52nd Annual
Highway Geology Symposium

PROCEEDINGS

May 16 - 18, 2001
Rocky Gap State Park
Cumberland, Maryland

Sponsored By:

Maryland State Highway Administration
Maryland Geological Survey

52nd ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

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Cumberland, Maryland

HOSTED BY:
MARYLAND STATE HIGHWAY ADMINISTRATION
MARYLAND GEOLOGICAL SURVEY

52nd Annual Highway Geology Symposium

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Rocky Gap State Park, Cumberland, Maryland

The Maryland State Highway Administration and the Maryland Geological Survey extend a warm welcome to highway geologists and geotechnical engineers from all over the United States and Canada.

We hope that our symposium increases your knowledge of our craft while enjoying the scenic beauty of western Maryland.

We welcome you to remain in the area after the symposium, to enjoy the rich scenery and heritage of our state.

A. David Martin, Maryland State Highway Administration
Dr. Emery Cleaves, State Geologist, Maryland Geological Survey

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 is 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 Symposia. 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, 51 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, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona where the 13th annual HGS meeting was held. Since then, it has alternated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Annual Symposium 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

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 geologists 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 the relativity 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 for the field trip. The Symposium usually begins with a full day of technical sessions. The field trip is usually the second day, followed by the annual banquet that evening. The final technical session generally ends by noon on the third day. Eastern states favor the spring of the year, while western states schedule their symposiums in the summer and fall 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 (in 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 generating 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 Grand 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 river.

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

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

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HIGHWAY GEOLOGY SYMPOSIUM

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Emeritus Status is granted by the Steering Committee.

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David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
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Sam Thornton
Berke Thompson*
Burrell Whitlow*
Earl Wright
Ed J. Zeigler
Steve Sweeney

*Deceased

HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

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

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

*Deceased

52nd Annual
Highway Geology Symposium
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Thomas Mills	- Field Trip
Gloria Burke	- Facilitator
Murray Miller	- Sessions
Nan Alemi	- Registration
Ann Fisher	- District Office Coordinator
Dennis Debus	- Registration
Michelle Sipes	- Facilitator

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Contents

The Bedford Narrows Rockslide; Investigation, Remedial Design, and Construction

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ABSTRACT

In the early morning hours of November 9, 1997, a large wedge of rock slid out of the cut at milepost 148.6 westbound of the Pennsylvania Turnpike near Bedford, Pennsylvania. The slide flattened a rigid rockfall fence, overtopped the concrete shoulder barrier and covered the shoulder and traveled lanes with large sandstone boulders. The field investigation showed that existing discontinuities, recent modification of the slope geometry, and a heavy rainfall all contributed to this slide. The geologic history of this highly folded and jointed location proved very important to interpreting the measurements of existing discontinuities and understanding the failure mechanism. Stereo net analysis of discontinuities and a three dimensional sliding wedge type of analysis were used to develop and quantify remediation strategies for the slope. The sliding wedge model was verified using the geometry of the November 9 failure and the geometry of a smaller failure that occurred during recent modifications to the slope. Based on the model, the following system was selected as the most suitable system to minimize the potential for future rockfalls.

- A grid pattern of passive, high strength rock dowels was used to provide resistance to large wedge type failures similar to the November 9 failure.

- Drilled slope drains extending deep into the cut face were used to reduce the potential for hydrostatic pressure buildup in the existing joint planes.

- Draped steel cable mesh attached to the dowels was used to minimize rock falls in an area of the slope where joint spacing was very close.

- A wire rope mesh was suspended across an old wedge failure that formed a chute for unstable talus at the top of the slope. The suspended mesh would act as a net to catch high-energy rocks that might overtop or penetrate the edge of shoulder rockfall fence.

This paper summarizes the details of the field investigation, analysis, and design to reduce the probability of future slides at this location. Lessons learned during the bidding process and subsequent construction will be discussed.

1. INTRODUCTION

In the early morning hours of November 9, 1997, a large wedge of rock slid out of the cut at milepost 148.6 westbound of the Pennsylvania Turnpike near Bedford, Pennsylvania. The slide flattened a rigid rockfall fence, overtopped the concrete shoulder barrier and covered the shoulder and traveled lanes with large sandstone boulders. This paper summarizes the details of the field investigation, analysis, and design to reduce the probability of future slides at this location. Lessons learned during the bidding process and subsequent construction will be discussed.

2. INVESTIGATION

2.1. HISTORY

A wedge type failure occurred sometime in the past at this cut, possibly during original construction. The void left by the failure was recorded in inspections as early as 1986. For the purposes of this report, this will be referred to as the old failure. In August of 1996, the roadway at the location of the failure was reconstructed. This reconstruction included widening of the shoulder, cutting the toe of the slope back to provide a rock fall zone and installation of a rigid rock fall fence. During construction, a wedge type failure occurred in the steepened slope. For the purposes of this paper, this will be referred to as the construction failure. The November 9, 1997 failure will be referred to as the recent failure in this paper.

2.2. WEATHER

Rainfall records from the weather station at Pittsburgh International Airport, located about 100 miles west of the site, indicate that about 2.3 inches of rain fell during the 2 days preceding the recent failure. More may have fallen at the site.

2.3. GEOLOGY

The turnpike was cut in to the side of a narrow gap through a ridge formed by the resistant Tuscarora and Bald Eagle formations (See Figure 1). Bedding is vertical to slightly overturned as a result of a steep anticline located to the east of the site. The Tuscarora Formation and the Bald Eagle Formation are separated by the Juniata Formation that

