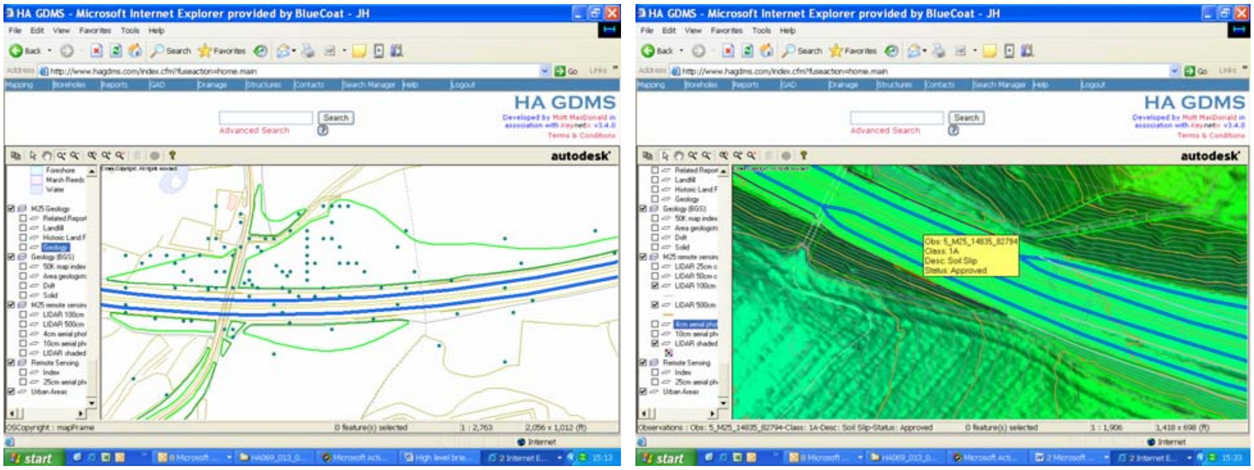

Figure 1 – The United Kingdom Highway Agency Geotechnical Data Management System gives easy access to borehole, LIDAR, and other information through a GIS application via the World Wide Web.

The state DOT examples of projected savings only account for using subsurface exploration data from their own records. A key element for a successful GMS is the ability to receive data from, and send data to, other entities outside of the organization owning the management system. There is an opportunity on many projects to use exploration data from other state, local, and federal agencies if that data can be readily obtained and easily accessed in a usable form. Data interchange standards are necessary to allow this exchange to occur efficiently and effectively.

The Federal Highway Administration (FHWA) Office of Federal Lands Highway (FLH) represents an example for highlighting the potential benefits of sharing geotechnical information across organizational boundaries. This office primarily develops, delivers, and administers a coordinated transportation program for roadways within, and accessing, federal land management areas comprising thirty-percent of the country. Many of the roads and bridges on this federal network intersect and border state and county owned roadways. The ability to easily and efficiently share existing geotechnical information between state and federal agencies could optimize all available geotechnical-related resources and activities considered in program development and project delivery nationwide. With a transportation program approximately the size of the thirteenth largest state DOT, the potential for savings is considerable.

A successful GMS must be able to share data between diverse data bases within the management system. In addition, it must be able to transfer data into and out of software programs and between various software packages that are used in project development and management.

Without transfer standards, data in one database must be manually mapped directly to another database, a difficult and time intensive process which must be repeated for each database accessed. With a data transfer standard, each database only needs to be mapped once to the standard. Afterwards, the database can be accessed by any application through the data exchange standard.

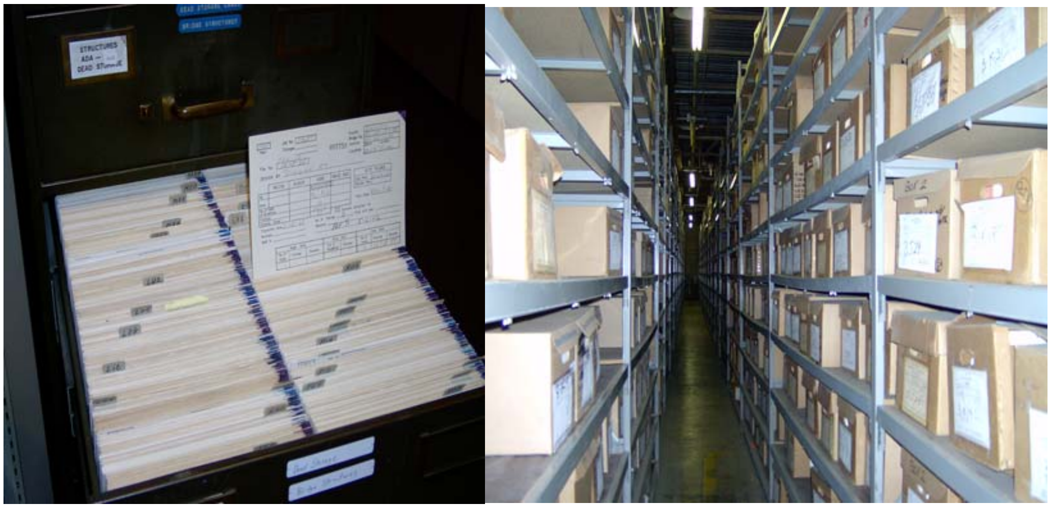

Figure 2 - Paper storage and retrieval is cumbersome and labor intensive. Electronic storage and retrieval significantly increases efficiency and accessibility.

An example of data interchange using a data standard is the use of a PDA in the field. The subsurface exploration data can be entered at the drill rig as the boring is being conducted. The data can then be sent electronically through the web via the data standard to the DOT office where it is electronically checked and validated with software, entered into a database along with pertinent project information, and used for boring log generation and/or modeling applications. The data can be maintained electronically in the database and retrieved when needed for future use.

Figure 3 – With a data interchange standard, data entered in the field can be sent, used with various software programs, stored, and reused.

A data interchange standard is more than a "cradle to grave" solution; it is more like a "delivery room to cryogenics" solution. It handles the data from point of generation, through project usage, and then allows it to be accessed and used for future purposes.

An advantage of a data interchange standard is that it allows the user to create or purchase software that performs data validation quality checks. This is possible because all of the data is in a standard format. The data validation can then be automatically performed by software at the point of generation, or performed as it is received or transferred.

Data interchange standards will have significant implications for software vendors and packages, and software customers by permitting interchange of data between individual software programs and databases. It will no longer be necessary to obtain an entire suite of software from a single vendor in order to assure seamless data compatibility. Nor will it be necessary to incur the cost and frustration of attempting to develop conversion programs to transfer data from one software program to another. Also, historical data will not be lost when changing from the software of one vendor to another vendor. A larger market will be open to vendors because the software will be able to access any database that is mapped to the standard. Software customization time and costs will be reduced as a consequence.

POOLED FUND PROJECT

In June 2004 FHWA brought together state and federal agency representatives by co-hosting the *National Geotechnical Management Workshop: Archiving and Web Dissemination of Geotechnical Data,* in partnership with the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) and the Pacific Earthquake Engineering Research (PEER) Center Lifelines. This workshop brought together a wide range of individuals, organizations, agencies, and companies interested in electronic interchange of geotechnical data. Based on interest expressed during a breakout session of state DOT and FHWA representatives, the FHWA and the Ohio DOT formed a GMS Group and began the steps necessary to formalize a transportation pooled fund project.

The goal of the GMS Group is to develop an open framework geotechnical management system that can be web enabled; can be used to store, manipulate, manage, and validate data; provides a means to efficiently and proactively manage geotechnical assets and geologic hazards; can be used as a tool to share information among interested entities; and can accommodate modifications to meet local needs.

Members of the Geotechnical Management System Group:

- California DOT (CALTRANS)
- Connecticut DOT
- Florida DOT
- Georgia DOT
- Indiana DOT
- Kentucky DOT
- Minnesota DOT
- Missouri DOT
- North Carolina DOT
- Ohio DOT
- Tennessee DOT
- FHWA Ohio Division
- FHWA Federal Lands Highway
- United Kingdom Highway Agency
- United States Army Corps of Engineers
- United States Environmental Protection Agency
- United States Geological Survey

Figure 4 – GMS Group Members

The first priority of the group is to develop geotechnical and geoenvironmental data interchange standards through pooled fund project TPF-5(111), “Development of Standards for Geotechnical Management Systems”. The name of these standards is Data Interchange for Geotechnical and Geoenvironmental Specialists (DIGGS).

diggs | data interchange for geotechnical and geoenvironmental specialists

Figure 5 – DIGGS Draft Logo

DIGGS is being developed through a cooperative effort of owners of existing data interchange standards such as the Association of Geotechnical and Geoenvironmental Specialists (AGS), United States Environmental Protection Agency (USEPA), United States Army Corps of Engineers (USACE), Consortium of Organizations for Strong-Motion Observation Systems (COSMOS), University of Florida, and others, as well as geotechnical and geoenvironmental specialists with an interest in data interchange standards. The organizations involved in the development of DIGGS have agreed to adopt it as their standard for use following it’s completion. Throughout the development of DIGGS, the GMS Group has had full, open communication and cooperation with representatives from the National Cooperative Highway Research Program (NCHRP) Project 20-64 *TRANSXML: XML Schemas for Exchange of Transportation Data*.

The ultimate goal of DIGGS is to include all geotechnical and geoenvironmental related data. The broad categories of data include geotechnical exploration data (collected from boreholes, test pits, laboratory tests, in situ tests, geophysical testing, etc.), geo-structural assets (such as deep foundations, shallow foundations, and retaining walls, and their associated construction control testing data), geo-hazards (such as landslides, rock slopes, karst, mines, etc.), and geoenvironmental data (from field and lab testing of soil, groundwater, and surface water and water level gaging). The data standards are being developed in a staged process by technical groups called Special Interest Groups (SIGs). The first SIG combined and modified existing standards for boreholes, laboratory and in situ tests, borehole geophysics, and deep foundations. Another SIG is combining and modifying standards for geoenvironmental data.

DIGGS utilizes Geography Markup Language (GML) compliant eXtensible Markup language (XML) schema conforming to standards developed by the Open Geospatial Consortium (OGC). The OGC is a non-profit, international, voluntary consensus standards organization. XML is a means to represent information, such as data, along with the role that the information plays. For example, for a borehole, the XML schema would indicate how a sample relates to the hole and how a test relates to a sample. XML was developed to facilitate data interchange between different databases and systems, particularly using the internet. GML is an XML schema containing a defined set of geographic tags to locate the data geospatially.

DIGGS will provide a standardized format for geotechnical and geoenvironmental data exchange between disparate databases. Significant advantages to the user of DIGGS include: ability to exchange data between databases within an organization and with external organizations, ability to efficiently incorporate data from consultants into any database, ability to perform data validation checks, ability to exchange data between software packages, and the ability to merge databases and integrate software into a geotechnical management system.

DIGGS VERSION 1.0

DIGGS is being developed and released in a phased manner. The first version of DIGGS is a consolidation and expansion of existing standards developed and used by the AGS, COSMOS, and University of Florida.

DIGGS version 1.0 covers borehole data, in situ tests, laboratory tests, borehole geophysics, and deep foundations. The draft version is currently being reviewed through a limited distribution to GMS Group members and selected individuals and organizations. Following receipt of comments and subsequent corrections, final release is expected in early 2008.

Several DIGGS compatible tools will be available when DIGGS version 1.0 is released. These tools include: a database with GIS interfacing for state transportation agencies, software for subsurface data reporting, a stand-alone data checker that allows viewing/editing of data as well as implementation of business rules, a virtual data center that enables data exchange across organizational boundaries, and the United Kingdom Highway Agency Geotechnical Data Management System.

The database tool mentioned above is being developed by the University of Florida for Florida DOT. The North Carolina DOT is developing and adding the GIS interface. The database will accommodate all data included in DIGGS version 1.0 and will be made available to any state DOT.

A Geotechnical Virtual Data Center (GVDC) is being developed through a COSMOS/PEER Lifelines Project. The project is currently developing a pilot web-based system linking example geotechnical data sets from Pacific Gas & Electric (PG&E), CALTRANS, California Geological Survey (CGS) and USGS. The ultimate goal of this project is to extend the pilot system and develop a web-based system linking multiple data sets, capable of serving the broad needs of practicing geotechnical and earthquake hazards professionals for efficient access to geotechnical data. The GVDC uses DIGGS as its standard for data interchange.

Software vendors are cooperating with the project and have been providing significant assistance with the development of DIGGS. Several vendors have already added the translation capability for DIGGS to facilitate the exporting and importing of DIGGS files by their software.

GEOENVIRONMENTAL DATA

The geoenvironmental component of DIGGS is being developed considering existing data exchange standards and the needs of data providers and data users. Several data exchange standards were looked at but the primary standards considered were the Association of Geotechnical and Geoenvironmental Specialists – Environmental (AGS-E), the Standard Electronic Data Deliverable (SEDD) developed by USEPA and USACE, and the EQuIS UK EDD.

SEDD is a comprehensive data exchange standard currently used by the USEPA for receipt of data. It was developed in response to the flood of test data being submitted for super fund sites. Since no common data format existed at the time, the data was being received in the format used by the particular lab. There are at least 220 different electronic data formats being used by environmental laboratories in the United States. Several laboratories have implemented SEDD and are inputting SEDD files into electronic review software. Preliminary results show a 30% to 50% cost savings when compared to the same level of manual review.

FUTURE WORK

New opportunities for collaboration occur regularly. All development efforts are taking into account data interchange structures developed by others to avoid conflicts and duplication of effort. DIGGS is building upon the successful work of others.

Immediately following completion of DIGGS version 1.0, the project will begin work on two-dimensional (2D) generic data standards for planar data such as test pits. Subsequent work, following geoenvironmental and 2D development, will include geophysics, geo-structural assets, and geo-hazards.

DIGGS is being reviewed by Joint Technical Committee 2 (JTC2), Representation of Geo-Engineering Data in Electronic Form, for adoption as an international standard. JTC2 is a Joint Technical Committee of the International Society for Rock Mechanics, International Society for Soil Mechanics and Geotechnical Engineering, and International Association for Engineering Geology and the Environment. The aim of JTC2 is to oversee the development of an internationally agreed form of representation of geo-engineering data that can be used to store such data on the World Wide Web and transfer data between computer systems.

The GMS is investigating a potential association or compatibility with GeoScience Markup Language (GeoSciML). GeoSciML is a geology data interchange standard being developed under international collaboration through the Commission for the Management and Application of Geoscience Information, under the International Union of Geological Sciences. It began as a North American Data Model developed by the USGS, Geological Survey of Canada, and the Association of American State Geologists. A potential for merging of geologic and geotechnical data presents a promising and powerful opportunity.

SUMMARY

DIGGS provides a standardized means of geotechnical and geoenvironmental data interchange. It is a tool that can be used by geotechnical and geoenvironmental specialists to improve planning, design, construction, preservation, and overall decision making. DIGGS will allow geotechnical and geoenvironmental data to be electronically retained and geospatially referenced while providing the utility of data interchange between disparate databases and across organizational boundaries. Consequently it allows the creation of seamless management systems with free data flow between databases and software packages.

Some of the many benefits of using DIGGS include:

- Database interfacing – the ability to exchange data between different databases regardless of structure
- Software interfacing – the ability to exchange data between software using different databases or other incompatibilities
- Automatic data validation through independent software
- Eliminate the need to manually re-enter data at different points in the data usage chain

State DOTs, FHWA, and the United Kingdom Highway Agency have identified the following benefits of a DIGGS compatible Geotechnical Management System for their organizations:

- Feed information seamlessly to Asset Management Systems, such as Pavement Management Systems (PMS) and Bridge Management Systems (BMS)
- Proactively manage geotechnical assets: data, geo-structures, and geo-hazards to significantly reduce costs
- Better estimate project schedules and costs during program development
- Reduce (or eliminate) subsurface explorations by re-using their own data, and using the subsurface exploration and feature installation data collected by other organizations
- Accelerate design schedules while reducing design costs
- Mitigate schedule, cost, and safety risk during design and construction
- Improve records management
- Reduced personnel time for data search and for data entry

Additional information regarding DIGGS can be found at: **www.diggsml.org**

Rockery Design and Construction Guidelines

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ABSTRACT

Rockeries, also known as dry stack walls, consist of earth retaining and/or protection structures typically comprised of rough onsite rocks stacked in an interlocking fashion with no mortar, concrete, or steel to retain cut or fill slopes. They are context sensitive solutions that in many cases are also relatively low cost. Several rockeries exist as historic or cultural features on many Forest Highway and National Park roads. Many were apparently built in the Civilian Conservation Corps (CCC) era of the late 1930's; some are still performing well and others have required extensive maintenance or have failed.

Generally, rockeries have not been designed according to the ASSHTO standard specifications or other accepted wall design procedures. On the other hand the wide range of implementation suggests excellent performance can be expected when certain conditions are met. There is little guidance available to standard acceptable design and construction guidelines and to what conditions need to be met. A rationally based and tested design procedure therefore is needed to provide designers and owners with the confidence that these structures can be used in modern highway engineering.

Commercially, rockeries have been constructed in several northwestern states for the past four decades. As rockery design procedures tend to vary regionally, the Federal Highway Administration – Central Federal Lands Highway Division (FHWA-CFLHD) contracted Sanders & Associates to research and evaluate the existing methods by which rockeries are designed and constructed, with the eventual goal of developing comprehensive design and construction procedures. The existing design procedures were also compared using several typical rockery design cases to determine how the resulting rockery designs differ and to determine the methods most appropriate for use by the FHWA's Federal Lands Highway Divisions (FLH) as a design approach. Based on the research performed by the contractor, a design methodology was developed to evaluate rockery stability as a function of the rockery geometry (height, base width, and batter), rock properties and placement, and lateral pressure imposed by the backfill materials. Guidelines are presented for analyzing, designing and constructing rockeries, and for conducting quality assurance during rockery construction.

INTRODUCTION

A rockery, also known as, “rockery wall, dry-stack wall, stone wall, and rock wall,” is a retaining or protection structure that consists of stacked rocks without mortar, concrete, or steel reinforcement. Although the rocks are stacked in an “interlocking” pattern, there are no mechanical connections made between the individual rocks. Rather, these structures rely on the weight, size, shape, and interface friction of the rock elements to provide overall stability.

As defined herein, rockeries should not be confused with rock buttresses, rock inlays or slope armoring, each of which can have the appearance of a rockery once constructed. Rock buttresses are more massive and generally have dumped stone rather than individually placed stone. Buttresses are usually used to improve global stability of a marginally stable slope that extends far above the buttress by adding mass to the toe. Internal stability of the buttress is maintained by its thickness and a 1:1 or flatter slope angle at its face. Rock inlays are similar except they are thinner and used to control seepage and maintain face stability, primarily, and only secondarily add mass to the toe. They are used where near-surface water needs to be controlled. Slope armoring consists of individually placed stones, often only one stone thick, covering cut or fill slopes. Armoring does not provide significant improvement with respect to global stability or drainage of near surface water. Armoring provides resistance to surface erosion and vegetation. Figure 1 shows typical rockery construction.

Figure 1. Typical rockery construction.

Un-reinforced stone structures have been constructed for thousands of years in many parts of the world. In the U.S., rockeries still exist that were constructed in the late 1800s. It is doubtful these historic rockeries were “engineered” in the current sense of the term. Rockeries were also constructed along many Forest Highway and National Park roads by manual labor in the 1930s-CCC era. Many of these roads have subsequently become part of the national highway system. Little is known about the design of these rockeries, although it is suspected many were constructed with little or no engineering. Nevertheless, while some have failed, many are still in use today. These older rockeries, as well as more recent rockeries, generally need to be evaluated by the FLH for conformance with current design standards as current and future transportation needs depend on their continued usage.

Although there is evidence the public sector was building rockeries in the 1930s, the private sector appears to have been somewhat slower to adopt commercial rockery construction. Rockeries have been constructed in the Pacific Northwest for the past four decades, and have seen increasing use in northern California and Nevada over the last 10 years. Because rockeries are a relatively inexpensive engineered retaining alternative with a natural aesthetic appeal, they continue to gain popularity throughout the western United States. The FLH continues to find situations where new rockery construction would be advantageous or where repair or modification of existing historic rockeries is required. However, conventional highway design standards are not available to confirm adequate internal stability or factors of safety, even where rockeries have performed adequately for decades. Moreover, there is limited coverage of dry-stacked rockeries in engineering textbooks and literature. Although attempts have been made to develop guidelines for construction, these are typically local efforts and tend to be more procedural than analytical. For example, in 1992 the Associated Rockery Contractors (ARC) (*1*) developed construction guidelines based on local experiences in the Pacific Northwest. However, while the ARC guidelines provide general "rules-of-thumb" for use by contractors during construction, they do not provide a rational basis for design. As a result, individual designers are left to develop rockery design and construction standards based on their personal experience.