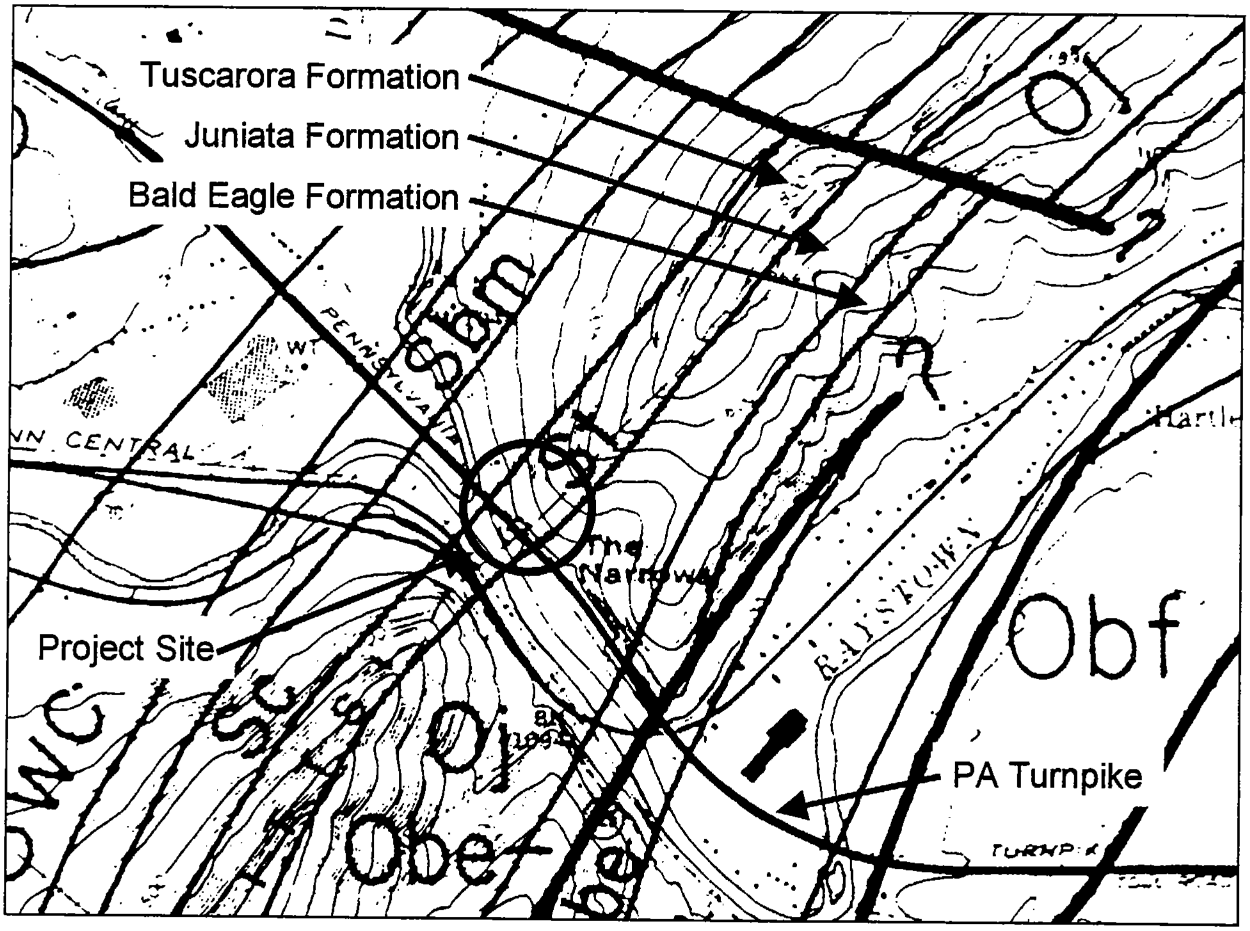


Figure 1 - Site Geologic Map

is less resistant to weathering. Several shear faults that run perpendicular to the axis of the fold are evidenced by the offsets observed in the ridge line. The gap through the ridge is likely the result of more rapid erosion along one or more of these shear faults. The failure occurred in the Tuscarora Formation.

2.4. DISCONTINUITY SURVEY

The entire cut sloped was photographed at a constant distance from the face of the slope. The resulting pictures were assembled into a composite photograph of the entire slope and printed on paper for use in field mapping of the face. All discontinuity orientations, seepage areas and other pertinent features were noted on the composite photo and logged. Discontinuities were also assessed with respect to roughness, infilling, and tightness using criteria developed by Barton.

2.5. LABORATORY TESTING

The unconfined compressive strength of the rock was determined from samples cored out of some of the rockfall debris. Compressive strength of intact material averaged 39,000 psi.

3. ANALYSIS

3.1. GEOLOGIC HISTORY

Understanding the geologic history of the area helps to understand the conditions observed in the cut. During initial compression of the rock strata from the southeast, it is likely that a conjugate joint set formed (See Figure 2). As folding progressed under increased stress, a reverse fault formed across the beds. The fault was offset along its length by numerous nearly vertical shear faults and zones. Folding continued resulting in rotation of the original conjugate joints, reverse fault, and connecting shear faults through an angle of up to 90 degrees. Erosion created the current ridge along the upturned beds of the Tuscarora and Bald Eagle Formations. Erosion of the gap through the ridge produced a stress relief condition that likely caused additional fractures to develop parallel to the gap. These fractures likely developed along pre-existing fractures or weaknesses caused by the original deformation of the rock.

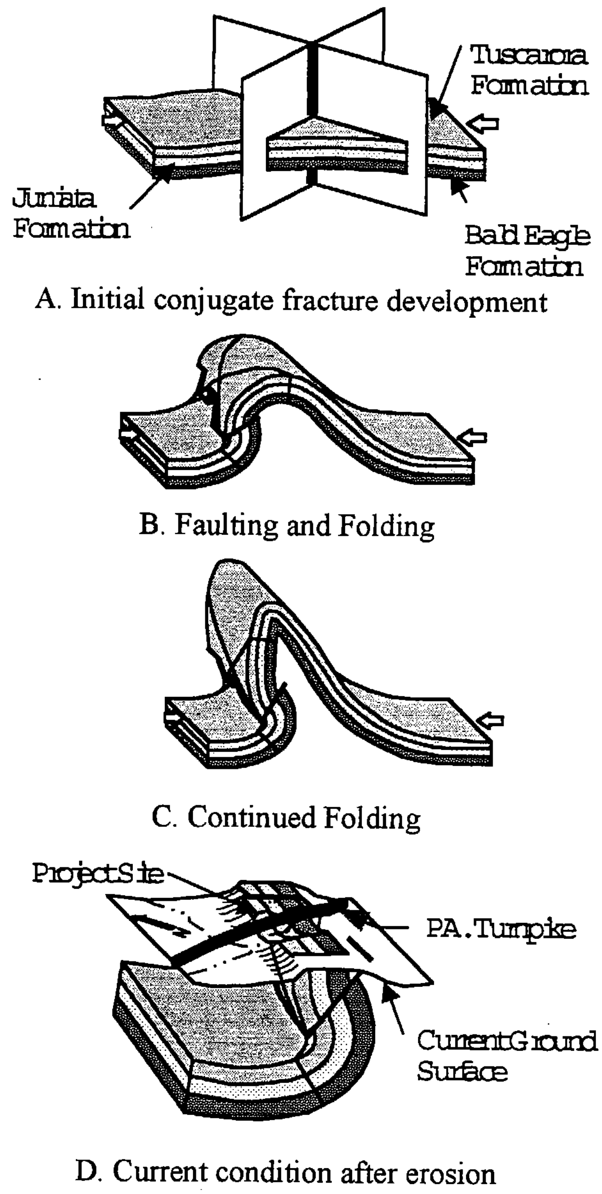


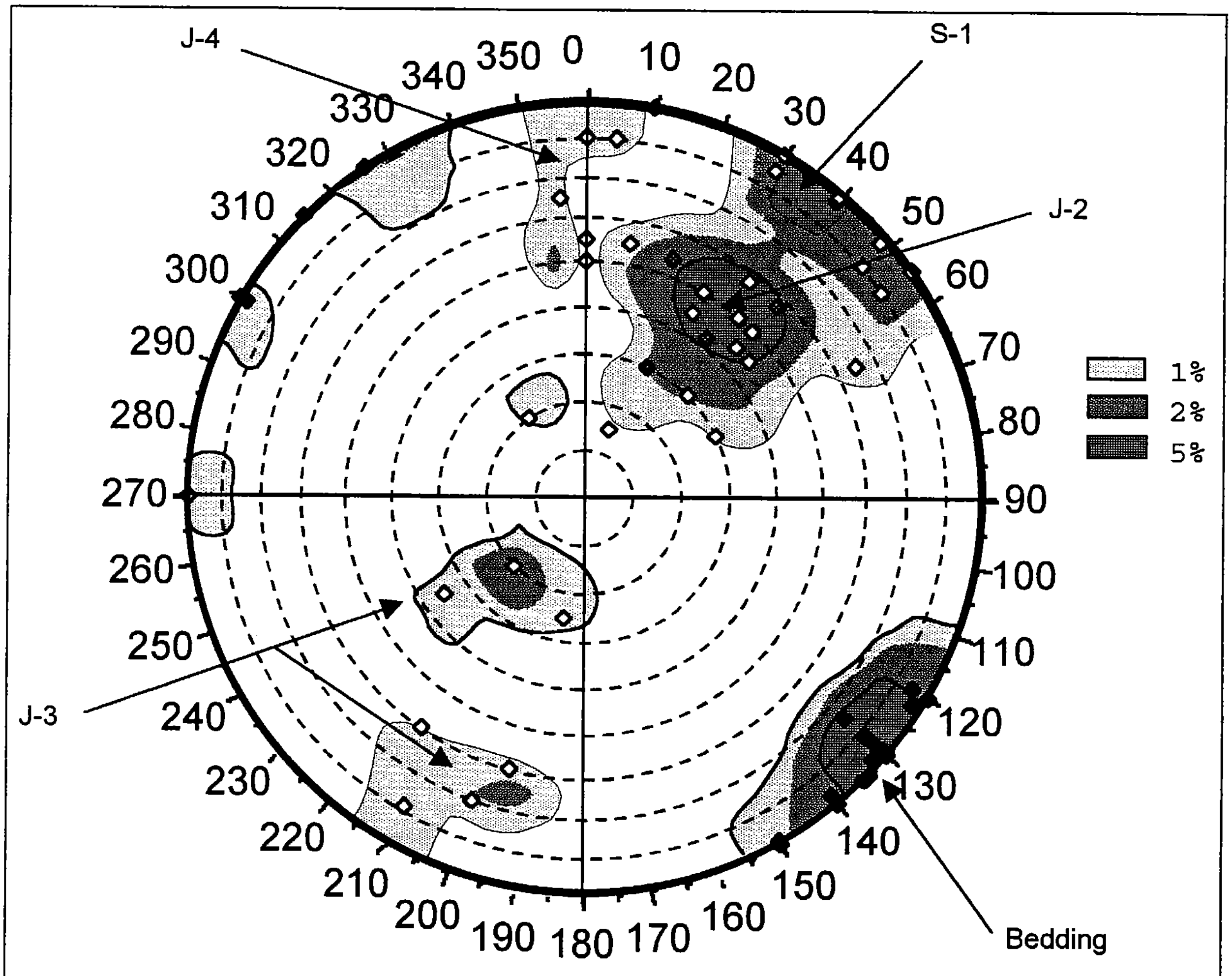
Figure 2 - Geologic History

3.2. DISCONTINUITY DISTRIBUTION

A stereo net plot of the discontinuities is shown in Figure 3. The plot shows five_major concentrations of discontinuities. These concentrations can be correlated to the site geology and history as described below.

- Joint set S-1 is A near vertical joint set striking approximately N 45° W, which is parallel to the road cut. Many of these joints form the existing cut face and show slickensides that indicate past movements. This joint set is probably a result of the shearing perpendicular to the fold axis that offsets the reverse fault as described above.
- Joint set J-2 is the predominate joint set observed at the site. This joint set forms one of the sliding surfaces of the observed failures. Dip directions range from 188° to 237° and dips range from 25° to 65° (based on the 1% contour limits). This Joint set is probably one of the initial conjugate joints formed during initial compression of the rock strata. It has been rotated through about 90 degrees to its current orientation. These joints have probably been accentuated and opened up by stress relief during erosion of the adjacent gap through the ridge, as they are nearly parallel to the resulting valley walls.
- Joint set J-3 is the compliment of J-2. It is a rather poorly defined joint set that dips into the cut face. The wide scatter of poles defining this joint set may be the result of the difficulty finding typical joints to measure in the field.
- Joint set J-4 dips due south at 50 to 80 degrees. These joints all occur in the Juniata formation where wedge type failures have not been observed. However, it is possible that such a failure could occur. The origin of these joints is uncertain and may reflect local conditions since they were only observed at the southeast end of the cut.
- The original bedding forms the fifth discontinuity observed at the site. The bedding is near vertical and generally strikes perpendicular to the roadway. The bedding planes form the second sliding surface of the

observed failures. Dip directions range from 300° to 330° with dip angles ranging from slightly overhanging (95°) to 75°.