The lack of statewide or national design standards and construction guidelines has sometimes resulted in permitting and performance problems. Many municipal agencies are slow to accept rockery plans because they are unfamiliar with the design concept and/or do not have accepted guidelines to use as a basis for review. In the absence of standard guidelines, some municipal agencies require conservative designs and rockeries become prohibitively expensive. In other cases, poor rockery design and construction procedures, as well as a lack of understanding of the nature of rockeries during the review process, have resulted in poor performance, including failures during or after construction. In many cases, the lack of quality assurance or construction oversight guidelines exacerbates problems related to poor design or construction practices. For example, the failed rockery shown in Figure 2 generally has rocks of inadequate size which are poorly stacked. In addition, drainage features do not appear to have been installed.

Figure 2. Rockery failure.

The main objectives of this study are to review existing analytical methods and construction techniques currently in use and to develop a unified framework for design and specification of rockeries in modern highway construction. The ultimate goal of the project is to provide designers, inspectors, and contractors with a basis for evaluating existing rockeries and specifying and constructing new rockeries.

Based on review of available literature and the evaluation of several existing design procedures, a unified analysis and design framework was developed that can be used in modern highway engineering. The framework is rational and follows recognized engineering principles derived from analysis procedures for gravity retaining walls.

REVIEW OF EXISTING DESIGN METHODS

A comprehensive literature search indicates that the few existing analytical methods used for the design and evaluation of rockeries differ considerably. However, although the design methods vary, many of the general rules-of-thumb used to guide rockery design are relatively consistent. The various design procedures and rules-of-thumb are summarized in Table 1. The methods proposed by Gray & Sotir *(2)*, Hendron (Gifford and Kirkland) *(3)*, and Hemphill can be difficult to adapt to a rigorous design methodology, but are easily modified for evaluation of rockery stability. These methods can provide a valuable screening tool to determine if further analysis is warranted. Design charts are available for the Gray & Sotir and Hendron methods. For the design methodology to be useful, it must be adaptive to several design variables, including rockery size and height, backslope conditions, soil conditions, and applied loading. Ultimately, a closed-form analytical analysis method, such as the method proposed by SAGE, appears to be the most versatile for rockery design.

Table 1. Comparison of analytical design methods.

Design Parameter	Gray & Sotir	Gifford & Kirkland	Hemphill (1990; unpublished)	SAGE (unpublished)
Design methodology	Solve equation for height-to-base-width (H/B) ratio based on soil friction angle (φ), overturning factor of safety (FS_{OT}), and facing geometry	Solve for critical H/B ratio based on φ and slope inclination values; use poorly constructed rockery (PCR) curves	Design rockery in 1 ft (0.3 m) increments from top down; solve for FS_{OT} above finish grade at base of rockery, and sliding factor of safety (FS_{SL}) for embedded portion of rockery	Design as a gravity retaining structure; satisfy internal stability through rock placement specification; include seismic forces
Minimum factors of safety:				
Overturning (FS_{OT})	No minimum specified	1.0	1.5 to 2.5	2.0
Sliding (FS_{SL})	N/A	N/A	1.5 to 2.5	1.5
Bearing (FS_{BC})	N/A	N/A	N/A	3.0
Maximum height, ft (m)	10 (3.0)	15 (4.6)	N/A	15 (4.6)
Minimum base width	N/A	H/3	N/A	H/2
Embedment determination	Prescriptive; 12 in (300 mm) minimum	Prescriptive; 12 in (300 mm) minimum	Calculated; based on FSSL	Prescriptive; 12 in (300 mm) minimum

Table 1. Comparison of analytical design methods (continued).

Design Parameter	Gray & Sotir	Gifford & Kirkland	Hemphill (1990; unpublished)	SAGE (unpublished)
Maximum face batter	3V:1H	4V:1H	N/A	4V:1H
Maximum backslope inclination	1V:2H	Flat	N/A	1V:1.5H
Rock size designation	Width	Man rocks	Width	Width
Used for fill conditions	No	Possible	Yes	Yes
Backdrain required?	Yes	Yes	Yes	Yes
Minimum backdrain thickness, in (mm)	8 (200)	Varies; based on back cut	N/A	12 (300)
Minimum cap rock weight, lb (kg)	N/A	N/A	N/A	200 (90)
Minimum cap rock width, in (mm)	16 (400)	N/A	16 to 24 (460 to 610)	16 (400)
Rock shape	Angular	Angular	N/A	Angular
Chinking required?	N/A	Yes	Yes; voids >6 in (150 mm)	Yes; voids >6 in (150 mm)

RECOMMENDED ROCKERY DESIGN GUIDELINES

Rockeries are composed of large blocks of stacked rock, heavy enough and dimensionally adequate to form a structure that resists overturning and sliding forces. In this respect, rockeries can be treated as gravity walls, and can be analyzed rationally using modified forms of conventional gravity retaining wall design methodologies. Design of any retaining structure involves the determination of driving and resisting forces. For rockeries, driving forces include lateral earth pressures behind the rockery, surcharge pressures (both vertical and horizontal), and seismic pressures. Resisting forces can include the total weight of the rockery and individual rocks, inter-rock friction, base rock–foundation friction, and, in some cases, passive pressure at the toe of the rockery. Where Coulomb earth pressures are used, the vertical component of the active earth pressure can also aid in stabilizing the rockery. A typical rockery section is shown in Figure 3, along with the design parameters and dimensions that must be determined prior to rockery design.

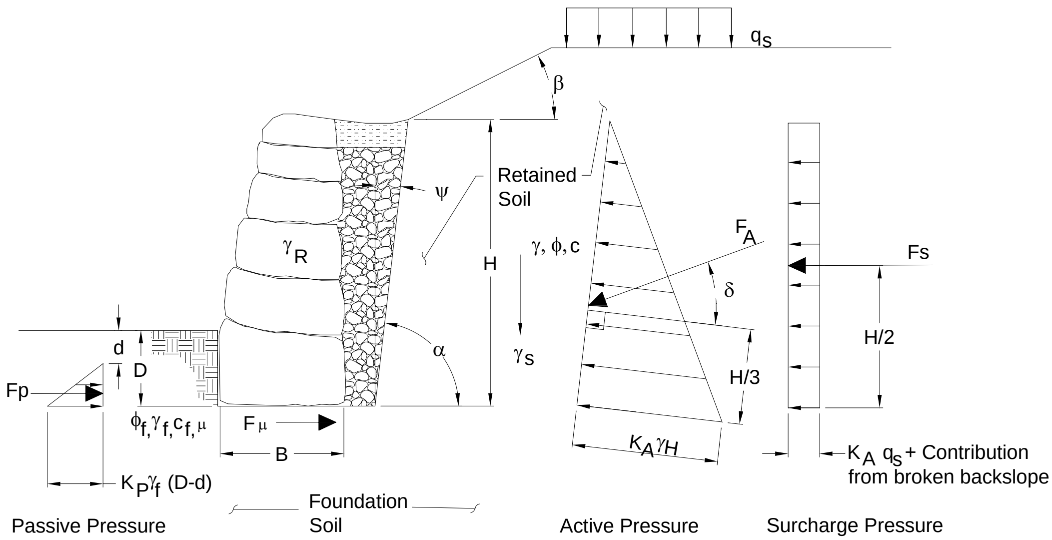

Figure 3. Schematic rockery section showing critical dimensions and parameters to be determined for design.

Design Parameters

For design, the following geotechnical parameters are required: 1) friction angle (Φ), unit weight (γ_s) and cohesion (c) for both retaining and foundation soils, 2) interface friction angle (δ) typically on the order 2/3Φ to Φ, 3) allowable back cut angle (Ψ), 4) Unit weight for rock (γ_R)); typically assumed to be 150 pcf (23.5 kN/m^3), including void space, 5) minimum required embedment depth (D), 6) allowable bearing pressure and estimated settlement due to the weight of the rockery, and the following crushed rock properties:

- The friction angle of the crushed rock (Φ_{CR}) is typically on the order of 40º to 45º, and is generally much higher than the soil (Φ=30º to 36º). This results in a smaller value of K_A.
- The crushed rock typically has a lower unit weight than the retained soil due to the increased void space. Typically, γ_{CR} = 105 pcf (16.7 kN/m^3), and γ_S = 120 pcf (18.6 kN/m^3).
- The crushed rock layer is generally relatively narrow, on the order of 12 in (300 mm) thick. As a result, the active failure wedge typically extends through the crushed rock and into the retained soil behind the crushed rock.

For rockery design, the theoretical failure plane crosses through two soil types (crushed rock and retained soil) and a compound failure wedge is developed. While it is feasible to develop closed-form equations for this condition, acceptable results can be obtained by making the simplifying assumption that the crushed rock is part of the rockery system and the lateral earth pressure is developed solely by the retained soil. Therefore, the lateral earth pressure acts on the back of the crushed rock layer at the crushed rock/slope interface rather than the back of the rockery facing elements. Because the friction angle of the crushed rock is almost always greater than that of the retained soil, this simplifying assumption is usually conservative.

Sliding Resistance

Rockeries generally resist sliding primarily through friction along the bottom of the base rock, which is a function of the normal force acting on the base of the rockery and the coefficient of sliding between the base rock and foundation soil. The normal force consists of the vertical component of the Coulomb active earth pressure ($F_{A,V}$, acting downward) and the weight of the rockery. The weight of the rockery can be estimated by assuming certain minimum dimensions for the rockery, breaking the rockery into a few easy to define geometric shapes, assuming a unit weight for the rockery mass, and computing the total weight as the sum of each component. The unit weight of the individual, sound, intact rocks is about 165 pcf (25.9 kN/m^3), which corresponds to a specific gravity of about 2.65. However, once the voids in the rockery are considered, a reasonable unit weight for a well-constructed rockery is about 150 pcf (23.6 kN/m^3).

Figure 4 shows a schematic of a rockery that has been divided into three sections for the computation of the rockery weight. Although the lateral earth pressures are assumed to act on the back of the crushed rock behind the rockery, the weight of the crushed rock is typically not included as a resisting force. Because the crushed rock is not physically connected to the back of the rockery and the facing rocks and crushed rock interact only through frictional contact, it is not clear that the weight of the crushed rock would provided a significant resistance to movement, particularly overturning. Therefore, the weight of the crushed rock is conservatively neglected. After the design is complete, the final rockery dimensions should be checked to verify the assumed geometry and weight are correct.

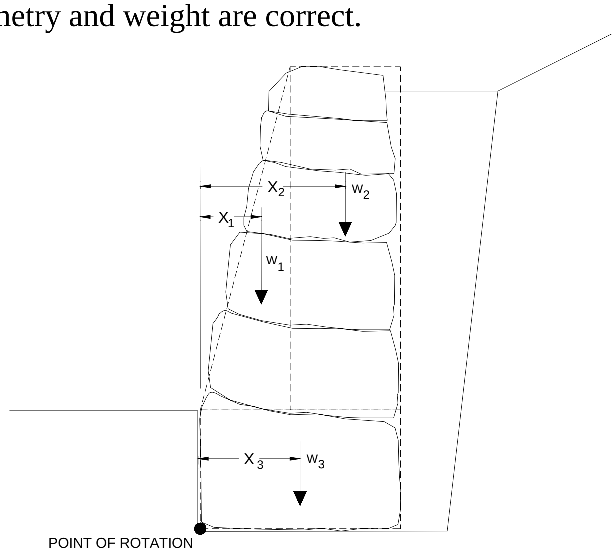

Figure 4. Estimation of rockery weight and centroidal distances.

The friction along the bottom of the base rock is computed by multiplying the friction factor for sliding between the rock and the foundation soil (μ) by the sum of the vertical forces acting on the base of the rockery. Typical ultimate (unfactored) values of μ for some common materials are listed in Table 2 *(4)*. For the design procedures presented in this paper, a static factor of safety of 1.5 and a seismic factor of safety of 1.1 are recommended for sliding.

Table 2. Typical friction factors for determination of FSSL.

Base Rock Texture	Foundation Material	Estimated Ultimate Friction Factor, μ
Rough	Dense, medium-grained sand Φ=36º	0.7
Smooth, angular rocks with flat faces	Stiff silt or clay Φ=30º	0.4
Rough	Moderately weathered bedrock Φ =36º	0.6
Rough	12 in (300 mm) thick layer of crushed rock Φ=40º	0.8
Smooth, angular rocks with flat faces	12 in (300 mm) thick layer of "foundation fill" with 100% passing 2 in (50 mm) sieve, 6% maximum passing No. 200 (0.075 mm) sieve Φ=35º	0.7

Overturning

The horizontal forces include the horizontal component of the lateral earth pressure ($F_{A,H}$) and the additional horizontal pressure due to a vertical surface surcharge (FS) will also tend to cause the rockery to tip forward about its toe The overturning moments caused by these forces are counterbalanced by resisting moments due to the weight of the rockery (W), the vertical component of the lateral earth pressure ($F_{A,V}$), and the passive resistance at the toe of the rockery (F_P). The overturning and resisting moments are computed by summing moments about the toe of the rockery. The total resisting moment due to the weight of the rockery is computed for each component of the rockery weight (W_i), as shown in Figure 4, multiplied by the horizontal distance from the centroid of each rockery segment to the toe of the rockery (x_i) A minimum factor of static safety against overturning of 2.0 and seismic factor of safety of 1.5 are recommended

Inter-rock sliding is similar to external sliding, except that the total weight is only computed for the rocks above the place of sliding as shown in Figure 5. For rock-to-rock friction, a nominal value for μ of 0.55 is recommended. Higher values can be used where additional data, such as high rock roughness or laboratory testing, indicates a higher value is warranted. The lowest factor of safety calculated should be used in the design.

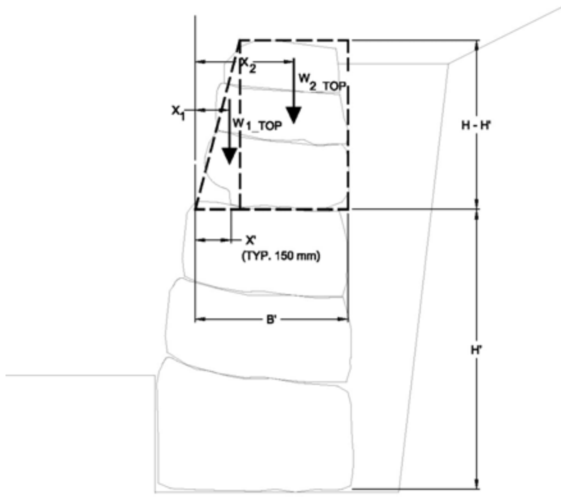

Figure 5. Geometric relationships for determination of internal stability.

Bearing Capacity

The final aspect of static design to be checked is the bearing capacity of the foundation soils. The allowable bearing capacity can be determined in accordance with Section 4.4 of the AASHTO Standard Specifications for Highway Bridges, 17th Edition – 2002 *(5)*. A minimum Static factor of safety of 2.5 and seismic factor of safety of 1.5 are recommended.

Global Stability

In some cases, the overall rockery design may be controlled by global stability considerations. This is especially true for cuts in previously placed fills or for walls with a sloping toe condition. The purpose of a global stability analysis is to check that the rockery or retained improvements will not be damaged by a slope stability failure through or below the wall facing.

Global stability analyses can be performed using most commercially available limit equilibrium slope stability programs. For static slope stability analyses, a minimum factor of safety of 1.5 is typically considered. For highway projects, it may be feasible to lower this factor of safety to 1.3; this determination can be made on a case-by-case basis by the Geotechnical Engineer.

Wherever global stability is checked for static conditions it should also be checked for seismic conditions. A minimum factor of safety of 1.1 should be used for seismic conditions. Depending on the results of the seismic slope stability analysis, a deformation analysis may also be required to check that estimated upslope movements are acceptable where upslope improvements exist or are planned.

ROCKERY CONSTRUCTION GUIDELINES

For most civil works, the performance of a structure is directly related to the quality of construction. For a rockery, this concept is magnified several times by the fact that rockeries are constructed from irregularly shaped, naturally occurring materials. Therefore, the skill of the contractor constructing the rockery has a large impact on the overall performance

Excavation

The foundation excavation should be sufficiently wide to permit placement of the specified leveling course. For level toe conditions, nominal embedment of 12 in (300 mm) is generally sufficient unless frost considerations apply, in which case the rockery should be founded below the zone of frost heave. The back cut inclination should be of a value that is consistent with the expected soil and rock conditions as well as recognized safety regulations, such as OSHA. The back cut is also is a specified input parameter in the design procedure, and, therefore, the assumed inclination of the back cut should be clearly stated on the plans. If the back cut must be laid back during construction changes in the lateral earth pressure on the rockery could occur, and these changes may be conservative or unconservative. For example, if the back cut is laid back at a shallower angle than anticipated in the design a larger volume of crushed rock will be required to fill the space behind the rockery which results in a reduction of the lateral earth pressure imposed on the rockery, which would be conservative. However, if relatively low lateral earth pressures were used during design, such as for a cut in bedrock, the lateral earth pressure imposed by the crushed rock may be higher than assumed during design, which would be an unconservative change.

Rock Placement

Proper placement of the rocks comprising the rockery requires skill and experience because of the irregular and non-uniform nature of the materials involved. Some rocks only fit in some places and not others, and finding the proper match between rocks to form a stable structure can be a trial-and-error process even if the operator is highly experienced.

Base rocks should be placed on a properly prepared foundation excavation, as discussed previously. The minimum base rock width, B, should be specified on the plans and should be based on overall rockery height, retained soil properties, and any surcharge loads. All rocks, including the base rocks, should be placed with the longest rock dimension perpendicular to the face of the rockery. The second largest dimension should be parallel to the layout line of the rockery, and the smallest rock dimension should be its vertical dimension. The base rocks should be placed such that the tops of the rock are sloped back at least 5% towards the back of the rockery. The allowable tolerance for base rock widths should be 6 in (150 mm).

Because the rocks must be “finessed” into proper interlocking positions, the use of proper equipment for rock placement can be the difference between a successful and unsuccessful project. An excavator with a rotating clamshell attachment are useful for properly placing rocks, as shown in Figure 6. The clamshell allows the rock to be grasped uniformly on two sides, and the powered rotation capability allows the operator to quickly make adjustments to the rock orientation and alignment. In addition, a clamshell with rotation capability also allows one rock to be placed and replaced at multiple locations to determine the best fit without the need to move the excavator or regrasp the rock. An excavator with a rotating clamshell should be specified in the plans, as it improves rockery construction and reduces time of installation.

Figure 6. Hydraulic excavator with a clamshell constructing a rockery.

Successive lifts of facing rocks should be placed above the base rocks in accordance with the design schedule. In general, the width of successive rows of facing rocks will be determined based on the design rockery face batter, which will generally vary between 4V:1H and 6V:1H. Each rock should be placed according to the following; 1) each rock should bear on at least two other rocks, 2) each rock should have at least three bearing points—two in front and one in back, 3) the front-most bearing points for each rock should be within 6 in (150 mm) of the average face of the rockery, 4) the rear of the rocks should be aligned along an imaginary vertical plane. If rocks larger than the minimum specified B are used, they can extend beyond this imaginary plane provided they do not interfere with rockery drainage, and 5) the tops of each rock should be sloped back towards the backdrain as previously described for the base rock.

An example of a relatively well constructed rockery is shown in Figure 7. Although a few vertical seams can be located, the rocks are generally bearing at the proper locations and stacking in an approximate running bond pattern.

Figure 7. Example of a well constructed rockery.

Crushed Rock Zone

The crushed rock zone behind the rockery provides drainage, improves overall rockery stability by providing a high strength material behind the facing rocks, and reduces the overall soil pressure on the rockery system.

The crushed rock should consist of 4 to 6 in (100 to 150 mm), crushed, and screened, angular rock. This material is often called “quarry spalls,” and should meet the gradation requirements presented in Table 3.

Table 3. Gradation requirements for crushed rock backdrain.

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)
6 in (150 mm)	100
4 in (100 mm)	0.0 – 25
3/4 in (19.0 mm)	0.0 – 15
No. 4 (4.75 mm)	0.0 – 5.0
No. 200 (0.075 mm)	0.0 – 2.0

The crushed rock should fill be void between the back cut and the rear of the facing rocks; however, it should be 12 in (300 mm) wide as a minimum. The crushed rock should be capped by at least 12 in (300 mm) of impermeable soil at the ground surface to prevent infiltration of surface water behind the rockery.

Drainage System

As the base and facing rocks are placed, it is generally most convenient to construct the rockery drain and crushed rock zone concurrently. The drain pipe should generally consist of a 4 in (100 mm) diameter perforated drain pipe surrounded on all sides by at least 4 in (100 mm) of screened, 4- to 6-in (100 to 150 mm), angular crushed rock unless unusual conditions exist as determined by the Geotechnical Engineer. The drain pipe should consist of either corrugated high-density polyethylene (HDPE) pipe or smooth polyvinyl chloride (PVC) pipe. The pipe should be placed with the perforations down. A diagram of the drain components is presented in Figure 8 and photographs of installed crushed rock backdrains are presented in Figure 9.

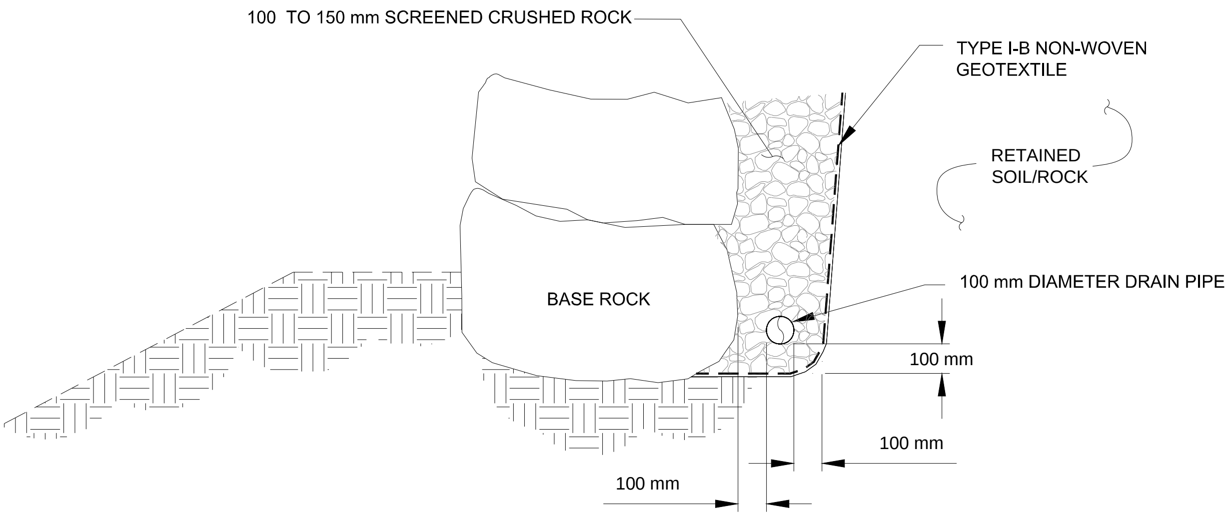

Figure 8. Backdrain components (partial section).

Figure 9. Placement of drain blanket and non-woven geotextile behind rockery.

Where tiered rockeries are constructed, drainage of the upper tier is an important detailing consideration. Typically, it is difficult to outlet the backdrain directly from the upper tier because it is typically located mid-slope. In these circumstances, the perforated drain pipe can be tied into solid discharge pipes and directed downslope. The solid pipes should be sloped to the back of the lower tier, taken down the back of the lower rockery, and outlet at a similar location as the lower tier drainage

In addition to subsurface water, surface water must also be controlled. To prevent a hydraulic connection between the rockery backdrain and surface water flows, the top of the crushed rock should be capped with at least 12 in (300 mm) of “impermeable” soil over non-woven geotextile. This soil cap can generally consist of on-site soils and should be “impermeable” to the extent that rapid infiltration of surface water cannot occur

As with any structure that retains soil or rock, surface water should also be directed away from the rockery where possible. Where the rockery is constructed at the toe of a slope or on a slope, a v-ditch consisting of concrete or impermeable soil should be constructed immediately behind the rockery to direct surface water to a suitable drainage outlet, as shown in Figure 10. In rare circumstances, the surface water drain system can be designed to allow “minor” surface water to enter the backdrain provided the drainage system is sized for the increased flow. Because it is difficult to limit surface water to “minor” amounts, this practice is generally not recommended.

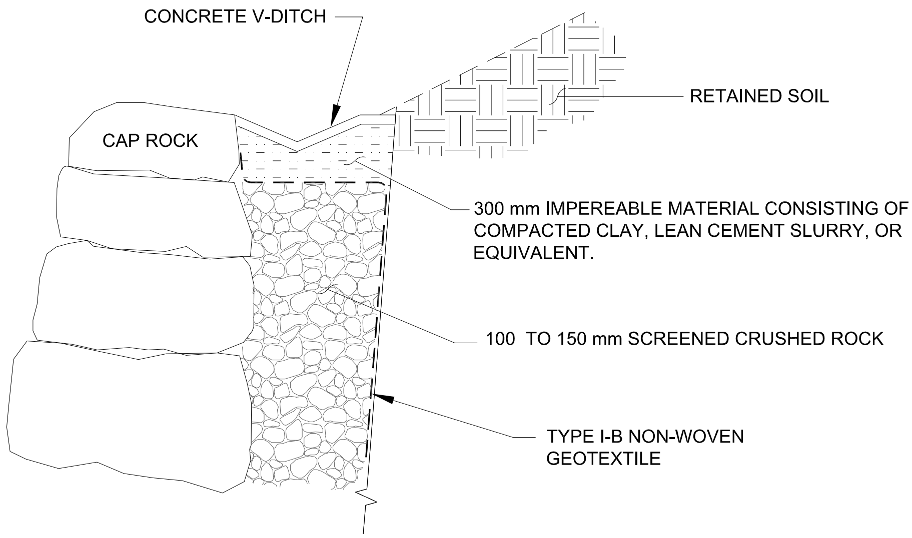

Figure 10. Graphic. V-Ditch and impermeable cap at top of rockery (partial section).

Cap Rock

The final rock placed at the top of the rockery is the cap rock. Because the cap rock provides a finished look to the top of the rockery, it is generally smaller and flatter than the facing rocks. To reduce the risk of disturbance to the cap rocks, such as by vandals or rock climbers, cap rocks should weigh at least 200 lb (90 kg). In addition, cap rocks should not be movable by hand. Cap rocks that do not meet these minimum requirements should be grouted or glued in place to prevent accidental dislodging. Particular care should be taken when placing and sizing cap rocks for rockeries with toe slopes. If improvements are located at the base of the slope, dislodged rocks could roll down the slope and pose a significant hazard. Where this condition occurs, consideration should be given to securely grouting all cap rocks regardless of size.

Chinking Rocks

Because of the irregular nature of the rocks, it is difficult to ensure that every rock conforms to the shape of all adjacent rocks. As a result, gaps will occur between rocks. Where these gaps exceed 6 in (150 mm), they should be filled with chinking rocks consisting of spalls from the parent (facing) rock. The purpose of the chinking rocks is to improve aesthetics and prevent the screened backdrain material from falling out through the face of the rockery. Chinking rocks should not be movable by hand, and can be grouted in place if necessary. In addition, chinking rocks are not to provide primary support for overlying rocks.

SUMMARY

In summary the design of a rockery that resists static and seismic earth pressures and lateral pressure surcharges is analogous to the design of a gravity retaining wall. The lateral pressures acting on the back of the rockery should be determined, and the rockery checked for an adequate factor of safety against sliding, overturning, and bearing capacity failure. Common experience is that rockeries are constructed up to12 ft (3.7 m) tall in fill conditions without mechanically stabilized earth (MSE) reinforcement, and up to 15 ft (4.6 m) as a facing for an MSE fill. Regardless, a maximum single-tier height of 15 ft (4.6 m) should be used in cut or fill conditions.

Limits on tier heights, and the recommended inclusion of MSE for higher heights are because of the difference between the theory of a stable rockery and how difficult it is to build one in practice. More so than for other types of retaining walls, the stability and longevity of a rockery are controlled by the construction practice and the natural shape of the stones that are practically available. In effect, the limited heights and inclusion of MSE limit the potential consequences of failure of one or more of the stacked rocks. Another way to reduce the risk, which is recommended, is to designate certain contractor staff to the construction and certain owner/engineer inspectors to the observation of the construction work. At best, this can reduce the likelihood of failure to a level that is believed to be generally consistent with other, AASHTO designed retaining walls. Because this is not always achievable, it is recommended that rockeries be used where the consequence of failure is not excessive, such as for some lower volume roads, cuts requiring only nominal support (for example, short-term stability is sufficient to build the rockery without shoring), and where there is great benefit in cost and aesthetics for doing so.

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Rock Slope Engineering –

The Industry Dictates the Approach

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ABSTRACT

There are significant differences in how the geological engineering practitioner approaches rock slope design, depending on industry catered to. The differences are dictated more by economics than by training. Practitioners employed in the mining industry design open pit mine slopes with the primary intent of optimizing ore extraction while minimizing risk to critical mine infrastructure. Practitioners in the transportation industry design highway rock slopes with risk to public safety and commerce in mind, usually with quite limited funding. Practitioners in the building/development industry also tend to work with limited funding and are often pushed into making high risk decisions based upon limited engineering geologic information. Although the concepts of "risk" and "value" are shared by the respective industries, each industry's perception of "risk" and what objects possess tangible "value" are often quite different. As a result, there are differing views on what constitutes a comprehensive rock slope site investigation and design program. Convincing industry managers to increase expenditures on site investigations and quality design is, at best, a difficult task when "cost" and "risk" cannot be reliably quantified. On one hand, mine managers are quite willing to justify long-term, rigorous engineering geologic site investigations when the cost is offset by relatively large profits. On the other hand, practitioners within the transportation industry must struggle with the trade-offs between overall project costs and potential impacts on public safety. The industry-dependent perceptions of "risk" and "cost", as related to rock mass characterization and rock slope design, are highlighted by specific examples at an open pit copper mine in Nevada and a condominium development in Connecticut.

INTRODUCTION

There are significant industry-dependent differences in how the engineering geologist, geological engineer, and geotechnical engineer (herein referred to as "practitioner") approach rock slope engineering. These differences span the globe and are not unique to one country or region. However, the differences can be easily seen by any practitioner who designs or has designed rock slopes within three specific market sectors, those being mining/resource development, public infrastructure, and real estate/development.

Variations in the approaches adopted by the rock slope engineering practitioner can best be realized when specific contributory factors are considered. These market-dependent factors, which could be many in number, are generalized for the purposes of this paper. In addition, it is thought that most the industry-dependent differences can be attributed to these market factors in some fashion. The primary factors that account for the variations in rock slope engineering approach, as cited in this paper, include: (1) economics; (2) hazard and risk perception; (3) performance expectations, all of which are related. The remainder of this paper is based on the author's experiences while practicing within the stated market sectors.

It is noteworthy that some industry-dependent differences also exist due to the varied educational backgrounds of those who practice rock slope design. However, it is generally safe to assume that even educational upbringing is geared toward a particular market sector or set of market-related activities (e.g. mining engineering vs. civil engineering). In light of the preceding, industry-dependent differences in rock slope design due to educational bias will be left for future scrutiny.

PRIMARY FACTORS INFLUECING ROCK SLOPE DATA COLLECTION AND DESIGN APPROACH

Economics

Economic factors instinctively influence all market sectors, however the incorporation of economic factors within the decision making process varies between the three stated market sectors.

Mining & Resource Development

Decision makers within the mining industry tend to rely on commodity prices, ore-body distribution, and resource valuation models during their initial (pre-feasibility) assessment of expected profit. This approach is markedly different from other market sectors in that decisions are often iterative in nature, with decisions made "today" being based on projections for "tomorrow". For example, expected profit is based on projected minimum production rates. If production rates are not met or exceeded, profit is subsequently decreased. The same logic holds for resource allocation during pit slope design studies. The preliminary configuration of the pit shell will undoubtedly morph with time, as new geotechnical and economic geologic information is obtained. In theory, mine planners wish to construct pit slopes as steep as possible in order to keep stripping ratios low. The rock slope engineering practitioner is tasked with tempering the mine planner's expectations with what is geotechnically reasonable. The end result is a pit wall configuration in a state of constant flux, as the mine planner presents what is expected, the rock slope engineer presents what is geotechnically feasible, and the production manager dictates what is achievable given the technology (Ref 1). All of the preceding decisions are based on expectations or projections. The amount of money spent up-front on geotechnical investigation and subsequent pit slope design is directly correlated with "current" projections of expected profit (i.e. projections change with time). If, for example, commodity prices drop for the long-term or the ore deposit is thought to be unsustainable given the mining and extraction methods, funds for geotechnical exploration and design are minimized. On the contrary, when a deposit is thought to be viable for the long-term or the ore grades high, mine planners allocate more funding for rock slope engineering services. All economic expenditures are a function of projected ore recovery and profitability.

Public Infrastructure

Allocation of funds for public infrastructure and capital improvement projects, most notably in transportation, is in stark contrast to that of mining. Funding is usually derived from public coffers at various levels of government (e.g. federal, state, and local). Highway improvement funding is usually supplied through a combination of federal highway grants and state executive (or legislative) allocation to a respective department of transportation (DOT). The money spent on highway improvements is generally based on a set of priorities, some of which may seem arbitrary at times. Many state DOT's have the reputation for being "reactionary" in nature, which can be argued, results from a pre-determined set of prioritized funding initiatives. Capital improvement funding is doled out according to perceived "need" and is often times political in its requisition.

Real Estate & Land Development

The real estate and land development industry operates on investments or re-distributed profits. The author's experience has shown that most decision makers in this industry tend to "brush-off" the financial impact that engineering geologic conditions may have on their projects. As a result of this philosophy, high risk rock slopes have been and are being constructed around the nation, particularly in the northeastern United States. The author has coined these types of rock slopes "design after the fact" slopes. Due to industry wide perceptions, design emphasis is placed on buildings, garages, and pool as engineered structures, while constructed slopes are not viewed in the same light. In fact, long-term costs have the potential to be higher based on any resulting liabilities and rock slope rehabilitation measures. Economic incentives are measured only through unit sales, and economic disincentives (e.g. risk to life and property, slope rehabilitation costs) are rarely assessed up-front.

Hazard and Risk Perception

All three market sectors inevitably weight risk with the lifetime of the intended structure. However, the concept of acceptable risk and incorporation of risk are very different between mining and that of public infrastructure and real estate development. Although hazard types may be similar, the quantification of risk within the three market sectors is very much dependent upon the design service period and the frequency of exposure.

Mining & Resource Development

Mines generally have shorter service periods than housing units, and housing units generally have shorter duration service periods than public highways. Most mines are also designed as temporary structures; hence the adage “if the slope is stable one day after the deposit is mined-out, it’s over-designed”. In light of a mine’s shorter duration and temporary “lifespan”, mine planners have to weigh the inevitable hazards with the risks posed to life, property, and ore extraction. For example, rockfall occurrence is expected at most open pit mining operations. The author has walked many pit slope benches and has seen rockfall occurring at all times of the operating day. The impact of rockfall on a haul truck operator can be minimized depending on costs of remedial measures and most importantly, risk to operations. Risk mitigation is ranked according to impact on operations, as operations affect cash flow. Mine planners generally accept that certain risks exist, and implement cost-effective strategies to deal with these risks depending on overall impact to operations. The measurement parameters are risk to mine operations and long-term profitability.

Public Infrastructure

In stark contrast to the mining industry, decision makers within the public transportation arena are faced with much longer duration service periods and higher exposure limits. For example, I-95 outside New York City virtually has continuous vehicular flow most times of the day. Although acceptance of rockfall occurrence may well be tolerated, impacts on the traveling public are not. In light of the preceding, risks to the traveling public posed by rockfall are managed through costly, hardened stabilization measures. The measurement parameter is risk to the public.

Real Estate & Land Development

In similar fashion to the mining and resource development market, decision makers in the real estate industry tend to view risk in purely monetary terms. However, in contrast to both public infrastructure and mining, the risk is frequently assessed after a project is complete. In the author’s experience, rarely is enough engineering geologic information collected up-front that would preclude higher risks in the long-term. Hazards and subsequent risks often develop based on a lack of reliable baseline geologic information coupled with input from inexperienced, low cost contractors. In the real estate and land development industry, the measurement parameter is perceived risk to long-term profits.

This perceived risk is handled by minimizing expenditures on engineering geologic site investigations.

Performance Expectations

Although each market sector has similarities in rock slope performance expectations, all three exhibit markedly different approaches in their efforts to achieve an “as-designed” condition.

Mining & Resource Development

Mining managers, in similar fashion to the other two market segments, expect slopes to behave as-designed for the lifetime of the mine. However, decision makers understand the geotechnically complex nature of the large-scale operation(s) they are undertaking. For example, they expect that there will be frequent rockfall events, that some back-break is inevitable, and that there is always the possibility of large-scale, rock mass failure. Their subsequent approach is typically to evaluate, adapt, monitor, and work around such failures – not necessarily prevent them from occurring.

Public Infrastructure

Managers within the public infrastructure market expect rock slopes to present minimal risks to the traveling public. In similar fashion to mining, frequent rockfall events can be tolerated, but in this case severely restricted in their ability to impact the roadway. Most (but not all) DOT managers expect that some quantity of long-term monitoring and maintenance is required for rock slopes that are within the highway limits. Some DOT managers go as far as to keep in-depth rock slope inventories to monitor, rank, manage, and allocate, while others take a “wait and see” approach. Rock slope performance expectations within this industry are relatively aligned, but measures taken to ensure that these expectations become reality are far from uniform.

Real Estate & Land Development

Decision makers within the real estate and land development industry, particularly in the northeastern U.S., are often misinformed with regard to the impact engineering geologic conditions can have on their bottom line. With this in mind, their slope performance expectations can sometimes be construed as unrealistic. Performance expectations are often pervaded with the “rock is rock” mentality, sometimes being true, but more often not. These minimum rock slope performance expectations are unfairly biased by low-cost, inexperienced contractors and ill-equipped design professionals. The coexistence of wealth with lack of “easily developable” land may also be increasing the frequency of these sorts of situations. The overall performance expectations, although not that unreasonable, are most times not tempered by engineering geologic reality.

Synergy Between the Factors

In reality, it is frequently observed that the three aforementioned factors co-exist and are not mutually exclusive of one-another. Risk and cost are intrinsically related, as increased costs are associated with minimized risk. It is reasonable to make the assertion that rock slope performance expectations are a product of equitably managed cost and risk factors, assessed from the beginning to the end of the project. This concept can be applied to the entire range of rock slope engineering services, from engineering geologic site investigation to final design to long-term maintenance. However, the main thesis of this paper is founded on how the equitable management of risk and cost in rock slope engineering practice does not cross market boundaries. Surely there will be differences in approach based on financial backing, size of project, and intended application. By the same token, it can (and should) be argued that basic engineering geologic information and minimum performance criteria is required for all rock slope design projects, irrespective of size and economic constraints. The remainder of this paper highlights example projects within the three market sectors that show how differences in perceived cost and risk are manifested within rock slope engineering practice.

DIFFERENCES IN ROCK SLOPE DESIGN BY MARKET SECTOR DUE TO PRIMARY FACTORS – EXAMPLES

Mining & Resource Development – Example at an Open Pit Mine Slope

Rock slope design in open pit mines is somewhat of a misnomer in that the practitioner is actually engaged in open pit mine design. The rock slope design is but one part of the big picture – ore recovery. The practitioner quickly becomes aware of the implications of pit slope configuration on production and profitability.

The author spent approximately two months at a historic open pit porphyry copper mine in central-eastern Nevada in 2004. Although the mine will remain confidential, it had the notoriety of being one of the most geotechnically “active” in North America, if not the world. Upwards of 10 individual open pits existed on the site, with two being actively mined. The entire project had recently been purchased by a “junior” mining company, well aware of the complex geotechnical conditions that existed at the site. The intent was to expand the existing open pit mining operation, as the price of copper had recently spiked. One of the pits slated for expansion was unique in that any rock slope engineering considerations had to be based not only on risks to mining operations, life, and equipment, but also on the process of production itself. One of the very ore blocks comprising the primary deposit was part of a large creeping slide mass, measuring some 600-ft in height by 300-ft in slope-parallel width (Photos 1 and 2).

Photo 1 – Open pit copper mine with failing pit slope (purple box)

The slide was deemed as “complex” in that there were multiple slide planes with more than one failure mechanism (components of structural wedge and rock mass failure). As could be imagined, mine planners were intricately involved with attempts to manage ore block displacement. The proposed mining methods and mining rates would have had definite impact on slide mass creep rates.

- drill and install deep, low-angle, slope depressurization holes at the bottom third of the slide mass;
- periodically pump far-field water production wells (additional depressurization);
- manage surface water run-off;
- manage the local mining production rate;
- manage geometric distribution of weight on the pit wall;
- establish a temporarily non-exploitable, 100-ft wide bench (similar to toe buttress) at the base of the slide mass for increased passive resistance;
- install automated total stations (survey instrument) to measure x, y, z components of pit wall creep during mining operations.

Photo 2 –Failing slope at open pit copper mine. Note tension cracks.

Costs for the investigation, design, and long-term monitoring were modest (<$80,000) in comparison to potential losses due to inadequate pit wall management controls. Mine planners were relatively successful in assessing the slope-related risks posed to operations and in subsequently implementing sound, cost effective, engineering strategies to manage future risk.

In this situation, mine planners had a healthy respect for the impact that engineering geologic conditions would have on operations. This respect was translated into risk management strategies by investing up-front in a sound rock mass characterization program. But why then, did the previous mine owner sell the operation? Why did decision makers, who full-well knew that some from of instability existed, opt out of such a characterization program? Their decision to pull-out of the venture was rooted in low commodity prices. Simply put, the cost to manage and implement effective risk management strategies was not offset by economic incentives.

Real Estate & Land Development – Example of a Development Rock Slope

The author was recently involved in litigation support for a condominium rock slope in southwestern Connecticut (to once again remain unnamed). The rock slope was the end result of mass rock excavation activities for construction of a condominium complex. A local excavation contractor was hired for rock excavation and removal, with minimal input from rock slope engineering professionals. Three separate geotechnical reports were created, each with differing recommendations with regard to rock slope design. The geotechnical information provided for rock slope design was scant. The case developed as a result of recent rockfall events and the close proximity of the slope to existing structures (Photo 3).