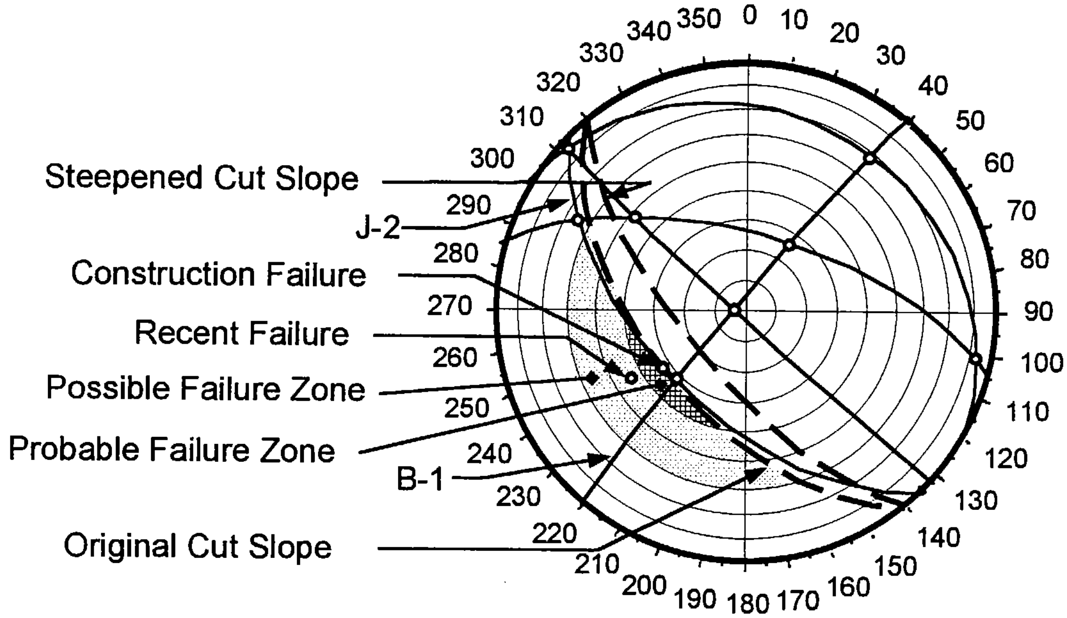
Plot shows poles to planes

Figure 3 - Distribution of discontinuities

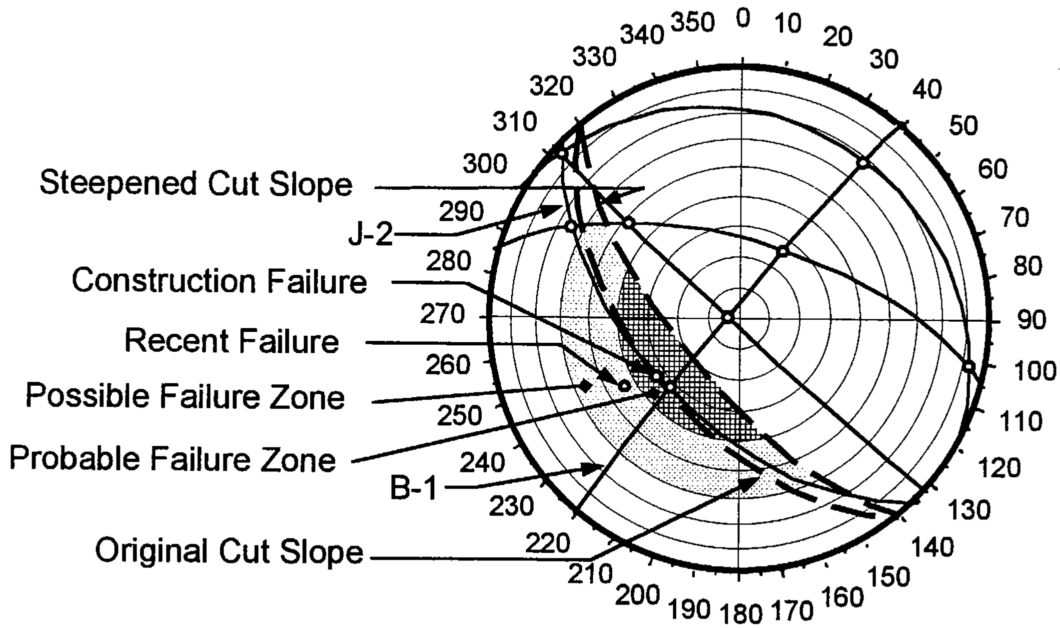
3.3. STEREO NET ANALYSIS OF POTENTIAL FAILURE WEDGES

If the great circles associated with the five discontinuities described above are plotted on a stereo net, the intersections of the great circles represent the plunge of potential failure wedges. Plunges less than 30 degrees are not likely to fail. Plunges greater than 30 degrees but less than 50 degrees may fail. Plunges greater than 50 degrees have a high probability of failure. Since failure can only occur if a wedge daylights on the slope, it is a simple matter to define zones of potential and probable failure on the stereo net. Such a plot is shown in figure 4. Based on this type of qualitative analysis, it can be seen that the J-2/bedding intersection is the only wedge that results in probable failures. Indeed, the three existing failures at the site were all defined by the J-2/ bedding intersection.

The effect of steepening the cut slope by removing material at the toe during the recent construction is easily demonstrated using this qualitative technique. Figure 4A shows the plot with the original cut slope as a control. Figure 4B shows the plot with the effective cut slope after construction as the control. It can be seen that the failure that occurred during construction was a direct result of steepening the slope. The November 9 failure could have occurred without steepening the slope had the joints been located closer to the face. However, it plots in the "possible" zone whereas the failure during construction plots in the "probable" zone. This explains why it stayed in place until a large rainfall triggered it.



A. Possible and probable failure wedges with original cut slope



B. Possible and probable failure wedges with steepened cut slope

Figure 4 - Stereo net analysis of potential failure wedges

3.4. DESIGN ALTERNATIVES

Design alternatives were not directed at fixing the recent failure. In effect, that particular failure had already fixed itself by falling onto the roadway. Since at least 3 major failures have occurred at this cut, all at different locations, it is clear that the failures are a result of unfavorable geologic conditions. Additional failures could occur at any location along this cut given a suitable triggering event. Thus the entire cut needs to be remediated to reduce the potential for future failures. Three alternatives were considered during the design process and are described below.

3.4.1. Flatten Cut Slope

In section 3.3, it was shown how steepening the cut could induce failures. Similarly, flattening the cut slope can prevent failures by not allowing critical wedges to daylight on the slope. Figure 5 shows how a cut slope of 45

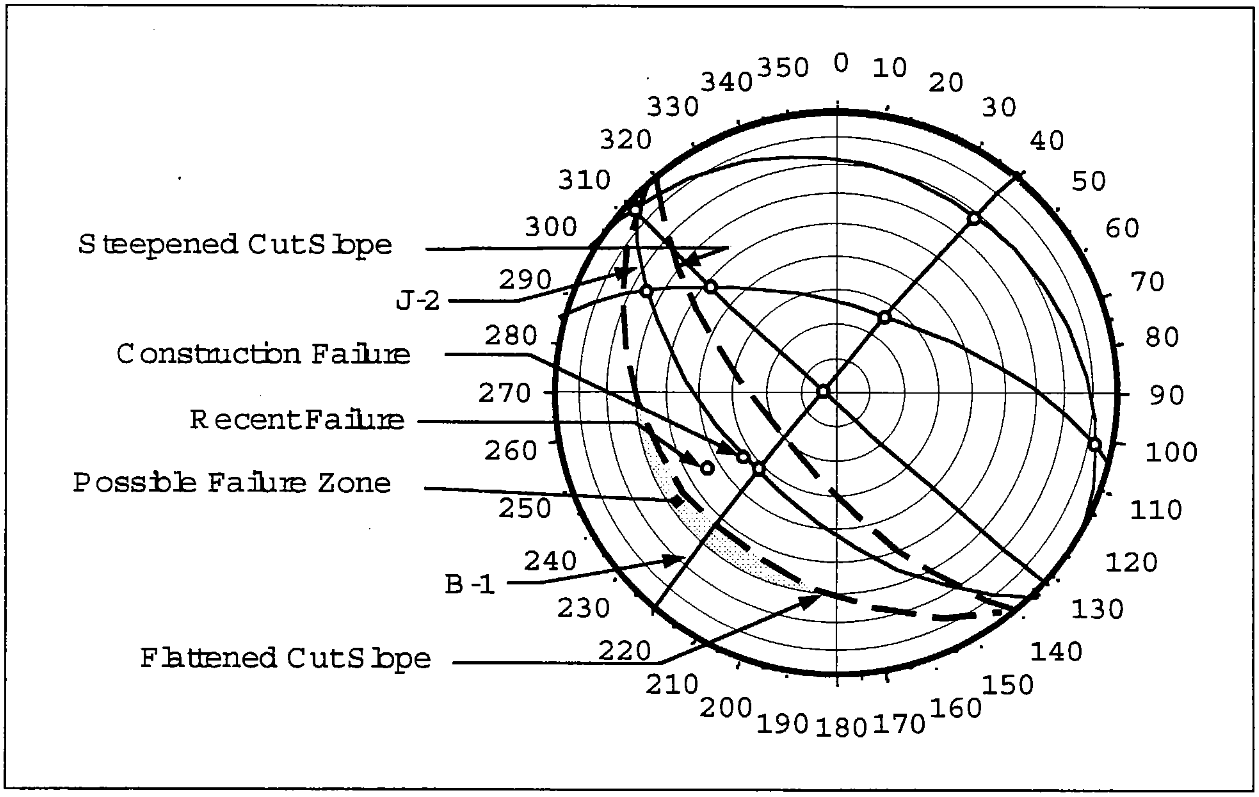


Figure 5 - Effect of flattened cut slope

degrees would have prevented both the construction failure and the recent failure. Though this probably would have been the least expensive solution, it was not pursued for the following reasons:

- It would have required acquisition of a significant amount of additional right of way
- The existing ground continues up at a steep slope above the existing top of cut and is strewn with large boulders. Laying the slope back may have disturbed this marginally stable slope and resulted in boulders rolling down the flattened cut slope.
- Any boulder rolling down the flattened cut slope would have a large horizontal component in its trajectory that would likely cause it to roll into the roadway. Thus some type of containment system would have to be included in the design.
- Excavation would require blasting and a large enough drop zone to contain the blasted material and allow for efficient loadout. As a minimum, both westbound lanes would have to be shut down full time during construction and all lanes would have to be shut down during blasting. The traffic delays associated with this option were unacceptable to the owner.

3.4.2. Protection/ Containment System

The existing rock fall fence was flattened by the recent failure. If a stronger fence could be designed, it may be possible to contain the slide material in the drop zone or roadway shoulder. Based on the geometry of the recent failure, about 1000 tons of material was displaced. Well over 4000 tons of material could be involved in future failures based on the range of joint orientations measured at the site. It was felt that it would not be possible to design a containment system capable of stopping this volume of material even if was only moving slowly.

3.4.3. Stabilize In Place

This option involves trying to prevent future slides by eliminating the trigger mechanism and strengthening potential failure wedges. This was the selected option and included the following specific design elements.

- Drilled slope drains extending deep into the cut face were used to reduce the potential for hydrostatic pressure buildup in the existing joint planes.
- A grid pattern of passive, high strength rock dowels was used to provide resistance to large wedge type failures similar to the November 9 failure.
- Draped steel cable mesh attached to the dowels was used to minimize individual rock falls in an area of the slope where joint spacing was very close.
- The construction failure had created a chute in the face that originated in an area containing numerous boulders and talus. This area was highly unstable and continued to produce rockfalls throughout the investigation stage. Individual boulders loosened by erosion would fall into the top of the chute and impact the existing rock fall fence or concrete barrier. It was not considered possible to remove or stabilize the boulders, thus a wire rope mesh was suspended across the chute. The suspended mesh would act as a net to catch and slow down high-energy rocks that might overtop or penetrate the edge of shoulder rockfall fence.

3.5. WEDGE ANALYSIS

In order to evaluate the effect of various stabilization strategies; including water pressure relief, stabilizing dowels, and slope geometry; a spread sheet was programmed to compute the factor of safety with respect to sliding of a wedge shaped rock mass (see Figure 6). Each wedge was assumed to be bounded by:

- The existing cut slope having a defined height and slope.
- An original ground slope starting at the top of the cut slope and extending to intersections with other bounding surfaces.

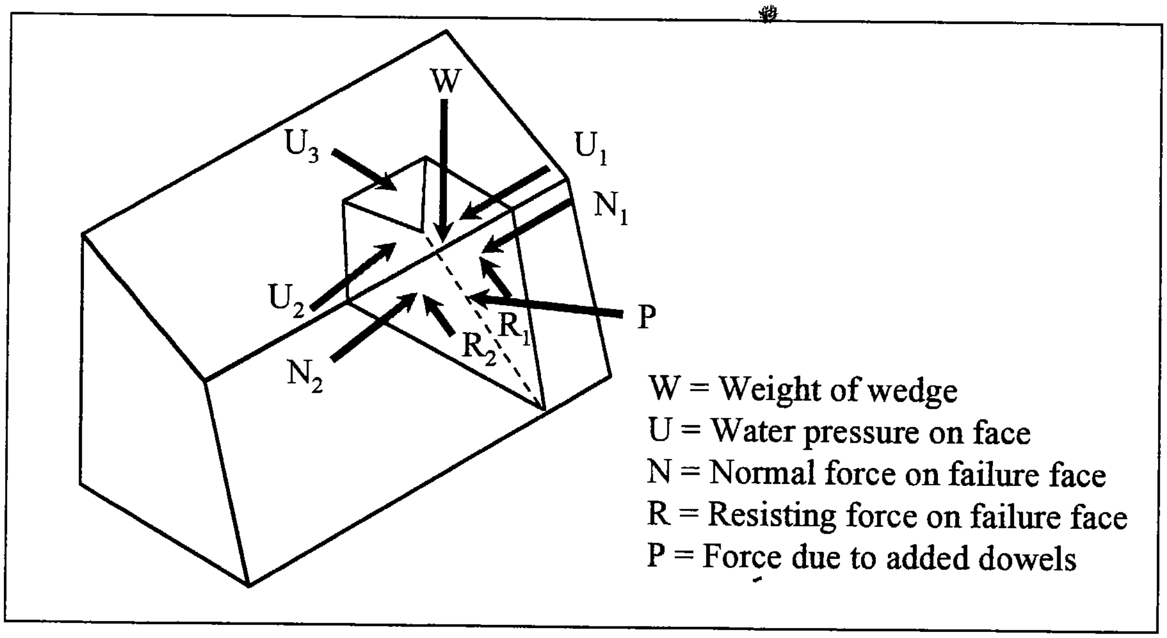


Figure 6 - Typical wedge analysis

- The bedding surface discontinuity having a range of orientations and slope.
- The J-2 discontinuity having a range of orientations and slope.
- A vertical plane parallel to the cut slope and located a defined distance back from the top of the cut slope. This plane is representative of discontinuity S-1.

Forces acting on the wedge were assumed to be:

- The weight of the rock.
- The resolved normal force on the bedding plane and J-2.
- Hydrostatic water pressure acting on the bedding plane, J-2, and S-1 as defined by the height of the water table above the base of the cut and the unit weight of the water.
- The force of rock dowels acting in any vertical or horizontal orientation with respect to the cut face.
- The factor of safety with respect to sliding was computed by dividing the available shear strength on the bedding plane and J-2 by the resolved driving force in the direction of sliding. The shear force on the bedding plane and J-2 was a function of the resolved normal force and the area of the surface.

3.5.1. Strength Parameters

The shear strength on the sliding surfaces was determined using Barton's criteria which considers the normal force acting on the joint, rock strength, joint roughness, and joint condition. The resulting strength envelope is parabolic in shape thus analyses used an effective cohesion and effective friction defined as the tangent to the strength envelope at the normal stress acting on the surface in question.

3.5.2. Passive Dowel Design

Once the force required to obtain an adequate factor of safety was determined from the wedge analysis, a pattern that would produce that force was laid out. The spacing of the dowels is a function of the allowable force per dowel and the force required. The length of the dowel was determined such that the bond zone necessary to develop the tension capacity of the dowel was always outside the limits of a potential failure wedge. The dowels were not pre-tensioned and were grouted full length. Although a small bearing plate was provided at the rock face, the intent of the design is that the dowel force be transferred to the sliding mass via the bond along the upper portion of the dowel. This will assure that the dowel force is transferred to numerous individual blocks of rock rather than a single block at the face of the cut.

3.6. HORIZONTAL DRAINS

Three-inch diameter drainage holes were drilled from near the base of the cut at an inclination of 10 degrees upward. PVC screens were placed in the holes to hold them open. The length of the holes was designed to intercept water in joints well behind potential failure wedges and thus reduce the probability of excess water pressure accumulating on a potential failure plane. Drainage holes were installed after the dowels to avoid grouting the drainage holes shut.