Photo 3 – Development rock slope in close proximity to existing structures

The plaintiffs (developer and condominium association) filed suit against the contractor, at which point the contractor counter-sued the engineer-of-record. The case became complex, with multiple parties being involved. An attorney representing one of the defendants (engineer) in the case hired Haley & Aldrich as expert witnesses for purposes of dispute resolution. The intent was to preclude trial by holding formal mediation proceedings between all parties involved in the case. Haley & Aldrich was tasked with creation of a "conceptual design plan" for rehabilitation cost estimation purposes. During the course of the proceedings, the author was asked the following questions:

- "What actions could have been taken before or during construction that would have minimized the severity of the current problems?"
- "What party do you believe bears the brunt of the blame?"

Both questions are valid in scope, but not necessarily straight-forward in their response. Most engineering geologists could write a treatise based on these two questions alone. After careful inspection of the slope and review of the discovery evidence, it became apparent that there were multiple failures, and at all levels of the project. Why is this generally the case within this specific industry? Why are developers exposing themselves to this degree? The answers to these questions were found deeply rooted within this very case. The primary causative factors responsible for the development of this case were:

- an inadequate engineering geologic investigation;
- inexperienced geotechnical professionals;
- inexperienced, low cost contractors;
- uninformed decision makers on the owner's behalf

All four preceding points are unfortunately found to co-exist within much of the real estate development sector with regard to construction in rock. The reasons for this are fairly straight-forward. Inexperienced geotechnical professionals tend to conduct inadequate engineering geologic site investigations. This is not to say they have malice for their client, but rather they lack the relevant training and experience. In addition, owners tend to want to save money up-front by hiring more "affordable" engineers and contractors. In the long-run, the decisions made were anything but "affordable". In the case cited above, a mere $20,000 that could have been invested in a robust site investigation and reasonable design eventually ballooned into a $400,000 settlement.

CONCLUDING REMARKS

It is found that managers within the mining industry have an understanding of the geological complexities inherent with their duties. As a result, they hire highly trained geotechnical specialists who are capable of conducting rigorous investigations and detailed analysis. When commodity prices fall, they cut back on geotechnical input and knowingly accept more risk. This is in stark contrast to managers within the real estate development industry, who are instinctively attracted to less costly geotechnical service providers. These two industries represent the extremes within the rock slope engineering clientele base, as manifested by the performance of their rock slopes. Transportation rock slopes fall somewhere between these two extremes, depending on specific state.

All owners, irrespective of industry, have reasonable expectations for adequate information, as they hire contractors and consultants under the assumption that they possess relevant knowledge and real-world experience. In this regard, the owner is at the mercy of the expert(s) he/she hires. However, it is ultimately each and every consumer's responsibility to research the product they purchase – this does not exclude the owner.

The trade-offs between cost and risk cannot be rationally assessed if the hazards are not identified. Hazards cannot be reliably identified, if identified at all, by "low-cost" geotechnical professionals. Quality workmanship cannot be always expected from the "cheapest" contractors. The finished condition of any slope is the combined result of engineering design and quality of workmanship. When either element is lacking, long-term performance expectations should be correspondingly low.

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Extreme Karst:

Investigation at PA State Route 33

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Pennsylvania Department of Transportation

October 2006

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Disclaimer

Any opinions expressed in this paper are strictly those of the author, and do not necessarily reflect positions held by those acknowledged above, or the Pennsylvania Department of Transportation.

Abstract

In January 2004, a sinkhole opened beneath the northbound span of PA State Route 33 over the Bushkill Creek. Both the northbound and southbound spans eventually had to be replaced. Shortly after completion, both new spans indicated signs of movement and distress. As a result a deep investigation of the karst conditions was initiated. The investigation included deep borings (over 500 feet), and a comprehensive hydrogeologic study. Water quality soundings, including continuous temperature and conductivity readings, were conducted for every boring. A deep (over 400 feet) heavily karstified zone was found to lie beneath the creek, in the area of SR33. High groundwater velocities were suspected due to a combination of: 1) problems experienced during grouting of micropiles for the replacement structure foundations, 2) the presence of large sinkholes in the creek just downstream of SR33, and 3) high pumping requirements of quarry not far upstream. It was further suspected that these high groundwater velocities were a result of a conduit like connection through the karst, between the stream and the quarry. This condition was thought to be resulting in the slow erosion of subsurface soil supporting the structure foundations A brine tracer study was conducted to confirm the suspected flow pattern and high groundwater velocities. Based upon the findings of the investigation, several options were developed to stabilize the foundations. The options were found to either be very costly, to have a low prospect for success, or result in other highly undesirable consequences. The structures will continued to be monitored, and further subsurface investigation will be conducted. It is intended to delineate the limits of the highly karstified zone, so as to be prepared should monitoring indicate that the existing structures may become unserviceable.

Introduction

State Route 33(SR33) is located in eastern Pennsylvania, northeast of Allentown. Twin Spans carry SR33 over the Bushkill Creek in Northampton County, Stockertown (see Figure #1). In January 2004 a sinkhole formed in the creek under one of the piers of the northbound bridge. The pier settled and the northbound highway was closed. Attempts at stabilizing the situation were unsuccessful and the bridge had to be dismantled under emergency contract, so that sudden catastrophic failure would not damage the SB structure. The pier settled 28 inches prior to demolition.

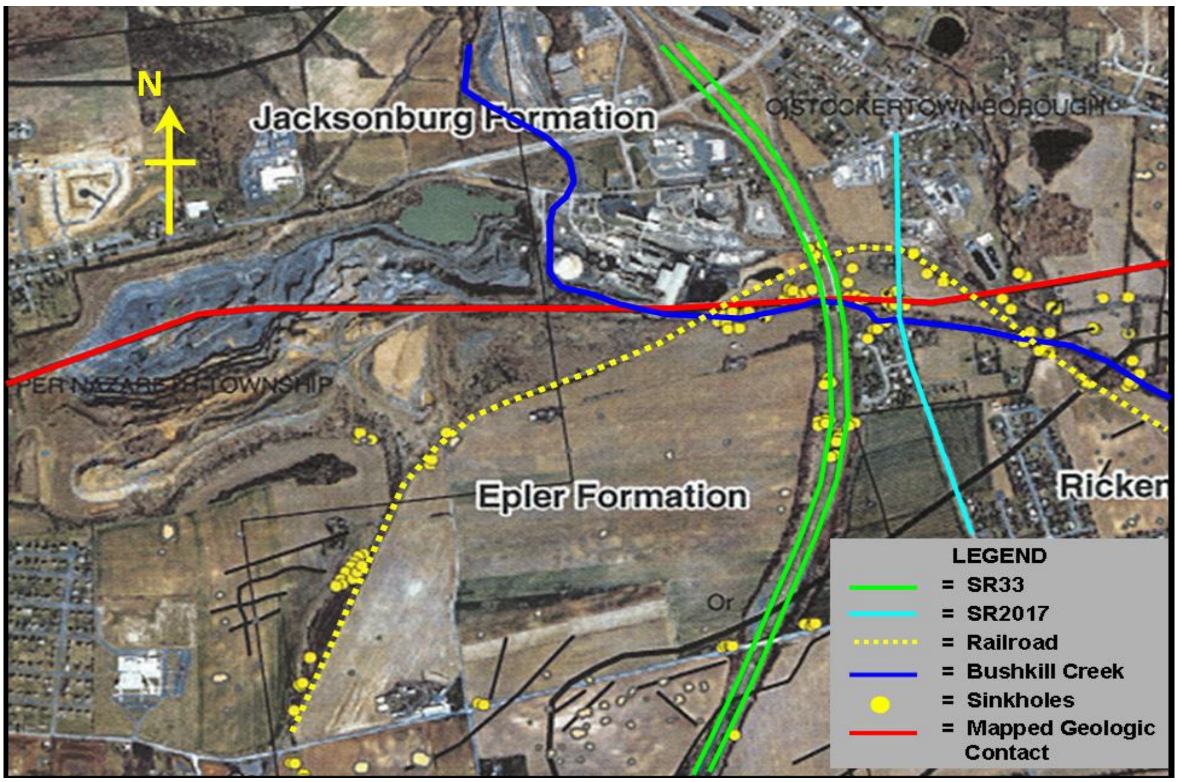

Figure #1 – Area Layout

This incident is just one of a series of recent sinkhole events in the area, spanning back to 1998 and beyond. These events appear to be linked in what is a combination of triggers and responses to a variety of both natural and human induced consequences. A limestone quarry is located upstream along the Bushkill Creek about one mile west of SR33. The quarry has been in operation since the early 1900's. Current maximum depth of the quarry operation is approximately 250 feet or approximately elevation 140.

Starting in 1998, there was a roughly two year period of higher than normal temperatures and extended drought conditions (see Figure #2). This is suspected to have resulted in severe soil desiccation. In July of 1999, precipitation amounted to only 0.33 inch (normal 4.1 inches), with temperatures for the month 6.7 degrees Fahrenheit (°F) above normal. In September of 1999 the drought broke when Hurricane Floyd passed through, dropping nearly 12 inches of rainfall on the area.

During this same period there was a sharp increase in pumping from the quarry. During the period from mid 1999 to early 2001, the pumping rate continued to rise from approximately 25 million gallons per day (MGD) to over 55 MGD (see Figure #2). As the pumping rate accelerated, the incidence of major sinkhole activity increased. In October of 2000 sinkholes on either side of the creek just downstream of SR33, forced the closure of another state highway bridge over the creek on SR2017 and resulted in the condemnation of a private dwelling.

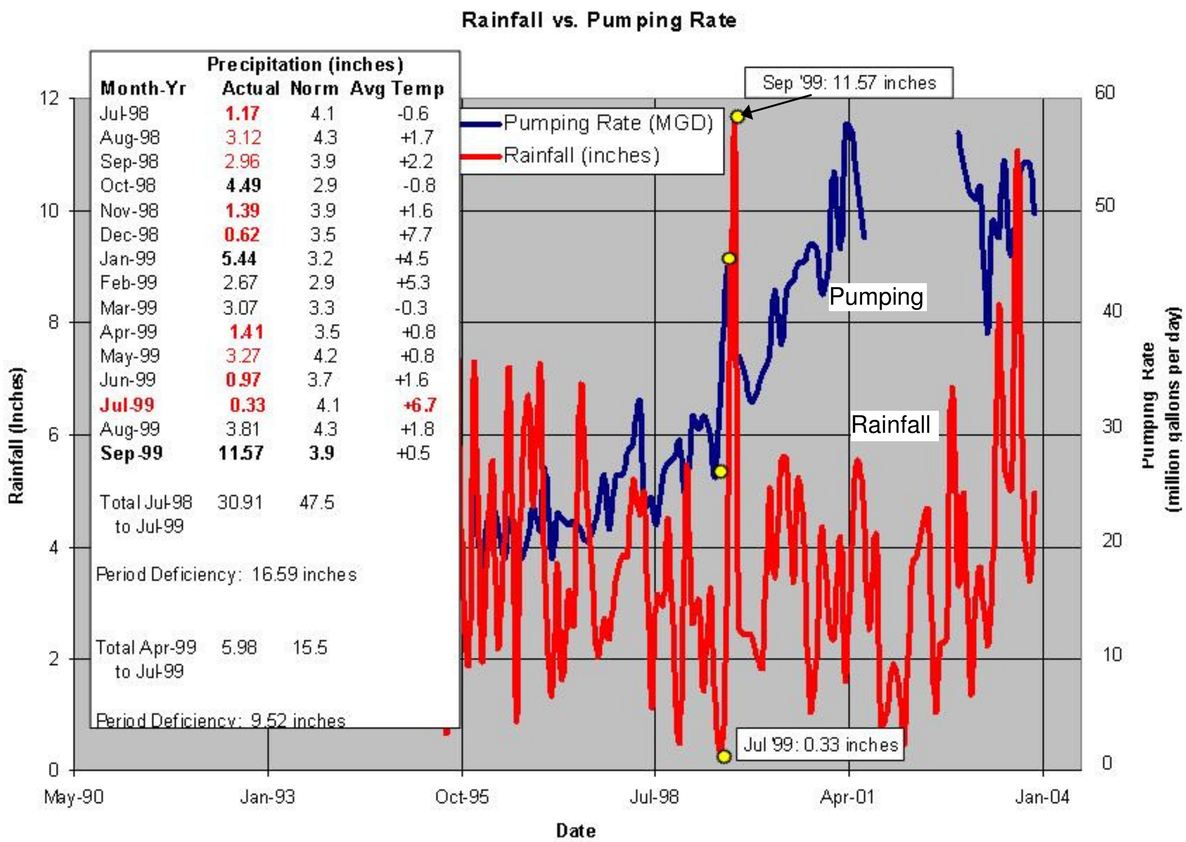

Figure #2 – Weather vs. Pumping Rate

In 2001 approximately 1200 feet upstream from SR33, the bridge carrying the Norfolk Southern Railroad over the creek, experienced a major sinkhole causing the collapse of a wingwall on the south abutment. January 2004 brought the loss of the SR33 northbound structure. In late spring of 2004 and then again in late summer 2004 major sinkholes plagued the reconstruction efforts of the SR33 bridges, with sinkholes developing in the pile groups of two of the four abutments. In early fall of 2004 the newly constructed northbound SR33 structure started showing evidence of movement in the north abutment and approach – the same area of the sinkhole event during reconstruction.

In the fall of 2004 a liner was installed by the quarry opening creek from the Norfolk Southern Railroad Bridge to approximately 400 feet west of SR33. The purpose of the liner was to cut off infiltration through several large sinkholes in this section of the creek. In January 2005 another major sinkhole event resulted in multiple collapses in the field east of SR33. The holes coalesced with each other and the creek, cutting a channel across the field from SR33 to approximately two thirds the distances to SR 2017. By late winter 2005 evidence of movement was observed in the new SR33 southbound structure. Again, greatest movement occurring in the area of the sinkhole that developed in the pile group of the south abutment during reconstruction.

Intermittent sinkholes continued to occur in and around the creek, with one event exposing a portion of the piles for the south abutment of the new SR33 northbound structure. The activity continues to threaten the stability of the bridges, and safety of the local community.

What links these events together? The obvious answer is Karst. But what is the nature of a karst system that causes distress within months of the completion of new structures with foundation elements as deep a 360 feet? What is the nature of a system that within a period of five years and distance of over one mile, severely damages or completely claims four bridges, results in the condemnation of one residence, reeks havoc on a stream and surrounding flood plain, over doubles the pumping requirements of a surface mining operation, necessitates frequent maintenance of a highway system, and continues to threaten the safety of the local community? It is clear that a lot of the clues lie within the question itself, but it was equally clear is that to be able to make reliable future decisions, this linkage needs to be recognized.

Investigation

The early evidence indicated a strong connection between the sinkhole activity in the creek and the volume of pumping necessary at the quarry. The theory was that much of the water was simply being re-circulated from the creek to the quarry and back into the creek. For this to occur at the volumes observed there would have to be a conduit like connection between the creek and the quarry. It is expected that this continual circulation through the subsurface was resulting in a persistent subsurface erosional process. Over time the erosion manifests as sinkholes and the slow loss of foundation support for the highway structures.

To determine the actual conditions and dynamics that existed, it would be necessary to undertake a very extensive and comprehensive, and ultimately very deep, site investigation. The primary goal would be to have a comprehensive understanding of site conditions and forces at work, so a solution could be developed and implemented for the long term stability of the bridges, and that would also hopefully improve the situation for the local community.

To this end, a comprehensive investigation was initiated jointly by the Pennsylvania Department of Transportation and the Pennsylvania Department of Environmental

Protection in cooperation with the operator of the quarry. Assistance was also obtained from the Pennsylvania Geologic Survey, the United States Geologic Survey, the United States Army Corp of Engineers and H2H associates, consultant to the quarry owner.

The investigation involved a broad range of techniques including subsurface borings, geophysics and geologic/hydrogeologic studies. Borings, water quality logging, a brine tracer study, and two stream gauging/seepage runs conducted by PADEP, provided the most significant results and will be of primary focus in this paper. Although the various geophysical methods provided good results, the ultimate depth of the karst system proved a limiting factor for these techniques (in the manner applied) of providing substantial conclusive information, towards understanding this particular problem.

Subsurface Borings and Water Quality Logging

A review of the original micropile installation records indicated a need to investigate the subsurface conditions more thoroughly, and to a greater depth. A series of twelve holes were planned around the two structures (see Figure #3). The holes were all drilled between the two bridges from the median of the highway, so as to avoid traffic control problems. Two holes were drilled immediately next to each abutment, and one hole was drilled approximately 100 feet back from the abutment. For the two holes next to each abutment, one was drilled to follow the batter (usually 3/12) of the front row of piles, and one was drilled vertical to match the vertical rear row of micro-piles. Each of the holes 100 feet back from the abutments, were drilled vertically. In order to accurately define the subsurface conditions around the structures, and clearly delineate and identify rock quality and type, high recovery four inch diameter rock coring was conducted during this series of borings.

The results of the borings are indicated in Sections A-A' and B-B' (Figures #4 and #5 respectively). Section A-A' is along the west (median) side of the northbound structure and Section B-B' is along the east side of the southbound structure (see Figure #3). The twelve borings varied in depth from 153 feet to 500 feet, with an average depth of 352 feet. The borings indicate a well developed very deep (approximately 400 to 450 feet) karst zone. The karst appears to lie directly under the creek, and transitions rapidly to a much higher quality limestone and dolomite both north a south of the structures.

Each of the borings was lined with perforated casing to permit logging of the variation of water quality with depth. Water quality measurements included temperature conductivity and ph. The holes were logged from bottom to top, with continuous recording of data. Of most value in the data logging was water temperature. Groundwater typically remains at a fairly constant temperature of 53°F. Significant variations (more the 1°F) from this value, up or down, indicate the presence of an external flow source. During the period of logging, the creek water ranged in temperature from 59°F to 63°F. Any rise in groundwater temperature would be an indication of rapid infiltration of creek water through the sinkholes in the streambed.

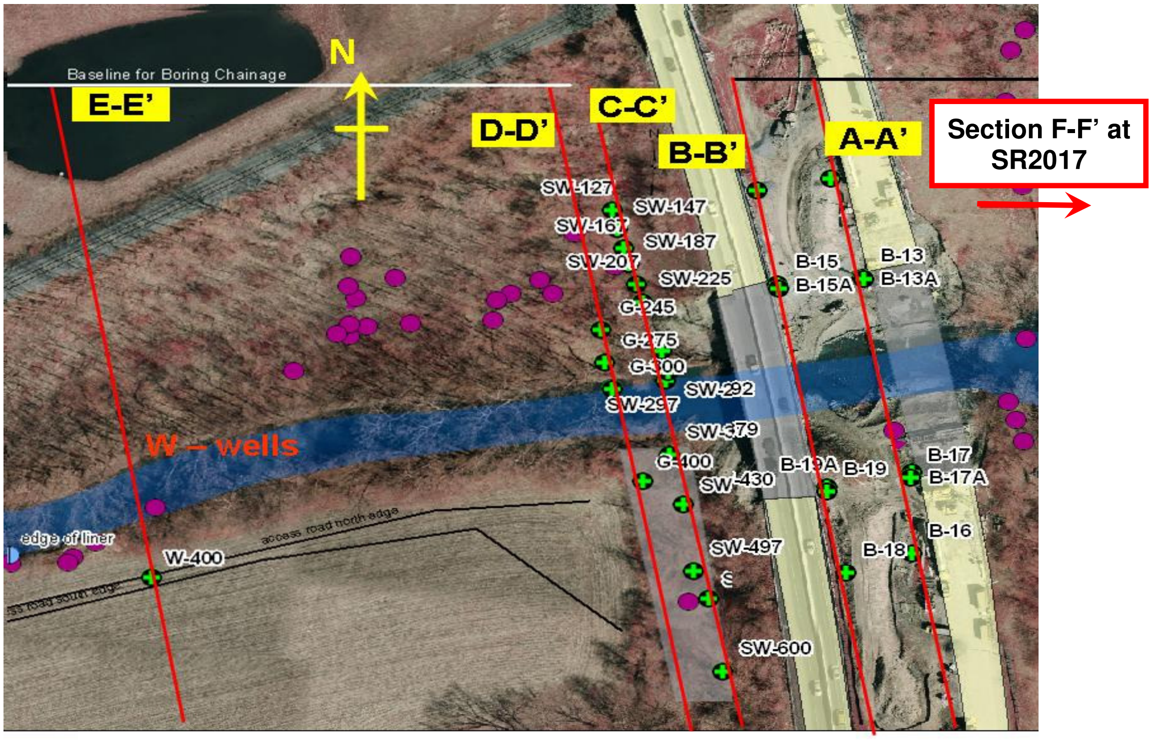

Figure #3 – Boring Layout

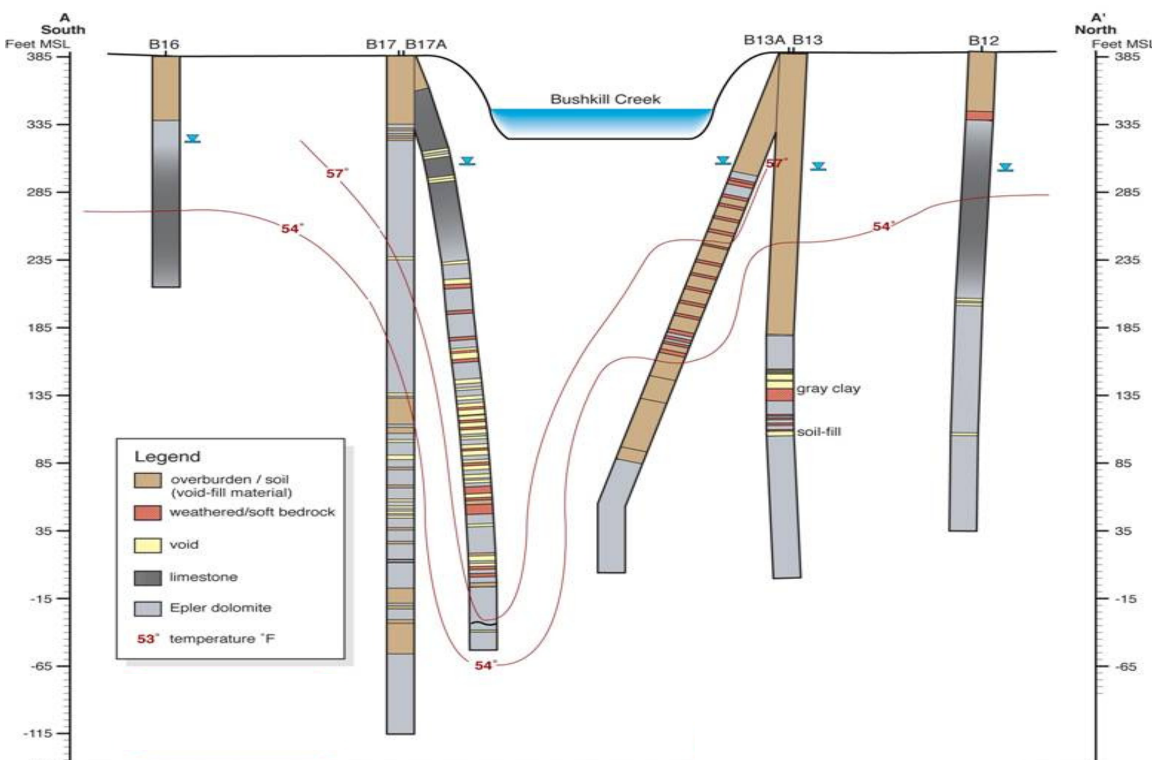

Figure #4 – Section A-A'