3.7. NET DESIGN

Two areas of slope netting were designed; an area of draped net in the portion of the cut where the Juniata formation was exposed, and a suspended net in the chute formed by the construction failure. All the netting was anchored using eye type fittings that screwed to the top of the dowels. The draped netting was intended to prevent small boulders from falling off the slope between rock dowel locations. The net over the chute was suspended from three large diameter cables strung over the chute and anchored to the rock beside the chute. Sufficient sag was left in each suspension cable to reduce the tension induced in the cable to an allowable value of 1/10 of the breaking strength of the suspension cable. Each net was overlapped with the lower net and the bottoms of the net were secured with tag lines running to nearby dowels.

4. CONSTRUCTION LESSONS LEARNED

Construction proceeded well due to the combined efforts of the contractor, owner and engineer. As with any construction job, a number of issues developed during bidding of the job and subsequent construction. The following are some of the lessons learned on this job that should be applied to future jobs.

- The bid drawings should include a full sized color photo mosaic defining the area of work and items to be done. A plan view or even an elevation view and cross-sections does not adequately convey the nature of the work. Our bid documents did include such a sheet, however when it was reduced half size and printed in black and white for the final bid documents provided to the contractor, it was nearly unreadable and therefore not as useful as a full sized color mosaic.

- Contractor needs to be provided room to work. The owner was justifiably concerned that the work be done with as little disruption to normal traffic flow as possible. However, expecting the contractor to work on a nominal width shoulder at the base of a rock slope is unreasonable. In the end, the contractor was allowed to shut down one lane during work operations and had to reopen it during peak traffic flow periods. Provisions also need to be in-place for complete stoppage of traffic when conditions on the slope may induce a rock fall that could breach the protection system.

- Joints are not necessarily planer features. The entire analysis is based on the simplifying assumption that the joints, on average, can be treated as planes. This is a valid assumption for global analysis but when it comes to individual specific wedges, the possibility of curved joints should be considered. A well-defined joint intersected the cut face just west of the recent failure. Based on projection of the apparent dip in the cut face, the strike and dip of this joint was considered to be too flat to result in another wedge type failure. The contractor elected to scale some of the material above this joint for safety reasons and it was determined that the joint curved upward a sufficient amount to make it a potential failure wedge.

- Do not underestimate the need to scale loose material from the slope prior to other stabilization measures. Not only is this required for safety during construction, but it also goes a long way towards preventing rock falls after construction is done. In this case, both the contractor and the engineer were surprised at the amount of material that was scaled from the slope. In places, the entire cut slope was scaled back 10 feet. At this site, the large amount of scaling may have been a result of over-blasting during initial construction combined with the complex structural geology.

- Slope netting needs to be anchored above top of cut. The top of the cut is usually the most weathered area and rock dowels designed to hold in more competent rock do not work well at this location. Better methods to install a high capacity anchor in bouldery soil above the top of the cut need to be considered.

- Because of problems locating a suitable anchorage for cables near the top of the cut. The cable suspension system was redesigned and reconfigured several times. This work should be considered as part of the job.

- Though the contractor used a heavy duty net suspended from a crane to control and contain rock fall during scaling operations, he left the existing rock fall fence in place to serve as a backup. The existing fence configuration is not based on a rigorous design and has never been tested to determine its capacity to absorb the energy of a falling rock. Observations during construction indicate that this fence was able to contain a large amount of scaled material without failure but its performance under severe impact still is questionable.

ACKNOWLEDGEMENTS

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Management of Rock Slope Hazards

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Abstract

Rock slope problems along roadways in Tennessee are inevitable given the development of the highway system and the local terrain. Varied geology and the large numbers of rock cuts along the roads make rock slope problems difficult to quantify and manage. Previously the Tennessee Department of Transportation has taken a reactive approach to rockfall management.

In order to provide a more active approach to management of rock slope problems a Rockfall Database was developed. The second version of this database is currently under development. Also, fieldwork is underway to provide data and testing of the information gathered. This second database should include components such as

1. *A hazard rating system*
2. *Questionnaires sent to maintenance*
3. *Locations, links and information about photographs, geotechnical reports and boring logs*
4. *Basic Geology and Structural Information*
5. *Contact information for people providing the data*
6. *Preliminary Cost estimates for repairs*
7. *Follow up questionnaires*
8. *Traffic Accident Data*

Fieldwork and adjustment of the database are ongoing. A joint project between the University of Tennessee and the Tennessee Department of Transportation will further expand the database into a Rockfall Management System in the future. Also, GIS capabilities are to be added. When completed, the project will allow TDOT to better plan for expenditures due to rockfall mitigation problems and better address the safety of the public.

Introduction

In 1987 near Winter Park Colorado a serious rock fall related accident occurred. It involved a nearly 7 ton boulder which crashed into a bus causing 8 fatalities and 4 serious injuries. As a result of the accident the NTSB (National Transportation Safety Board) performed an investigation. The NTSB recommended that the FHWA "…issue a Technical Advisory to the various States that describe the accident near Winter Park, Colorado and encourages the States to use a systematic rock fall management program" (1). Following that accident the Oregon state Highway Division, in FHWA Report No. OR-EG-90-0, published the *Rockfall Hazard Rating System Implementation Manual* (2). This was later expanded into NHI courses no. 130220 and 13219: Rockfall Hazard Rating System (3,4). In these courses the FHWA strongly suggested that a statewide survey of potential rock fall sites be performed using this classification system. Any survey of this type would generate a huge amount of data that would need to be cataloged in some fashion.

The problems caused by rock fall do not occur in isolated areas of the state of Tennessee. With the exception of the part of the state West of the Tennessee River, rock falls occur throughout the state. While newer road projects take into account the research that has been done over the last 30 to 40 years, Tennessee is still left with a large number of rock cuts designed under lesser standards. Many of our old roads were built prior to some more modern controlled blasting techniques. Uncontrolled blasting can create blast damage 10 to 20 feet into a slope face. This can provide a seemingly infinite supply of rock fall (5).

In the past Tennessee has employed a reactive approach to rock fall problems. The "squeaky wheel" gets repaired. However, this means that we may be ignoring more hazardous sites because these sites have not garnered as much attention. We cannot define the extent of the problem because we do not know where all of the problem sites are located. This prevents the department from taking a more active approach to mitigation. We cannot allocate money specifically for rock fall repair across the state because we do not know how much we need.

Aside from the monumental task of actually cataloguing all these sites there remains the problem of implementing a systematic approach. We have adopted the Rockfall Hazard Rating System as detailed by Oregon DOT and by the FHWA when sites are evaluated. When individual sites are visited they receive an RHR score based on the system. What remains, then, is a method by which all of these sites are cataloged and our risk vs. cost of repair can be evaluated; not just on an individual project site, but on the statewide system of roads. In their rockfall manual the FHWA suggests the development of a rockfall database to perform this function. While the database developed by FHWA was certainly adequate to this task, changing technology has made their database unusable by many systems running Windows NT.

We therefore set out to design a rockfall database using a more modern program that is compatible with our current systems. Given the cost and large amount of effort required to perform a state-wide survey, we wanted to design a database system that could be useful when this survey had not been completed – or even had not been implemented. This "reactive approach" to rockfall problems mandates a "wait until there is a problem" and then proposes a solution. However, many sites have common problems, and our maintenance forces have knowledge of individual rockfall sites that can be very helpful when designing a repair.

We recognize that it is impossible to fix all of the rock slope problems in Tennessee simultaneously. Therefore we also needed a database that would allow us to define our problems and allocate limited highway dollars more wisely.