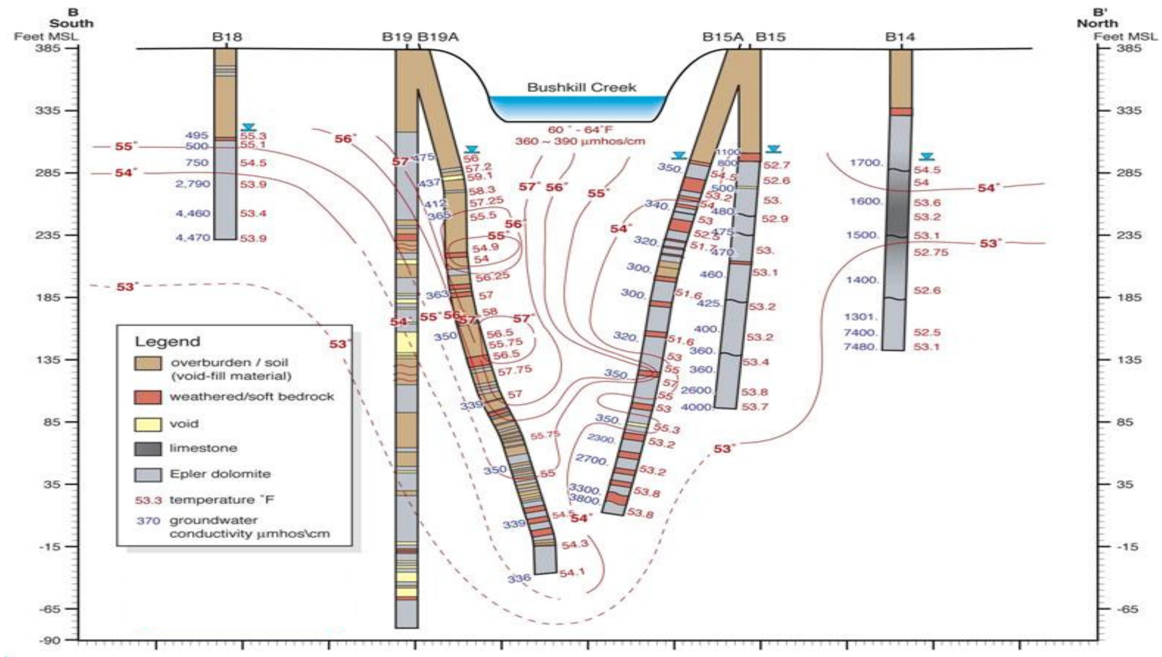

Figure #5 – Section B-B'

Inspection of the groundwater, temperature contours on Sections A-A' and B-B' shows elevated groundwater temperature in some borings to depths of approximately 400 feet. This data suggested rapid infiltration of creek water directly to the subsurface through sinkholes in the streambed. The large depth and lateral extent of the rise in groundwater temperature, and the magnitude of the temperature rise, indicate high volume, high velocity infiltration. This is consistent with conduit (versus diffuse) flow, which points to the likelihood of a karts conduit system. Such a system may consist of anything (from a single large conduit, to multiple interconnected small channels through the karst.

In order to better define the karst system, and confirm and more accurately delineate a conduit system, another series of deep holes was planned. Based upon preliminary concepts, these holes may also involve implementation of a comprehensive remediation plan. The new series of holes would be drilled at the base of highway embankment immediately west of SR33 southbound (see Figure #3). These holes would form Section C-C'. Ultimately Section C-C' would be comprised of 15 holes varying in depth from 65 feet to 545 feet, with an average depth of 377 feet. Twelve of the holes were drilled vertical, and three of the holes were drilled at angles dipping south, so as to investigate conditions directly beneath the creek without having to disturb the stream. The three non-vertical holes were drilled at angles of approximately 7, 15 and 29 degrees (roughly 1.5:12, 3:12 and 7:12 respectively). The nature of all drilling in the karstic condition was very difficult, but was extremely challenging and time consuming for all angled holes. This series of holes was conducted only measuring penetration rates and identifying cuttings. No core sampling was performed. A total of 32 borings were conducted for the investigation with a total footage of 12,129 feet, and an average depth of 379 feet.

The results of the Section C-C’ borings are indicated on Figure #6. The findings at C-C’ are similar to those at A-A’ and B-B’, but with a better defined section due to the higher density of borings. The karst is again well developed with a depth exceeding 450 feet and width of approximately 260 feet. Boring depths range from 65 feet to 545 feet, with an average of 377 feet. The center of the karst lies approximately sixty feet south of the creek, indicating that the axis of the feature strikes a bit north of due east. As with all other borings, the holes were lined with perforated casing to permit logging of ground water quality with depth. Temperature was again the most valuable parameter measured. As with Sections A-A and B-B, elevated ground water temperatures were detected at great depths. Elevated temperatures were measured to the bottom of boring SW-497 at a depth of 545 feet (elevation minus 203 feet MSL). The depth and lateral extent of elevated groundwater temperatures again indicated a rapidly flowing conduit system (see Figure #7).

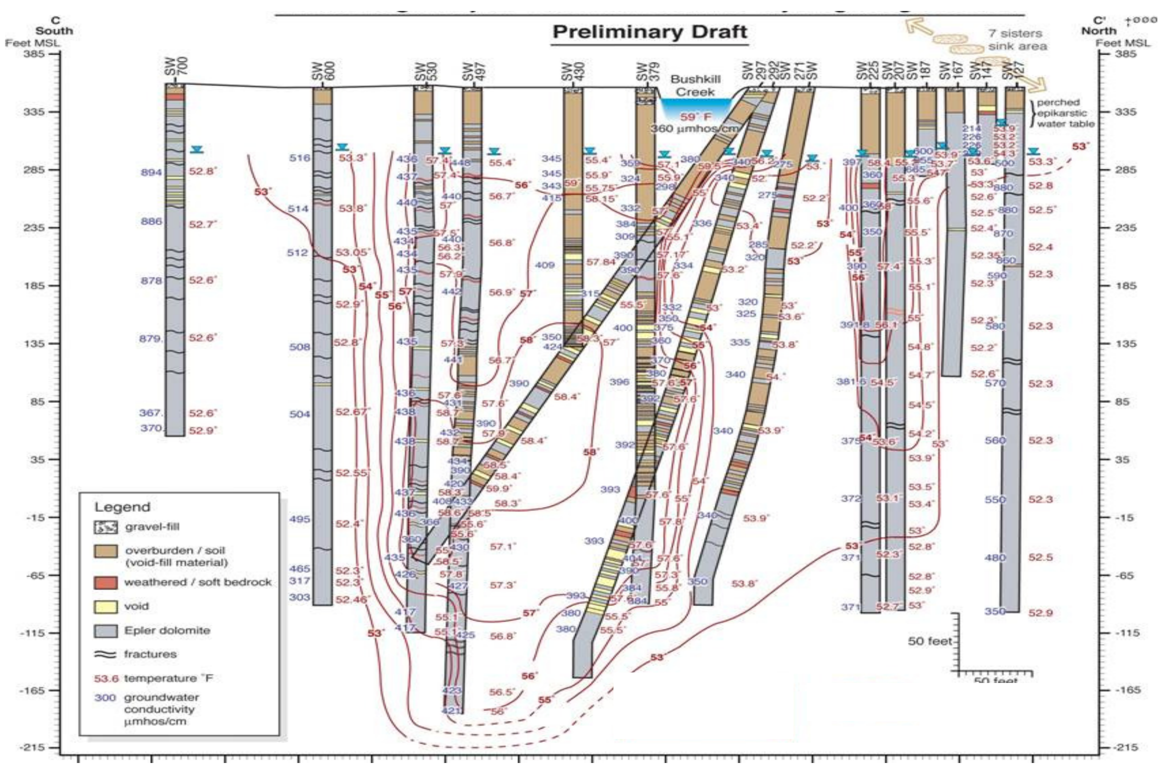

Figure #6 – Section C-C’

A comparison of the three sections is shown on Figures #8 and #9. Several other factors can be observed in this figure. Not only does the axis of the karst trend towards the south when moving westward be it also tends to deepen somewhat. The other major difference of note is the overburden layer is much deeper in the center of the karst in section A-A' and B-B' than in C-C'. This may however be a result of interpretation from the different drilling method (air, non-recovery) at Section C-C'.

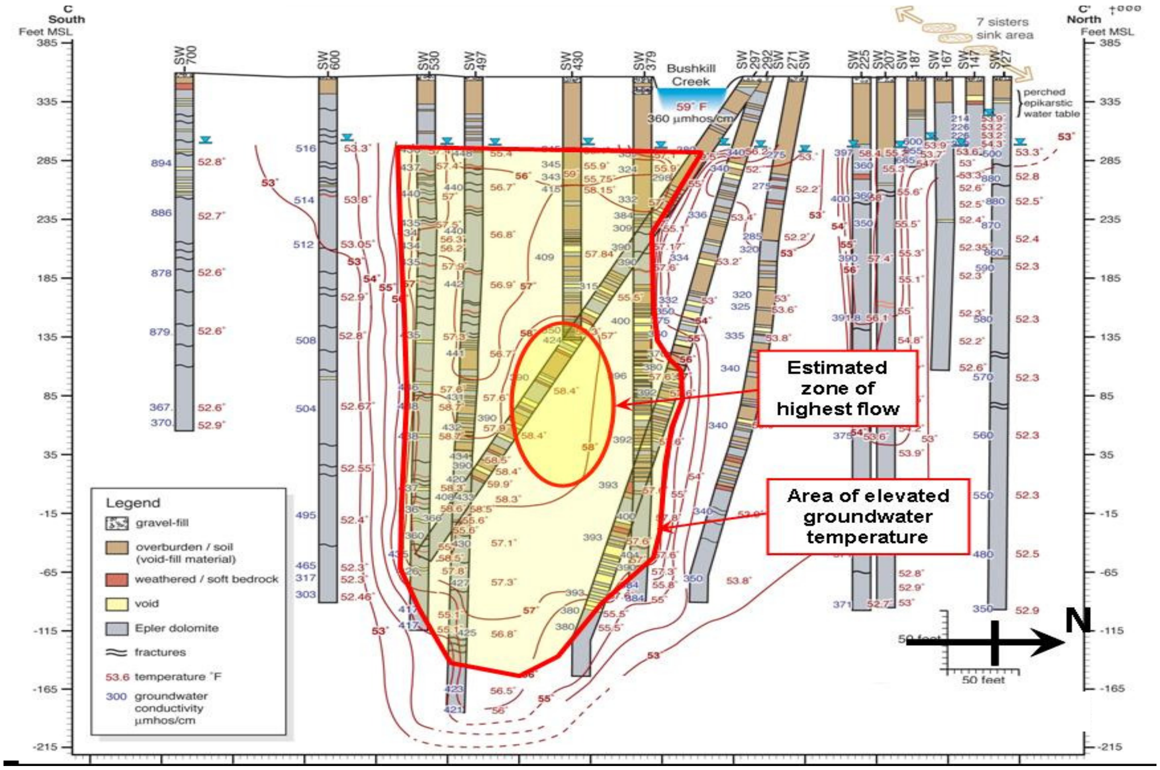

Figure #7 – Conduit System, Section C-C'

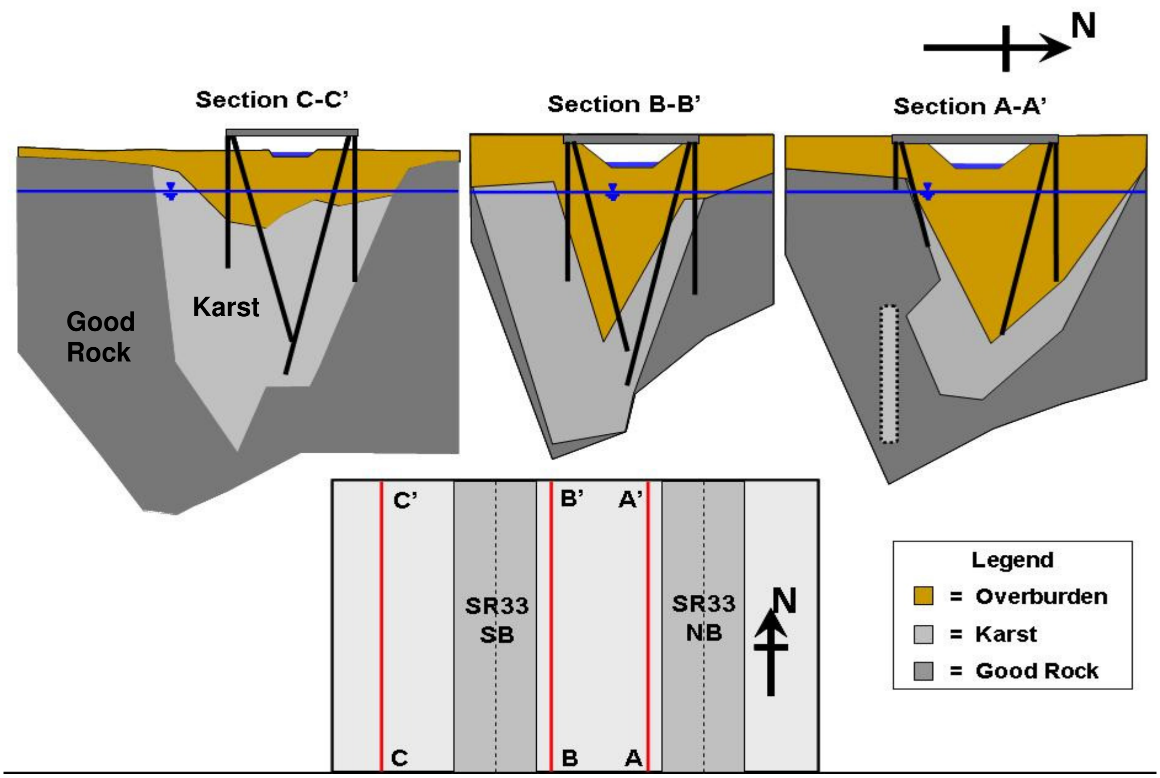

Figure #8 – Simplified Sections w/Structures

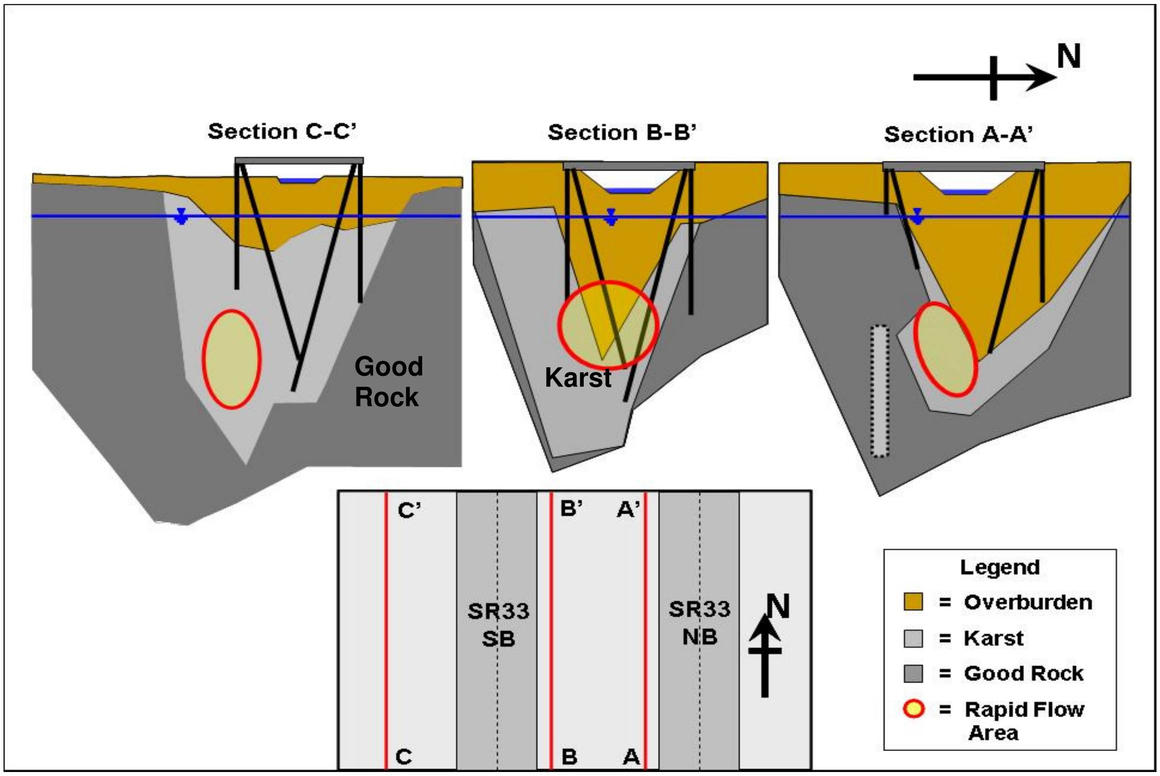

Figure #9 – Simplified Sections w/Suspected Conduit Zones

During the fall of 2004, a liner system was installed by the quarry operator in the creek from the Norfolk Southern Railroad Bridge, downstream to approximately 400 feet west (upstream) of SR33. The purpose of the liner was to cut infiltration of creek water into sinkholes in the stream. The goal was to reduce the volume of pumping required in the quarry. Daily pumping at the time was at a rate of approximately 55 million gallons per day (MGD). Unfortunately no seepage run information was available for this stretch of the creek prior to the lining operation, so quantification of loss of creek water through sinkholes repair and covered during the lining, cannot be made. The liner was completed in approximately the latter part of November 2004. During lining the water was diverted (by pumping and pipes) around the creek from upstream of the Norfolk Southern Bridge to beyond the limits of the liner west (upstream) of SR33. Full flow was maintained downstream through the SR33 and SR2017 bridges, during the entire liner operation.

In late January 2005, sinkhole activity increased dramatically just downstream (east) of SR33. A sinkhole area just beyond SR33 on the north side of the stream channel that had sporadic activity through 2004 became extremely active. A zone in the adjacent field, which had been experiencing reoccurring sinkholes for some years, developed a series of large sinkholes that coalesced and connected to the creek. The resulting channel that developed on the north side of the creek extended from just west of SR33, approximately two thirds of the distance to SR2017 (see Figure #10).

Figure #10 – January 2004 Accelerated Sinkhole Activity

In the fall of 2000, sinkholes formed adjacent and beneath the north abutment of the SR2017 bridge over the Bushkill Creek. Damage to the abutment as a consequence of the sinkholes, resulted in the bridge taken out of service. Sinkholes also occurred in the north roadway approach to the structure with a major sinkhole as far away as approximately 200 feet.