So, then, our database has a several fold purpose:

1. Provide a method to systematically catalog and sort rockfall hazard data as a statewide survey is implemented. This database should be able to rank potential rockfall problems by roadway, region and district. If design options for repair are available, the database should also be able to rank these sites using some type of cost vs. risk analysis. Highway budgets are limited and it is an impossible task to "repair" all of the potential rockfall sites at one time. Therefore we need some method to prioritize our work.

2. Provide a method to systematically catalog and sort historical rockfall hazard data. This is useful in the reactive approach because it features a survey sent out to our maintenance forces asking for some basic information (and sometimes photographs) of problem sites. When a rockfall repair is needed, this historical information adds to data at the engineer's/geologist's fingertips. Hazard ratings would be filled out for each site visited. Occasionally, while we perform our site investigation and give recommendations, our designs are not implemented. This is often due to the cost of the projects. The database then gives a reference point if and when that site is given priority.

3. Provide a reduction of liability. A working management system decreases the liability exposure of the state. It demonstrates that the department is trying to address the issue instead of ignoring the problem. The NTSB when it was commenting on California's rockfall management system recognized that it is impossible to fix everything at once, thus an ongoing program was recommended. (1)

A well designed management system should also allow for:

1. Knowledge transfer – the ability to get all of the data in one place to as to get a better "big picture". It should help in the correlation of conditions with problems.
2. Better roadway design - by allowing the tracking of problems over time and by keeping up with mitigation measures used as well as the success or failure of those measures. Allows better feedback.
3. Gets maintenance forces more involved and active. Many maintenance districts within Tennessee may try to handle problems on their own. However, their staff may not be properly trained for this task. Also, with no records kept in a central location we may repeat the same mistakes in many different places.
4. Prioritization of problems and a quantitative measure of hazard.
5. Tracking of historical data.
6. Provide a first look at the cost of mitigation of a site.
7. Records of previous work performed at individual sites. It is not unusual for a site to need to be repaired more than once. Comprehensive mitigation for some sites can be extremely expensive. When a site gets higher priority all of the previous information gathered needs to be easily accessible.

In response to this need a Rock Slope Hazard Database has been developed. This database incorporates the Rockfall Hazard Rating System detailed in FHWA publication SA-93-057: Rockfall Hazard Rating System (3). However, it also includes a number of other components to allow for better management of rock slope problems.

Basic Anatomy of Tennessee's Rock Slope Management Database

Rockfall Location Form

Central to the Database is the Rockfall Location form. This key form describes the location of each individual rock slope we have information about, assigns a file number and provides a link to all of the data gathered about that site. The file number is unique and is made up of the location information. For any file number then, the location of the site is encoded within the file number. It contains buttons to take the user to the satellite forms such as *Rockfall Hazard Rating, Rockfall Questionnaire, Geology Information, Follow Up, Reports/Boring Logs* and *Picture Location.*

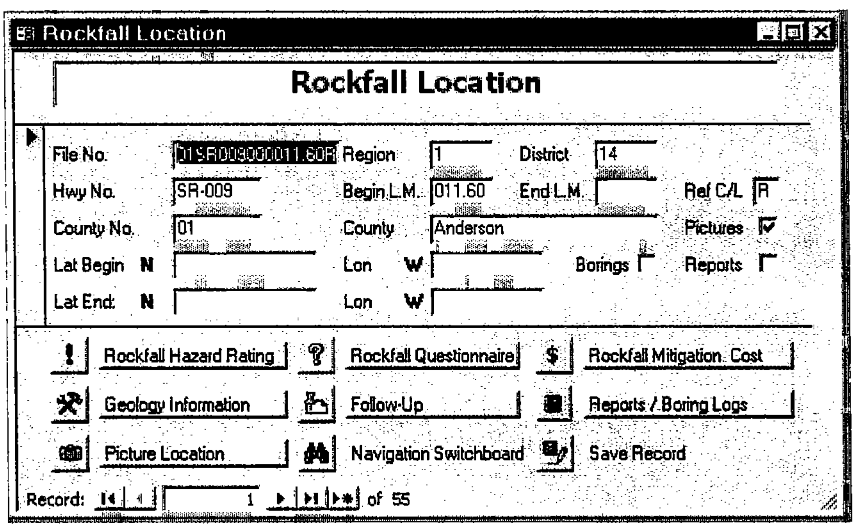
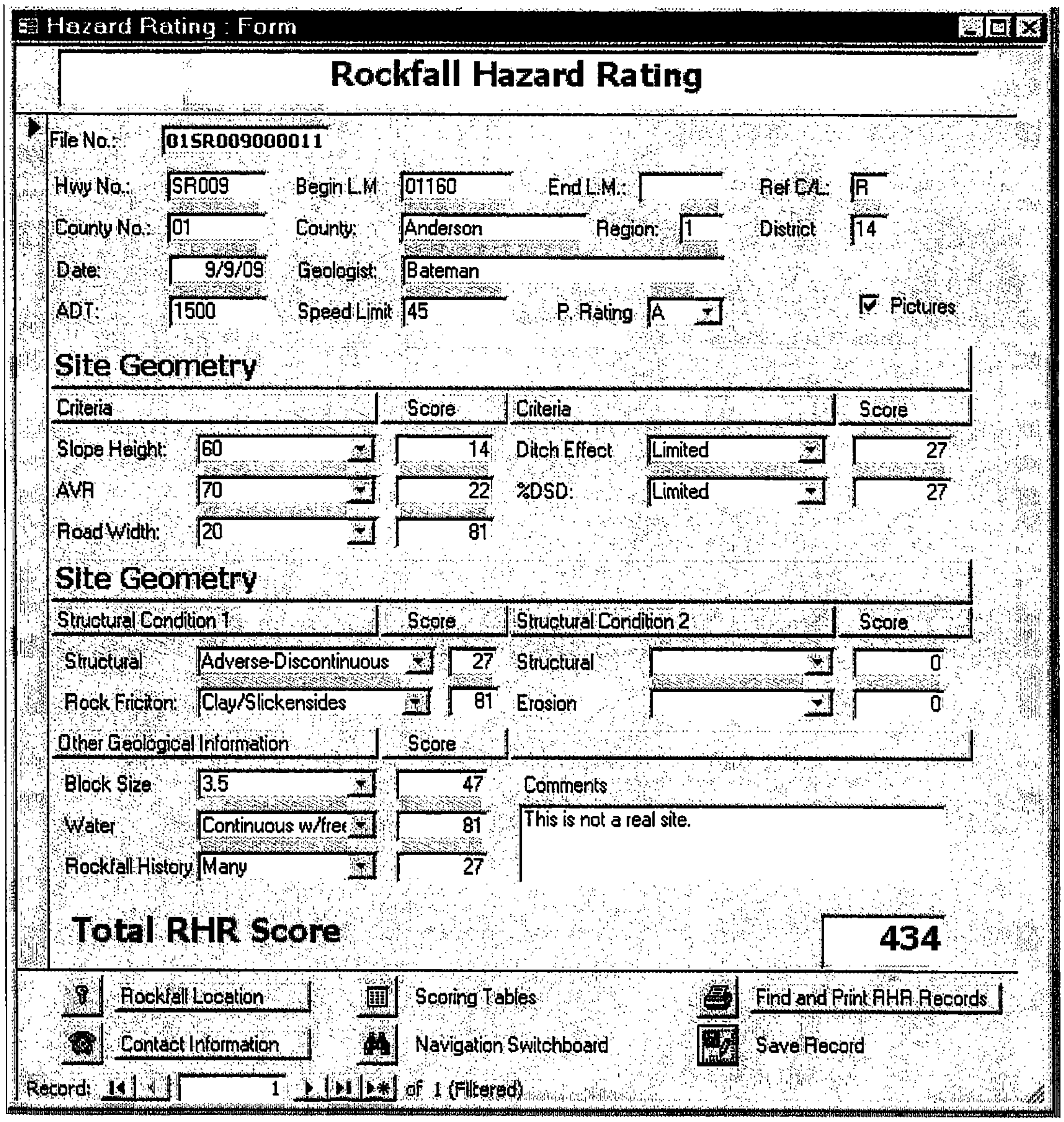