Borings were performed as part of the design effort to develop replacement structure at SR2017. The results of these borings are shown on Section F-F' (see Figure #11). The borings indicated that the top of rock surface drops rapidly until boring R-12 approximately 200' north of the structure. The depth to rock drops from roughly 20 feet at the SR2017 north abutment, to over 90 feet at boring R-12. Going north from R-12 the top of rock surface rises to a depth of around 40 feet at boring R-15, 400 feet north of the creek.

Discussions and information from Dr. Richard Parizek from Penn State University, who was hired as a consultant by PA Dept. of Environmental Protection (PADEP), indicated that deeper overburden soils are associated with geologic fracture tracings. This is consistent with the three deep boring sections at SR33. Inspection of an April 2004 color infrared photo of the field just east of SR33 shows a lineament through the field at the same location where the sinkholes coalesced and connected to the creek in January 2005. Site reconnaissance was conducted of the area to the east of SR2017, around both the Bushkill and Little Bushkill Creeks. A broad depression was observed in the field just east of SR2017. The axis of the depression aligns well with the lineament observed in the field on the west side of SR2017. Again, information from Dr. Parizek indicated that depressions of this nature are associated with fracture tracings.

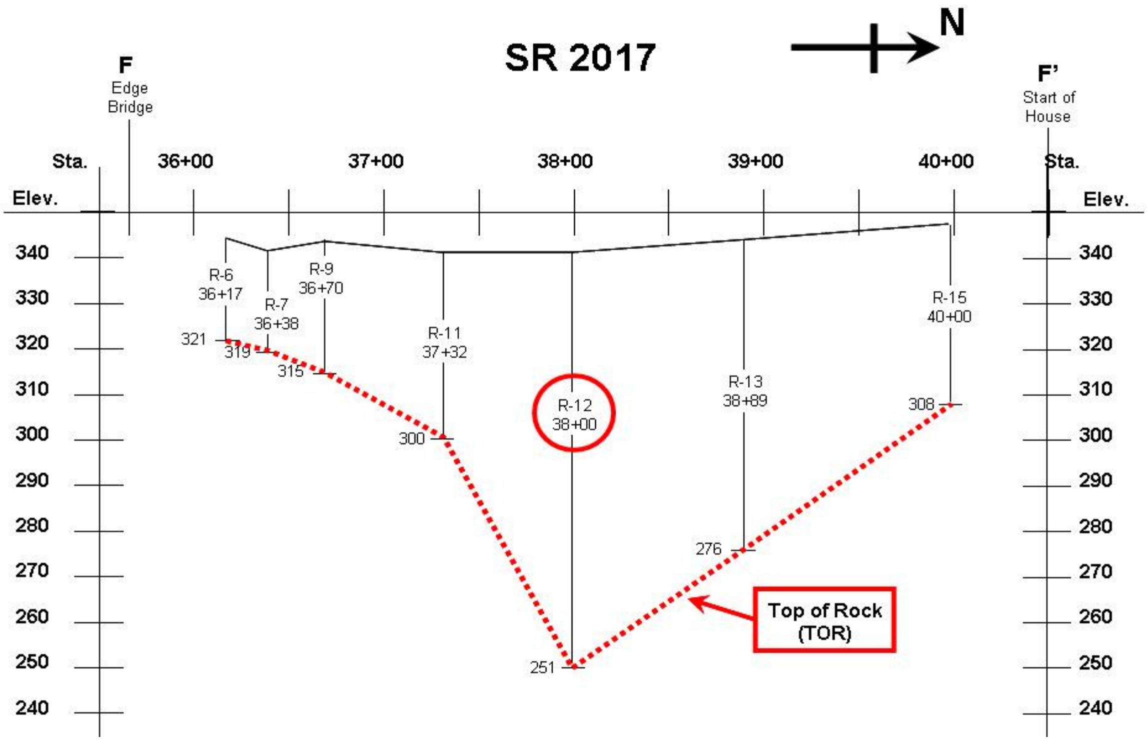

Figure #11 – SR2017, Section F-F'

Synthesis of Findings

All of the surface and subsurface investigation information was compiled and analyzed. An overlay on the color infrared air photo was prepared (see Figure #12). The alignments of the karst system determined from the borings at SR33, the lineament observed in the field west of SR2017, the deepest location of top of rock at boring R-12 on Section F-F', and the axis of the depression in the field to the east of SR2017, all align reasonably well. These alignments suggest that the deep karst system deviates from beneath the creek just downstream of SR33, and follows the alignments indicated through the fields and across SR2017.

Stream Gauging/Seepage Runs

During the investigation, the Pennsylvania Department of Environmental Protection was able to conduct two seepage runs along the Bushkill Creek. Results of the runs are indicated in Figures #13 and #14. The stream measurements were able to estimate how much water was lost in sinkholes in the creek. One run was conducted in the summer of 2005 in low flow conditions, and the other was conducted in the spring of 2006 during high flow conditions. Both runs indicated significant loss of creek water between SR33 and SR2017. During low flow a smaller volume was lost that comprised a higher percentage (71%) of the total flow. Conversely, during the spring 2006 high flow measurements a much higher volume of water was lost in sink holes, but comprised a lower, but still very significant percentage (50%) of the total flow.

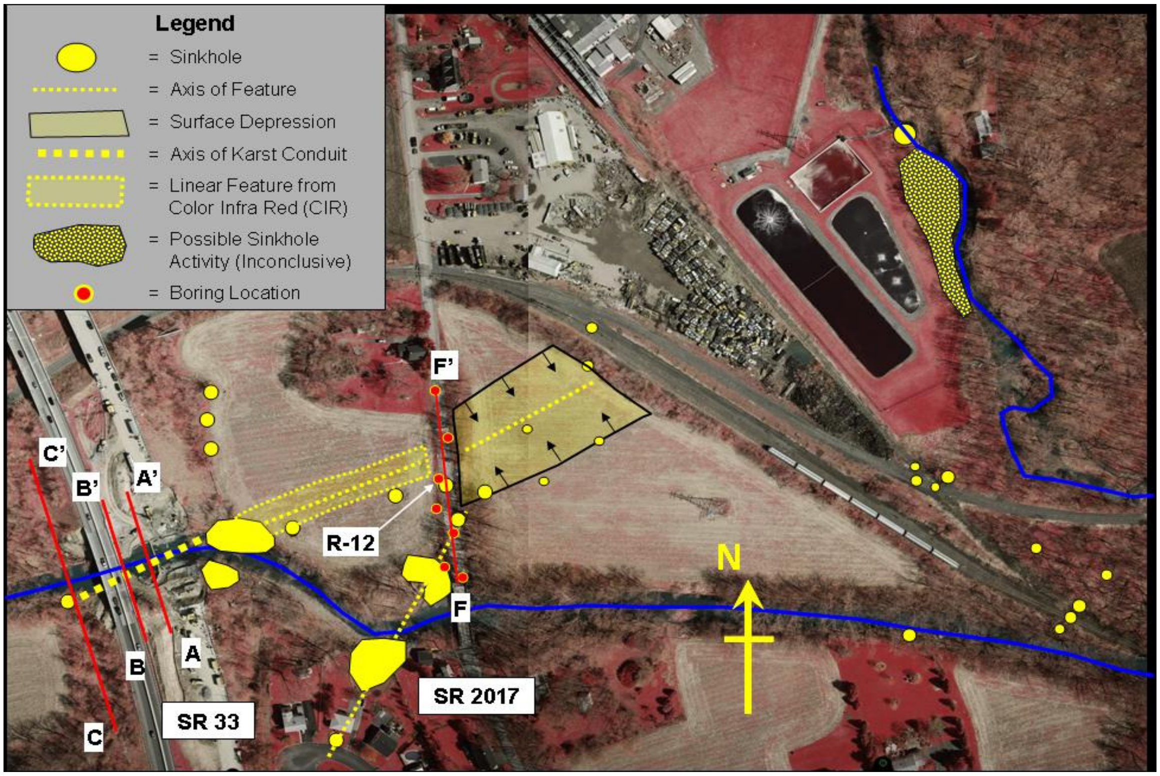

Figure #12 – Investigation Summary

Brine Tracer Study

As a result of the borings, water quality soundings and seepage run results, it was suspected that the high volume of flow lost in the creek was simply entering a deep conduit system connected to the stream sinkholes. Once in the conduit system, the water would flow back into the quarry where it was again collected and discharged back into the creek, where a portion of the flow would again be recirculated. This is supported by the relatively low volume of natural creek flow upstream of the quarry discharge.

It is suspected that this continual circulation of water is causing a steady and persistent erosion of subsurface soils. Such subsurface erosion would gradually remove support for the SR33 structure foundations. It is suspected that damage to the subsurface from this process is a major contributor to the movements observed at the bridges and roadway approaches.

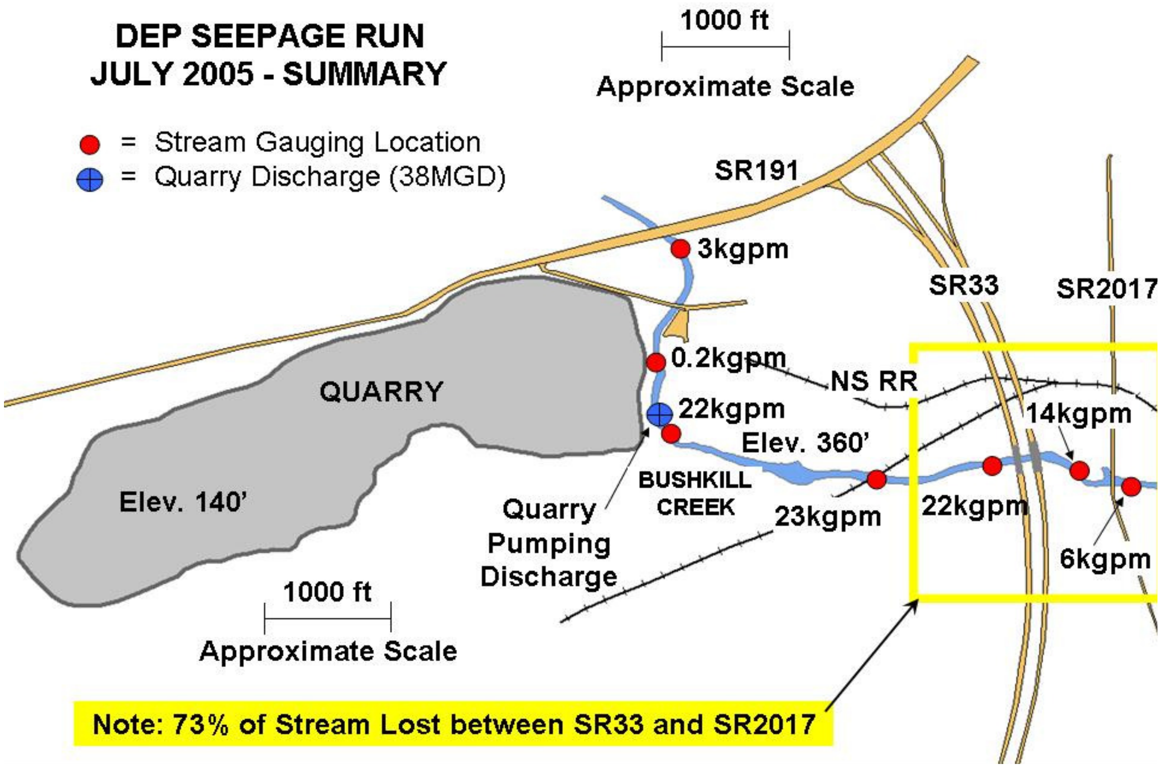

Figure #13 – July 2005 Seepage Run/Stream Gauging

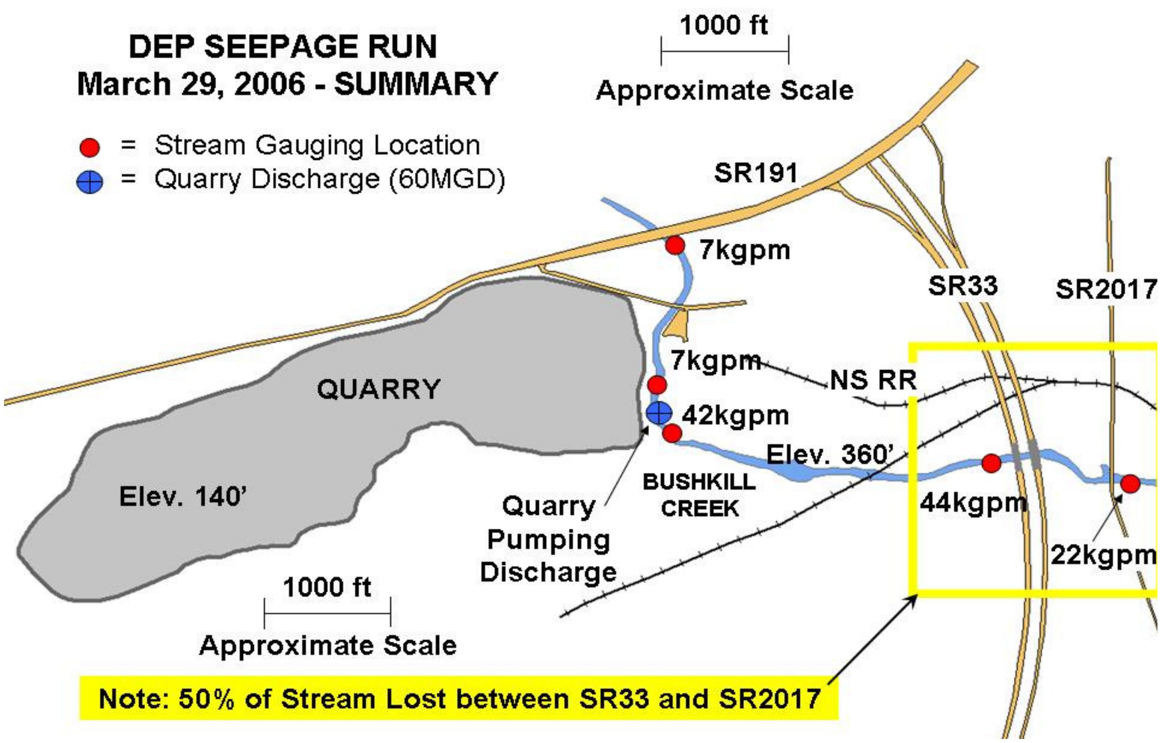

Figure #14 – March 2006 Seepage Run/Stream Gauging

In order to confirm the suspected flow pattern, and quantify the velocity of the flow, a brine tracer test was conducted at the site. Three sinkhole locations were isolated adjacent to the creek. The three holes were located between SR33 and SR2017 where the high flow lost was measured from the creek seepage run data (creek flow measurements). Locations of there holes are indicated on Figure #15. The holes were isolated with cofferdams, allowing sufficient water to flow in, to keep the sinkhole conduits active. Immediately prior to the test the remaining flow was blocked off, and a measured quantity of saturated brine solution was poured into the holes. A total of 500 gallons of brine solution was injected. The 500 gallons was divided between the three holes. Once the brine was injected, the flow of creek water into the holes was immediately re-established.

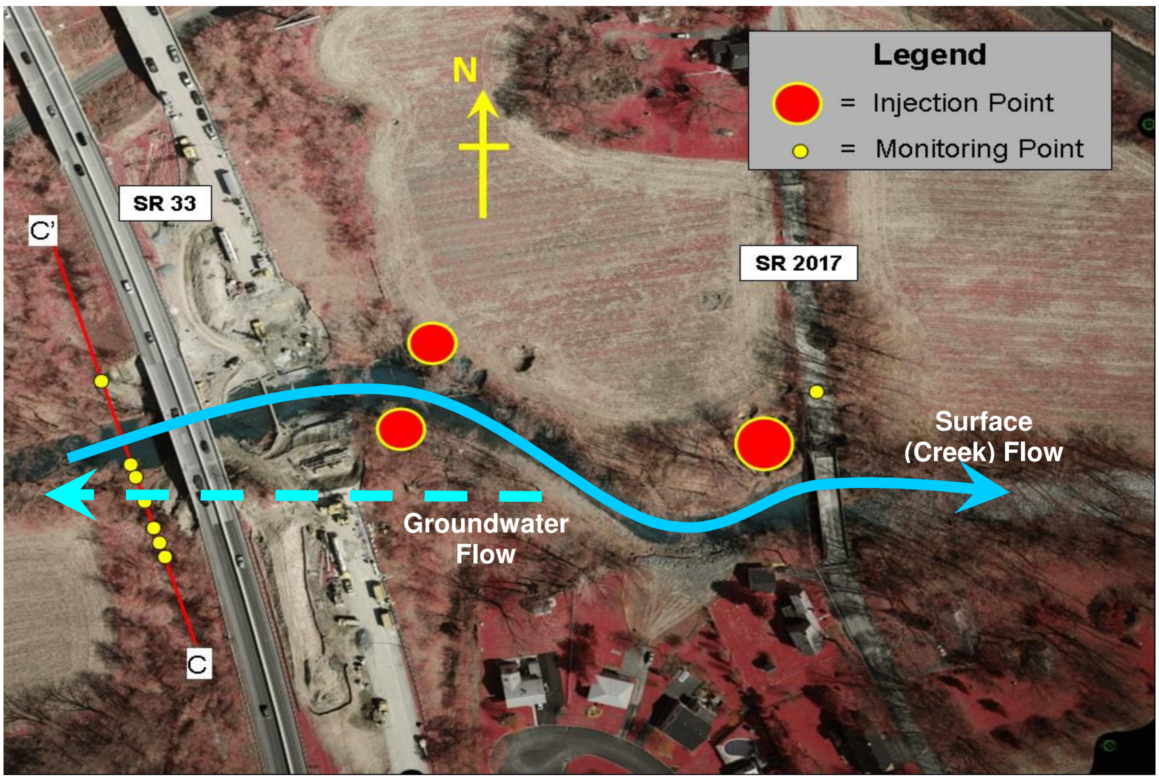

Figure #15 – Brine Tracer Injection Points

A total of sixteen In-Situ Troll 9000 loggers were installed to measure brine concentrations. The locations of the loggers are indicated on Figure #16. One logger was located in a well in the north bridge approach on SR2017. Seven loggers were located in various wells (SW bore holes) and depths along Section C-C'. One logger was installed in hole W- 400 west of SR33, near the end of the stream liner that was installed in the creek in the fall of 2004. Three loggers were located in wells owned by he quarry (two north of the creek and one south), and four loggers were located in artesian flows in the quarry floor. The loggers were installed approximately two weeks prior to brine injection in order to collect background data, and remained in place until it was certain that all the brine solution had passed through the system. Depth of placement of the sensors in the wells was determined by temperature logging. Sensors were located in locations of greatest groundwater temperature anomalies.

Figure #16 also shows the results of the tracer test. Brine was detected at nine of the sixteen sensor locations. No brine was detected at the SR2017 well or the three quarry wells. Brine was detected in four of the seven wells along Section C-C' (see Figure #17), and three of the four artesian flows in the quarry floor. Analysis of the data yielded average groundwater flow velocities of 5 feet/minute at Section C-C', and 19 feet/minute at the quarry floor artesian flows. These results support the theory of the presence of a conduit system in the karst, of connection of the creek sinkholes with the conduit system, of rapid groundwater flow conditions, and of recirculation of water back to and from the quarry. Based on the findings at Section C-C', there appears to be a dispersed conduit system, however higher velocity flows are suspected to exist in the area below boring SW- 430, that lies within the cluster of sensors. During drilling of SW-430, air was lost directly through boring SW-379 (sufficient to prevent completion of the boring), suggesting a well developed voided area.

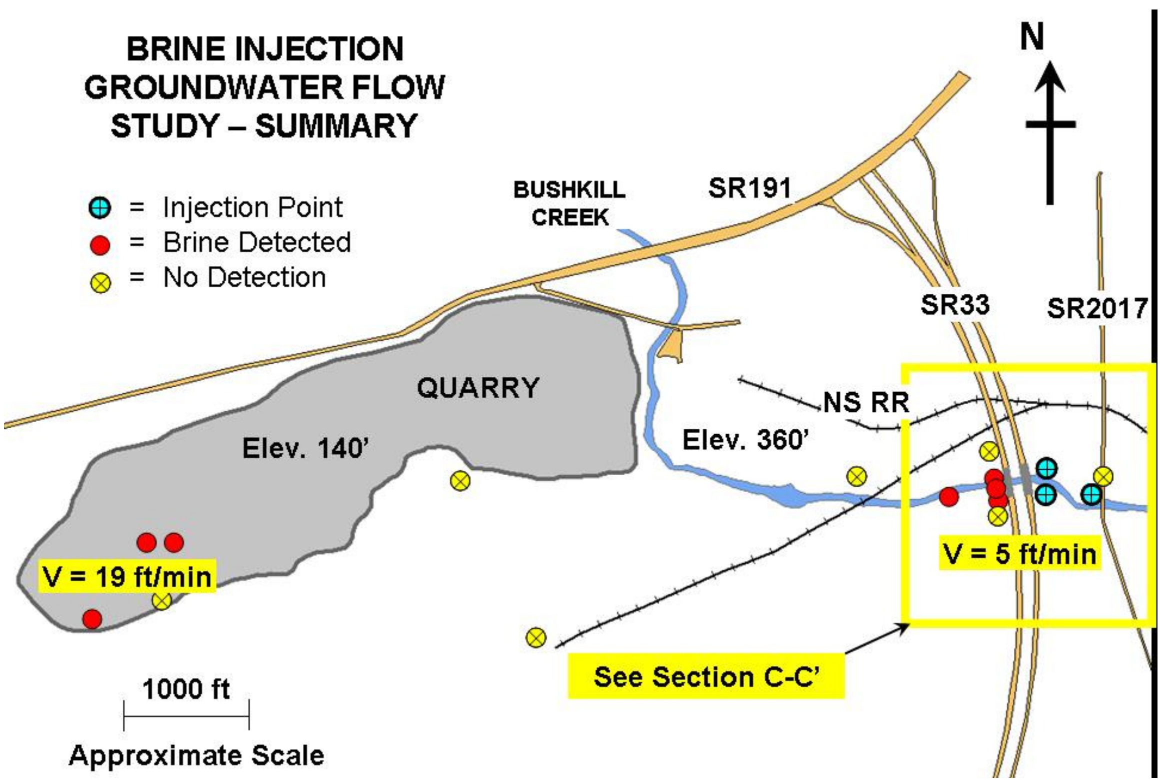

Figure #16 – Brine Tracer Sensor Locations and Results

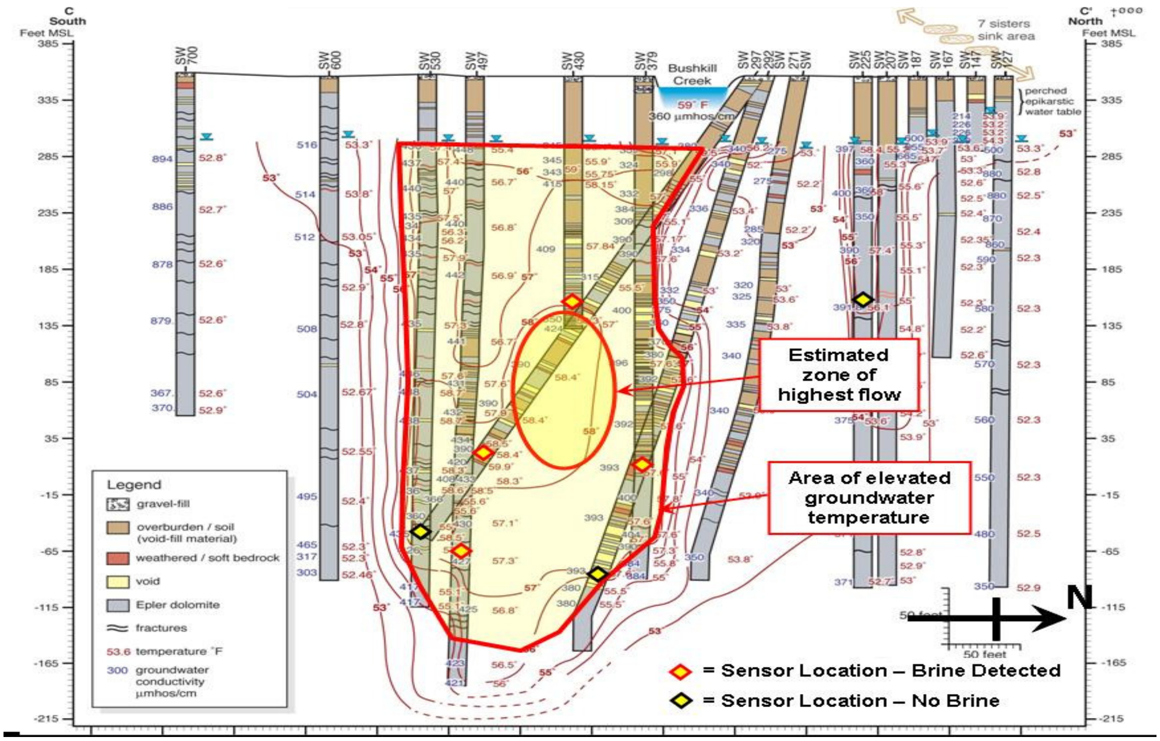

Figure #17 – Brine Tracer Sensor Locations Section C-C'

Mitigation Options

Now that a sufficient understanding was obtained of the karst and conduit systems, and groundwater flow conditions, mitigation options to stabilize the structures could be developed. A variety of alternatives were considered including various grouting plans, box culverts and stream lining. Stream lining was considered both in combination with the other mitigation techniques, and as a stand alone option.

The first grout option considered is what will be referred to as mass grouting. In this option the goal was to grout the entire deep karst mass to both stabilize the SR33 foundation and provide a barrier to retard groundwater flow back through the conduit to the quarry. This option would be done in conjunction with the stream liner, to provide a "belt and suspenders" approach.

The original mass grouting concept involved the construction of a grout curtain using both hot bitumen and portland cement based grout. The hot bitumen would flash set in the cold groundwater, plugging the major conduit flow paths. The temporary plug would reduce groundwater flow velocities to a level sufficient to permit grouting of the remainder of the karst with Portland cement grout. The grout curtain would be installed immediately west of SR33 Southbound, followed by grouting of the actual NB and SB structure footprints (see Figure #18). Once it was decided to consider a dual approach using both grouting and a stream liner, the bitumen grouting was eliminated, since diversion of the creek to facilitate stream lining would reduce groundwater flows sufficiently to permit conventional Portland cement grouting.

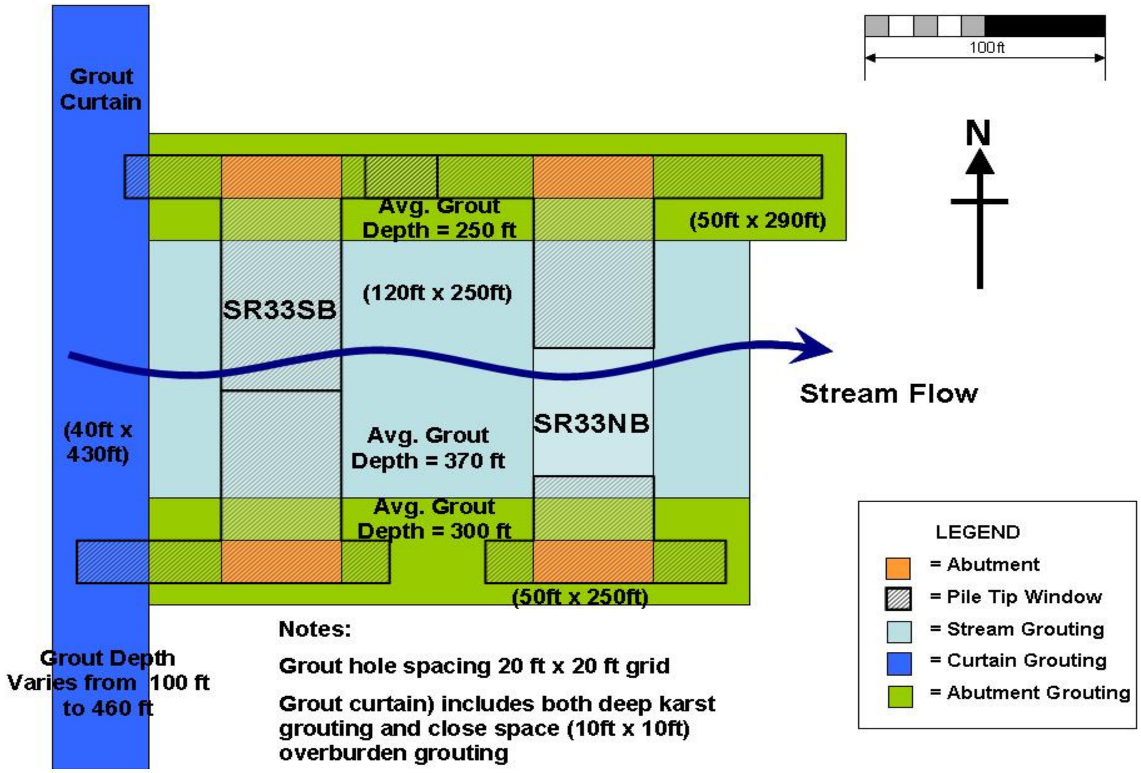

Figure #18 – Mass Grouting Plan

The other grouting option was termed surgical grouting. Surgical grouting would be targeted specifically at the structure foundation elements. The goal would be to provide and adequate vertical and lateral support for the micropile group, to stabilize the foundation within the deep karst (see Figure #19). The stream liner would be used to interrupt the flow cycle from the sinkholes in the creek to the quarry, so as to disrupt the continued subsurface erosion process.

Another alternative considered was to construct a series of post-tensioned box culverts that would be tied into the stream liner both upstream and downstream. A light-weight flowable fill would then be placed on top of the culverts to support the existing prestressed concrete beams (see Figure #20). Loads from any future movements would be transferred to the relatively large bearing area of the culverts.

The final option was construction of a stand alone steam liner. The liner would span from the end of the liner installed in the fall of 2004 approximately 400 feet upstream of SR33, to approximately 100 feet downstream of SR2017 (see Figure #21). The liner would consist of a combination of native clay soil, an LDPE geomembrane, and concrete with a waterproofing additive (see Figure #22). The liner would be tied into anchor trenches in the banks of both sides of the creek. Water would be diverted around the area of the creek to be lined with a pumping and piping system. The goal with the liner system was simply to interrupt the flow cycle between the creek and the quarry, again disrupting subsurface soil erosion.

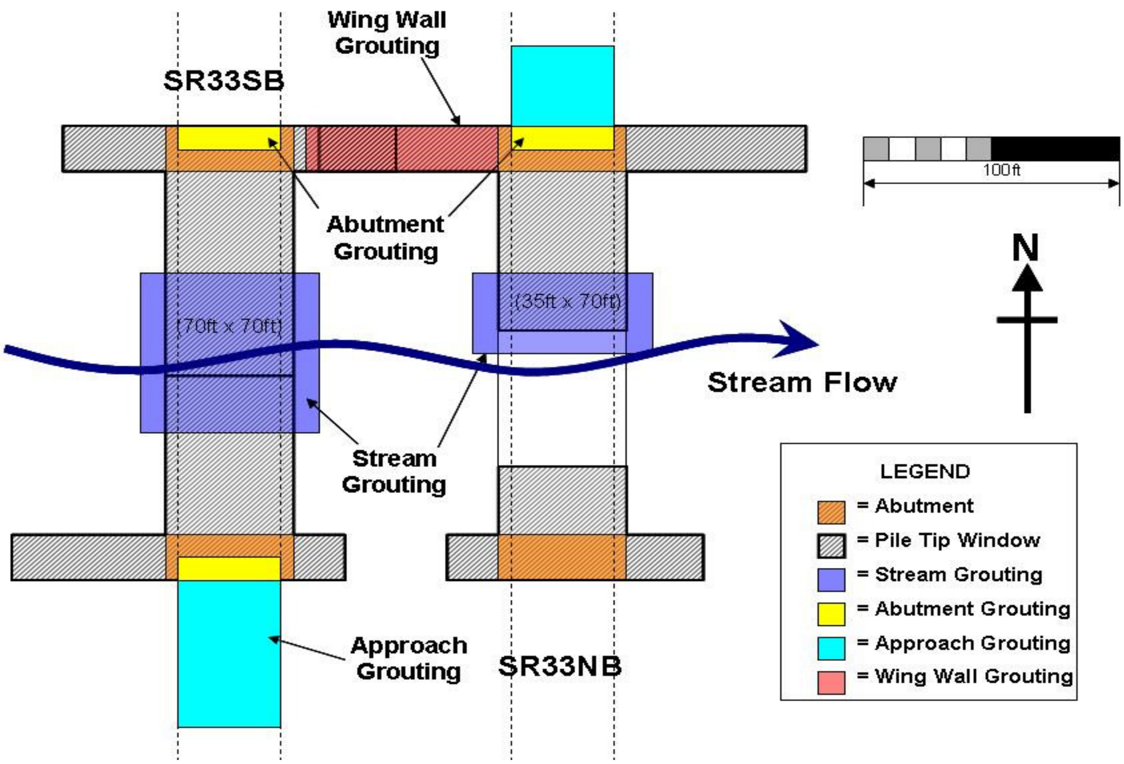

Figure #19 – Surgical Grouting Plan

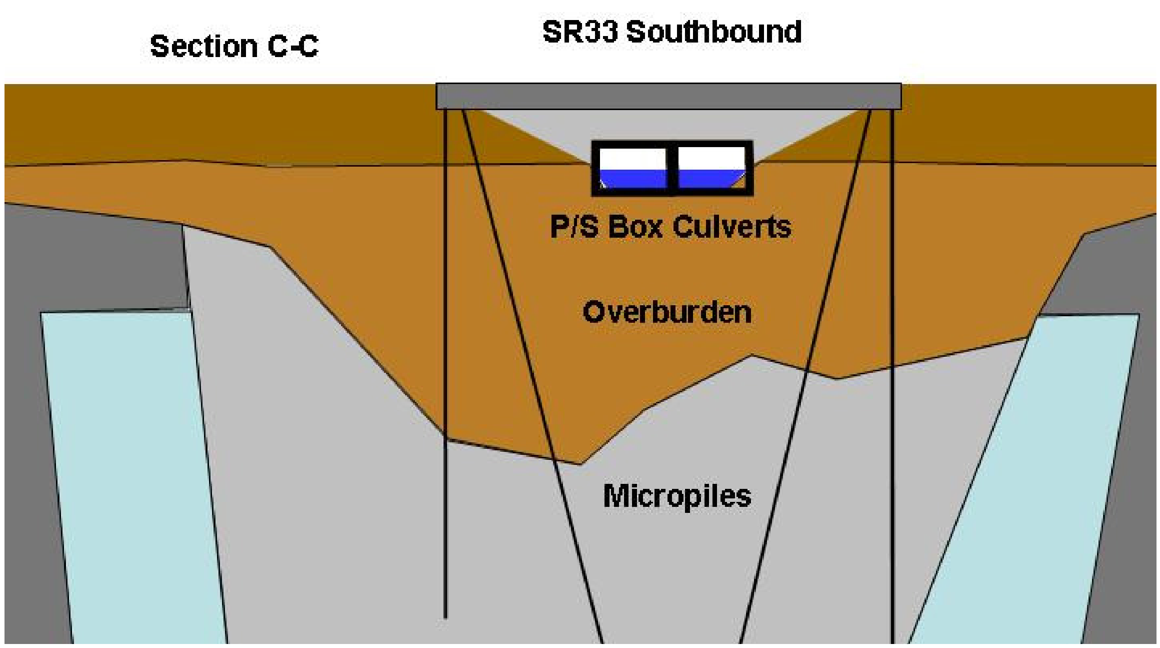

Figure #20 – Prestressed Box Culverts and Flowable Fill

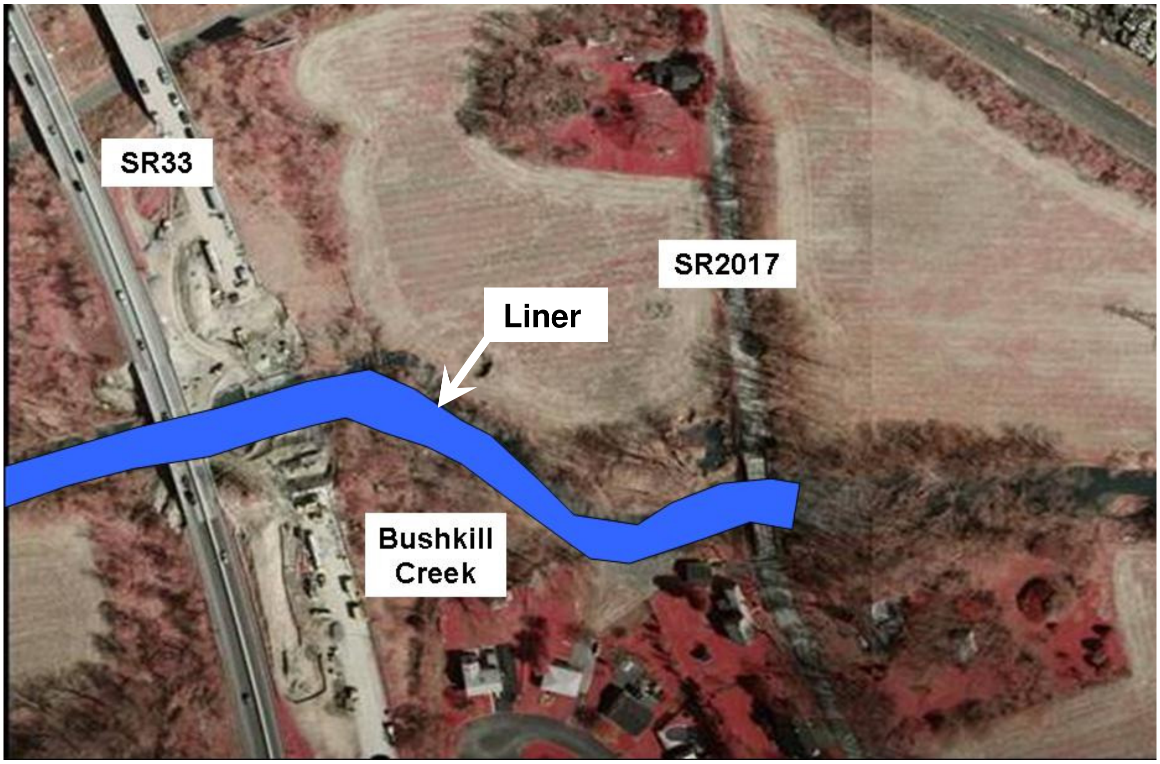

Figure #21 – Stream Liner

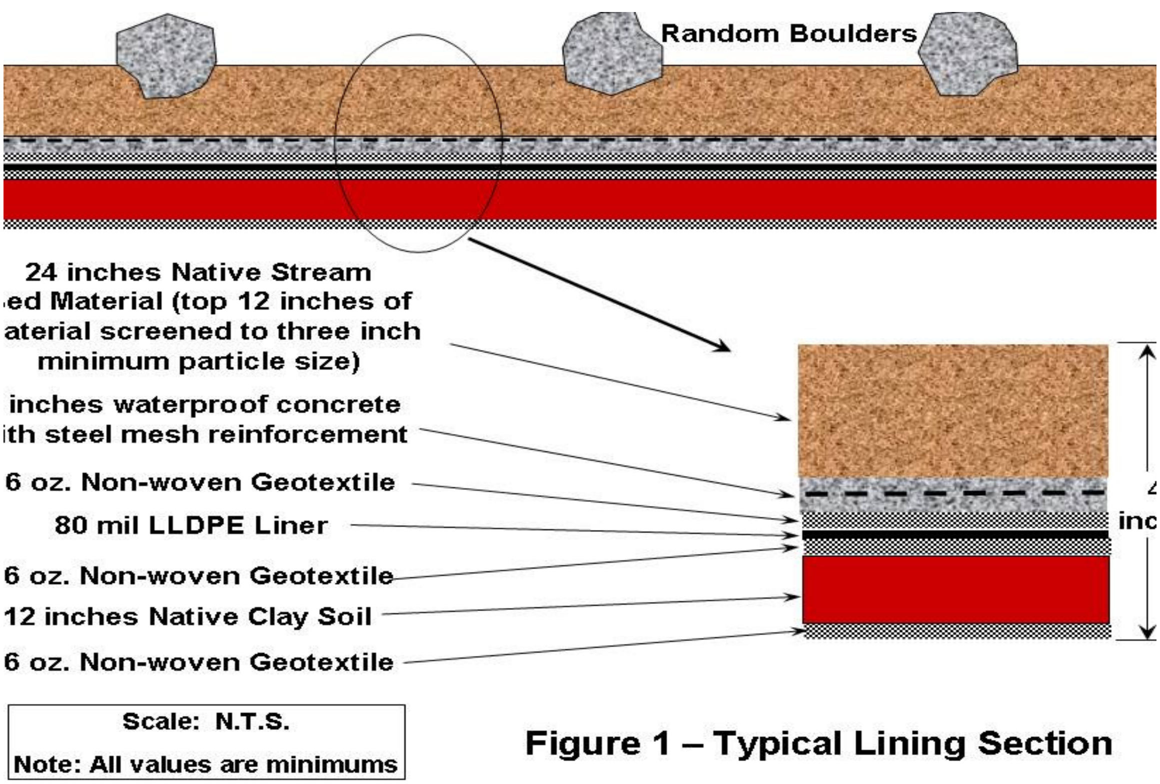

Figure #22 – Typical Liner Section

Option Analysis

All options were analyzed based upon a number of criteria including cost, potential for success of intended goals, impact on local residents, environmental impact, construction time, potential negative or unintended consequences, and constructability.

The grouting options had several major problem areas. The cost of either option (see Figure #23) exceeded the estimated cost of replacing the structures with foundations in competent rock. More importantly there were several significant practical issues. In discussion with contractors familiar with this type of work, it was quickly realized that the time required for construction would far exceed the time frame available to complete the liner system. It was intended to conduct the two operations simultaneously. Due to high pumping costs to divert the creek ($750,000 per month) and environmental concerns with extending the pumping duration, this was neither economically feasible nor practical to obtain necessary agency approvals. If the grouting were to be done, any holes with in the footprint of the stream would have to occur after completion of the liner. This would require temporary coffer dams, and penetrating the newly completed liner system. Construction staging would be a major logistical hurdle.

OPTION ANALYSIS SUMMARY

Option	Description	Cost (millions)	Stream Lining Required	Expectations to stabilize bridges
A	Mass Grouting**	28*	Yes	Low
B	Surgical Grouting**	17*	Yes	Medium to Low
C	Box Culverts	10*	Yes	Low
D	Stream Lining	5	-	Very Low

* Includes $5 million for stream lining

** Grout cost likely much higher than estimated due to migration losses

Figure #23 – Option Summary

Another problem was the required depth of grouting. Drilling to the depths required (over 300 feet) would be very difficult and time consuming. Drilling holes underneath the existing structures would require low head rigs multiplying the time and difficulty factors. Even with low head rigs, there were some locations of the structure (up the side slopes towards the abutments), that were simply not practical to attempt drilling and grouting.

And finally the most important consideration was the inability to verify the success of the operation, and quality of the final product. There would be no way to ensure that at these great depths the drill steel did not wander, resulting in grout placement other than in the intended locations. Hole wander was a major problem, especially for angled holes, during the investigation phase. There was also no way to verify that the grout actually would fill voids as intended, and would not migrate well beyond the targeted treatment areas. In short, after a lot of time, cost and effort, potential disruption of traffic, and potential environmental impacts, there would be now way to reasonably assure success. The cost estimates of the construction are also assumed to be low based upon past experiences with grouting and the experience of others, due to anticipated grout takes much higher than estimated.

The box culvert option also posed several problems. One problem was the length of the proposed boxes (over 300 feet). With the high quality of the Bushkill Creek, the impact of cutting off sunlight to such a long section would be environmentally significant. Another problem was that adding such a significant load, even though well distributed, over such unstable conditions, would yield unpredictable and likely adverse results. Any settlement of the culverts would also impact the integrity of the lining system, since the two would be integrated.

At this point, the liner system appeared to be the only option that could offer some level of protection to the structure foundations. Discussion continued on exactly where to end the lining system. It was also anticipated that there would be some negative impacts (short term sinkhole activity expected), but that long term the liner would provide the best opportunity to stabilize the creek, and interrupt the cycle of surface and subsurface flow. Interrupting this cycle was expected to greatly reduce the subsurface erosional activity leading the high sinkhole activity, and long term stability concerns of the structures.

Impact of Proposed Liner

The PADEP obtained the services of the United States Geologic Survey (USGS) to conduct a groundwater modeling study of the entire watershed area. The preliminary results of this study became available in June 2006 as USGS Open File Report 1143. The main purpose of the study was to determine the impact to groundwater levels from surface mining operations in the area. As some type of lining system was under consideration, impact form a liner was also investigated and modeled. The preliminary model results indicated that installing a stream liner an the proposed location would create significant groundwater drawdown (see Figure #24). The preliminary results indicated a groundwater collapse of greater than 50 feet, and up to 90 feet, in the immediate vicinity of SR33. The drawdown would be anticipated to occur very rapidly based on the results of the bring tracer study.

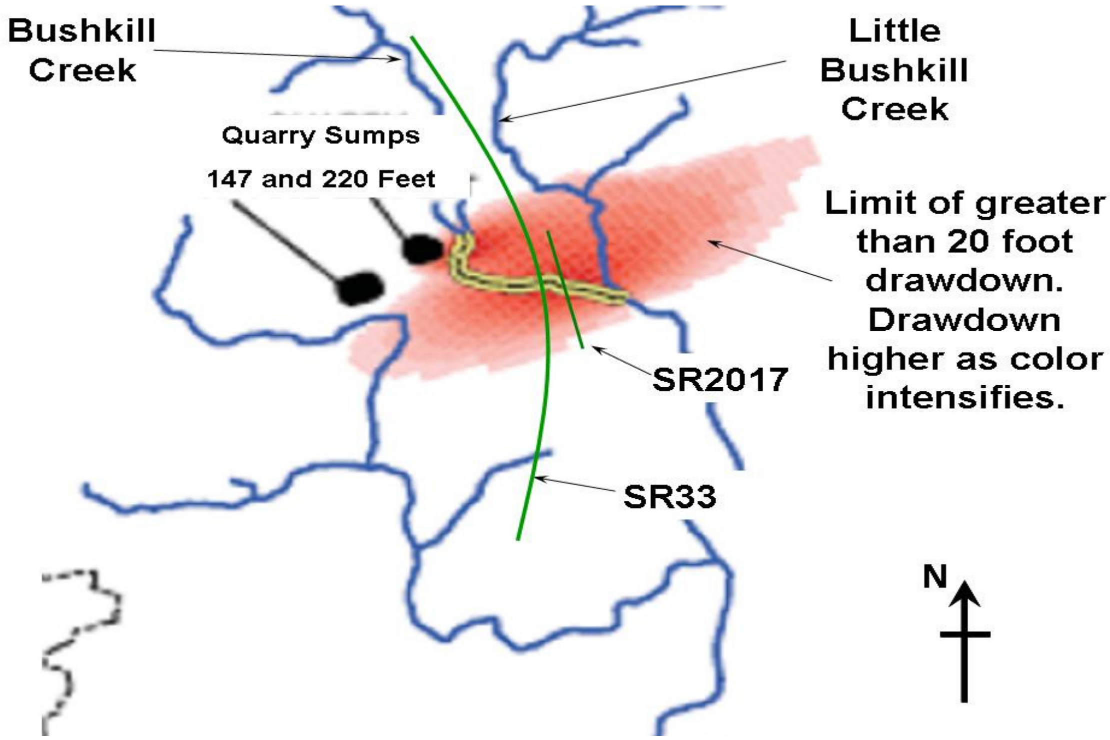

Figure #24 – Impact of Stream Liner (Ref. USGS Open File Report 2006-1143)

The impacts on subsurface conditions of such a large and rapid drawdown include: 1) high increase in effective stress, 2) high seepage pressures, and 3) increased sinkhole activity.

The increase in effective stress would occur from the loss of buoyancy provided by the groundwater. The stress change resulting from a 50 foot drawdown is the equivalent of placing 25 feet of compacted fill over the entire area literally overnight. If the drawdown were 80 feet then it is the equivalent of 40 feet of fill. The change in stress conditions in the overburden would result in soil compression causing foundation settlement of 15 to 24 inches (respectively). This settlement does not include the collapse of the many encountered soft and voided zones within the structure foundations. Due to the highly variable nature of the subsurface, the settlements are not anticipated to be uniform, but rather highly differential within and between abutments.

The permeability of the conduit zone is much greater than the overburden soils. The result is that the groundwater would drop very rapidly, but the overburden soil would remain saturated and drain very slowly. This would create high seepage pressures as the water in the soil tries to drain, but is retarded by the soil matrix. The added pressure would cause additional soil compression, increasing the total resulting settlement. Accurate estimates cannot be made, but are anticipated to be significant. The rate of settlement would also increase during the period of increased seepage pressures, until the overburden soil drains.

During the construction of the existing structures, two of the abutments experienced large sinkholes through the pile groups. These sinkholes certainly imposed unanticipated stresses on the pile groups, that they were not designed to resist. It is not possible to quantify the increased load on the pile groups. Both of the abutments that experienced sinkholes during construction lie in closest proximity to the center of the deep karst formation. Changes in groundwater levels are a known cause of sinkhole activity. It is expected that such large and rapid changes in groundwater elevation resulting from liner operations, would likely yield increased sinkhole activity, with the highest risk areas anticipated to be beneath these two abutments (north abutment of SR33NB structure and south abutment of SR33SB structure).

The cumulative impact of the consequences of the groundwater collapse would be large structure settlements far beyond normal design criteria. In addition, the settlements would not be uniform across individual substructures, or from abutment to abutment of the same span. Settlements would cause increased stresses in both the foundations (micropiles) and the superstructure (abutment and beams). Potentially compounding the problem is that the beams were pinned to the abutments as part of a retrofit treatment to aid in the abutments ability to resist the lateral forces from post construction movements. While this resists backwall failure due to additional horizontal loads induced by abutment rotation, it also increases rigidity.

During construction of the northbound span, the first seven (west side) micropiles in the front (battered) row, were installed significantly shallower than those pile on the east side of that abutment. This was due to continual difficulty in grouting of the micropiles due to rapid groundwater flow. To overcome this problem, larger drilling equipment was brought to the site, and the piles were installed much deeper into more competent rock sockets not subject to the rapid flow conditions. As a result of this condition, under large groundwater drawdown, the north abutment would experience substantial differential settlement from the west side to the east side (see Figure #25). This would increase the existing diagonal crack in the abutment that resulted from previous differential settlements of much lower magnitude. Torsional stresses would also be imparted to the beams and deck.

Since the south abutment of the northbound span is founded on relatively shallow massive rock, the span would experience differential settlement between the north and south abutments (see Figure #26). This would cause the abutments to rotate towards the creek, significantly increasing compressive stresses on the beams and deck. This could cause undesirable cracking should these stresses exceed structural capacity of the beams or deck.

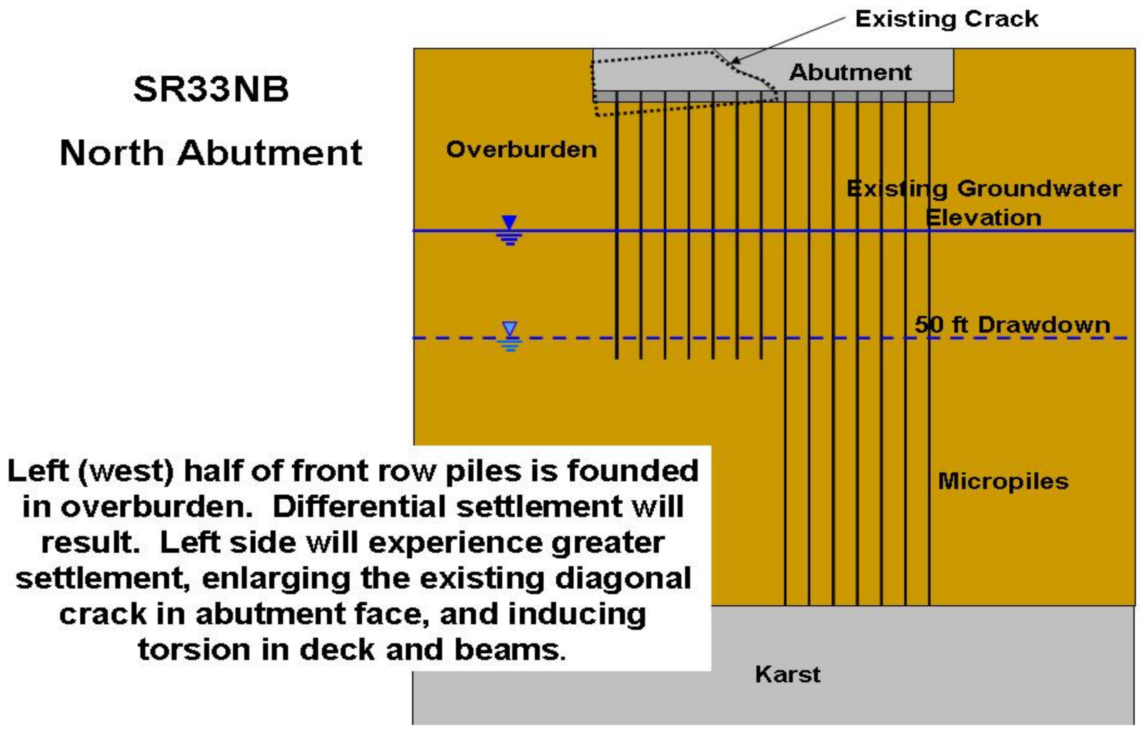

Figure #25 – Impact of Stream Liner, SR33NB North Abutment

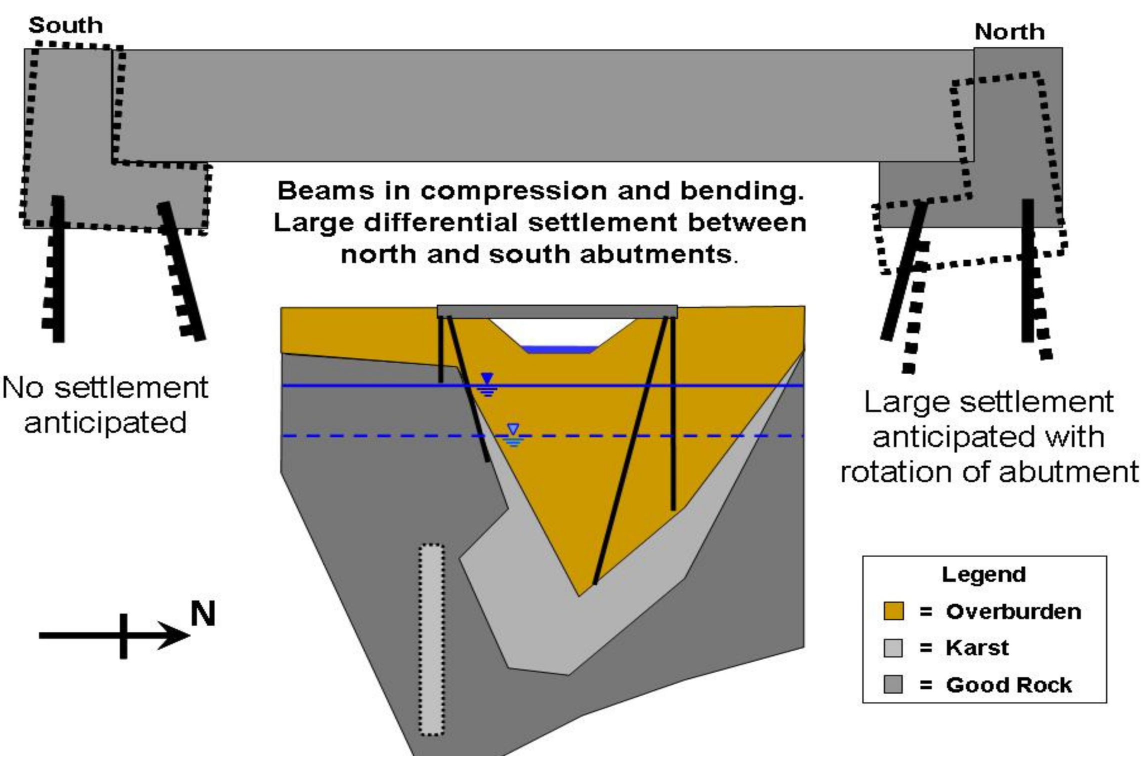

Figure #26 – Impact of Stream Liner, SR33NB, Section A-A'

The southbound span would also experience increase stresses from settlements. The north and south abutments will experience differential settlement between the deeper front battered piles and the shallower better founded rear vertical piles. This again would result in rotation or tilting of the abutments towards the stream (see Figure #27). The settlement would also have a greater impact on all battered piles versus the vertical piles. The settlement would impart loading on battered piles causing bending stresses that were not intended in the design. The top and bottom fixity of the piles (in rock at the bottom and the abutments at the top) would cause the stress to concentrate at these two locations. The impact on the southbound span is similar to the northbound, causing rotation of the abutments towards the stream. In the case of the southbound span, movements would occur in both abutments, but with the more severe condition at the south abutment.

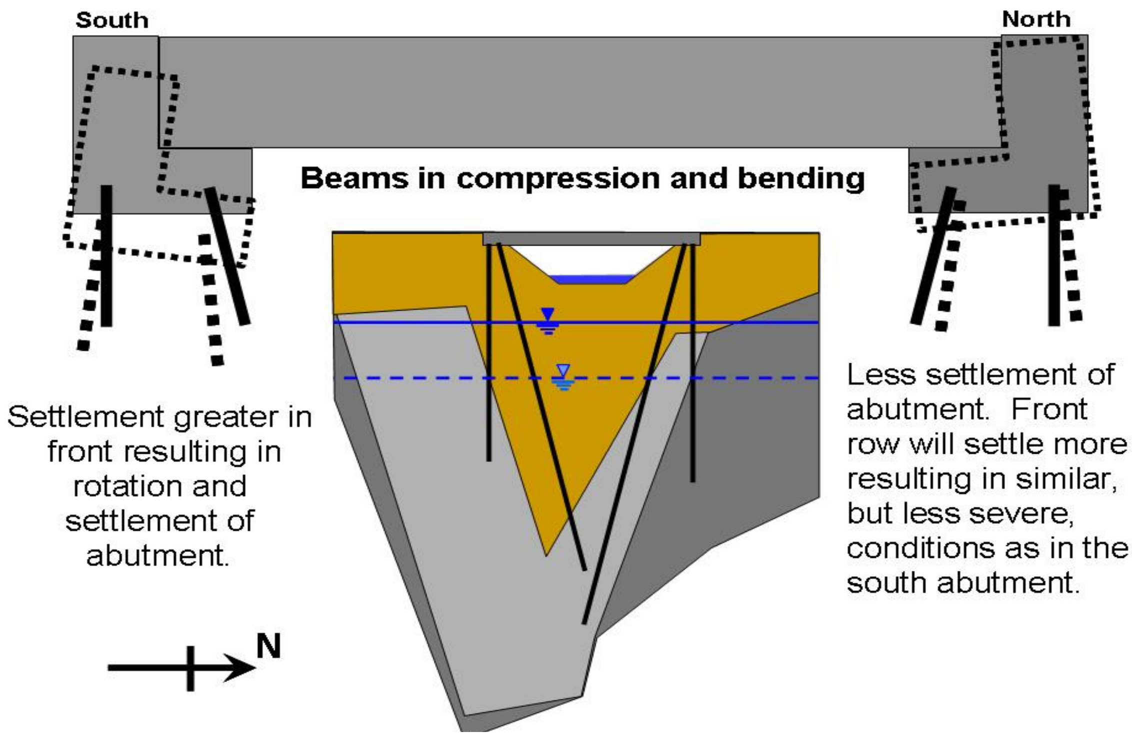

Figure #27 – Impact of Stream Liner, SR33SB, Section B-B'

Additional sources of stress on the piles would result from voided zones along the pile length and from rigid rock "float" that was drilled through to reach the required rock bond zone conditions. This rock float may settle with the soil causing additional stress concentrations in the slender piles.

It is anticipated that significant settlement would also occur in the structure approaches, particularly on the northbound north side and southbound south side, which are closest to the deep karst, and have already experienced settlements necessitating repairs. Further approach repairs will likely be necessary to maintain the highway in service.

In summary, as soil compression progresses, it is anticipated that after a relatively short period, it is unlikely that the structures would be able to remain in service. Rapid failure would be a possibility. While neither lining or not lining provides for protection of the SR33 bridge foundations, and either could lead to damage, closure or failure of the structures, the no lining option is much less likely to result in a near term loss of service. Not installing the liner would provide more time to prepare for potential replacement of the structures, as foundation damage from the erosional process is expected to progress at a slower rate. Redundancy of the pile groups and pinned connection between the superstructure and the abutments would help minimize impacts of localized erosion and sinkhole activity, and should provide greater warning since loss of support for the piles could be observed gradually in surface movements.

Conclusion

While no specific mitigation measures have been identified to improve the situation for either the SR33 structures, or the conditions for the local residents, the nature, extent and dynamics involved with the extensive karst feature have been identified, and a clearer understanding of the problem has been defined. Further investigation work will continue so as to adequately delineate the limits between the heavily karstified zone and the more competent rock. This investigation is intended to provide the necessary information to provide guidance for maintenance of the existing bridges, and to be prepared in the event that continued monitoring indicates the structures may become unserviceable.

The monitoring includes both surface and subsurface measurements and observations. Surface measurements are currently being collected three times a week. General inspection of the structures is conducted monthly. Data from a series of deep inclinometers is collected every two weeks. Collectively these activities provide a means of monitoring the health of the structure (superstructure and foundations), and an indication of subsurface stability.

Of the various investigation methods applied, the relatively simple process of monitoring groundwater for anomalous temperature readings has proven exceptionally valuable. Readings inconsistent with normal groundwater temperatures, provide an immediate indication of rapid surface water infiltration. Loggers that measure water level and temperature have been placed in a number of wells surrounding the structures. Observed groundwater temperatures continue to fluctuate, with seasonal creek water temperatures indicating direct and rapid infiltration of surface waters into the karst conduit system.

Another investigation technique that proved effective and beneficial was the brine tracer study. While more complex and accurate (and more costly) methods of tracing groundwater flow exist, this comparatively straightforward technique permitted verification of projected subsurface flow conditions. The process also permits quantification of flow velocities and mass flow analysis. Combined with traditional boring methods, these procedures provided a comprehensive understanding of what proved to be highly unusual and complex subsurface conditions.

References

Risser, Dennis W., 2006, Simulated Water Budgets and Ground-Water/Surface-Water Interactions in Bushkill and Parts of Monocacy Creek Watersheds, Northampton County, Pennsylvania—A Preliminary Study with Identification of Data Needs, U.S. Geologic Survey Open-File Report 2006-1143.