Figure 1. Rockfall Location Form

Rockfall Hazard Rating

The point of this form is to allow for an assessment of rock fall hazard at any particular site. It allows for quantitative measure of hazard so sites can be compared and ranked. The scoring and criteria for this form are taken directly from the in FHWA publication SA-93-057: Rockfall Hazard Rating System (3,4).



Hazard Rating : Form

Rockfall Hazard Rating

File No.: 01SR009000011

Hwy No.: SR009 Begin L.M: 01160 End L.M.: Ref C/L: R

County No.: 01 County: Anderson Region: 1 District 14

Date: 9/3/09 Geologist: Bateman

ADT: 1500 Speed Limit 45 P. Rating A Pictures

Site Geometry

Criteria		Score	Criteria		Score
Slope Height:	60	14	Ditch Effect	Limited	27
AVR	70	22	%DSD:	Limited	27
Road Width:	20	81			

Site Geometry

Structural Condition 1		Score	Structural Condition 2		Score
Structural	Adverse-Discontinuous	27	Structural		0
Rock Friction:	Clay/Slickensides	81	Erosion		0

Other Geological Information		Score
Block Size	3.5	47
Water	Continuous w/fre	81
Rockfall History	Many	27

Comments

This is not a real site.

Total RHR Score 434

Rockfall Location | Scoring Tables | Find and Print RHR Records

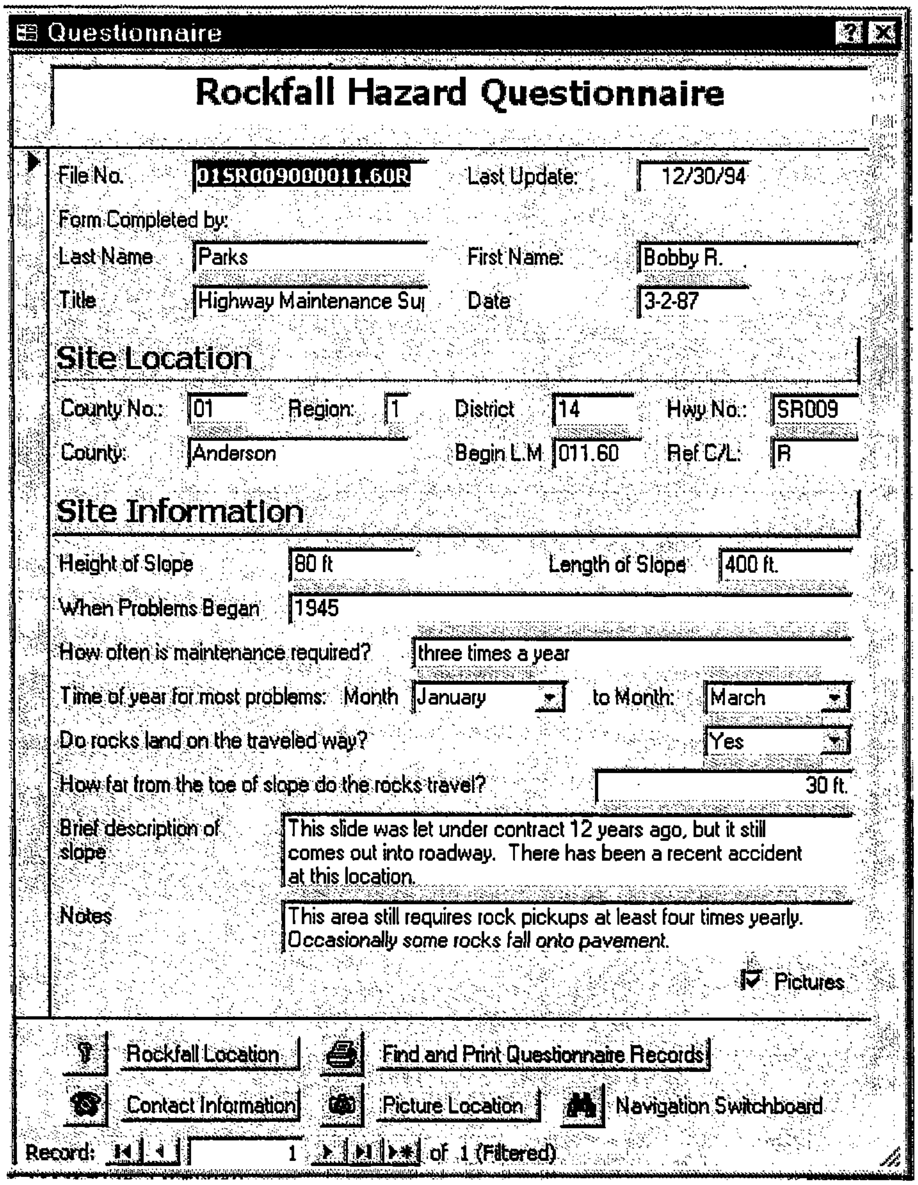
Contact Information | Navigation Switchboard | Save Record

Record: 1 of 1 (Filtered)

Figure 2. Rockfall Hazard Rating Form

Rockfall Questionnaire

Several years ago a form was mailed out to maintenance forces asking them to describe problem rock slope areas within their districts. The questionnaire is designed to capture this information. It is also meant to be a form that is mailed to the maintenance districts periodically. This will allow us to more readily identify problem areas, to assess the frequency of problems at these sites and a view on how these sites change over time. The maintenance people are generally "the first on the ground" they are aware of problems before everyone else. This allows a view of the most troublesome sites from a maintenance perspective.



Questionnaire

Rockfall Hazard Questionnaire

File No. 01SR009000011.60R Last Update: 12/30/94

Form Completed by:

Last Name: Parks First Name: Bobby R.

Title: Highway Maintenance Su Date: 3-2-87

Site Location

County No.: 01 Region: 1 District: 14 Hwy No.: SR009

County: Anderson Begin L.M. 011.60 Ref C/L: R

Site Information

Height of Slope: 80 ft Length of Slope: 400 ft.

When Problems Began: 1945

How often is maintenance required? three times a year

Time of year for most problems: Month January to Month: March

Do rocks land on the traveled way? Yes

How far from the toe of slope do the rocks travel? 30 ft.

Brief description of slope: This slide was let under contract 12 years ago, but it still comes out into roadway. There has been a recent accident at this location.

Notes: This area still requires rock pickups at least four times yearly. Occasionally some rocks fall onto pavement.

Pictures

Rockfall Location | Find and Print Questionnaire Records

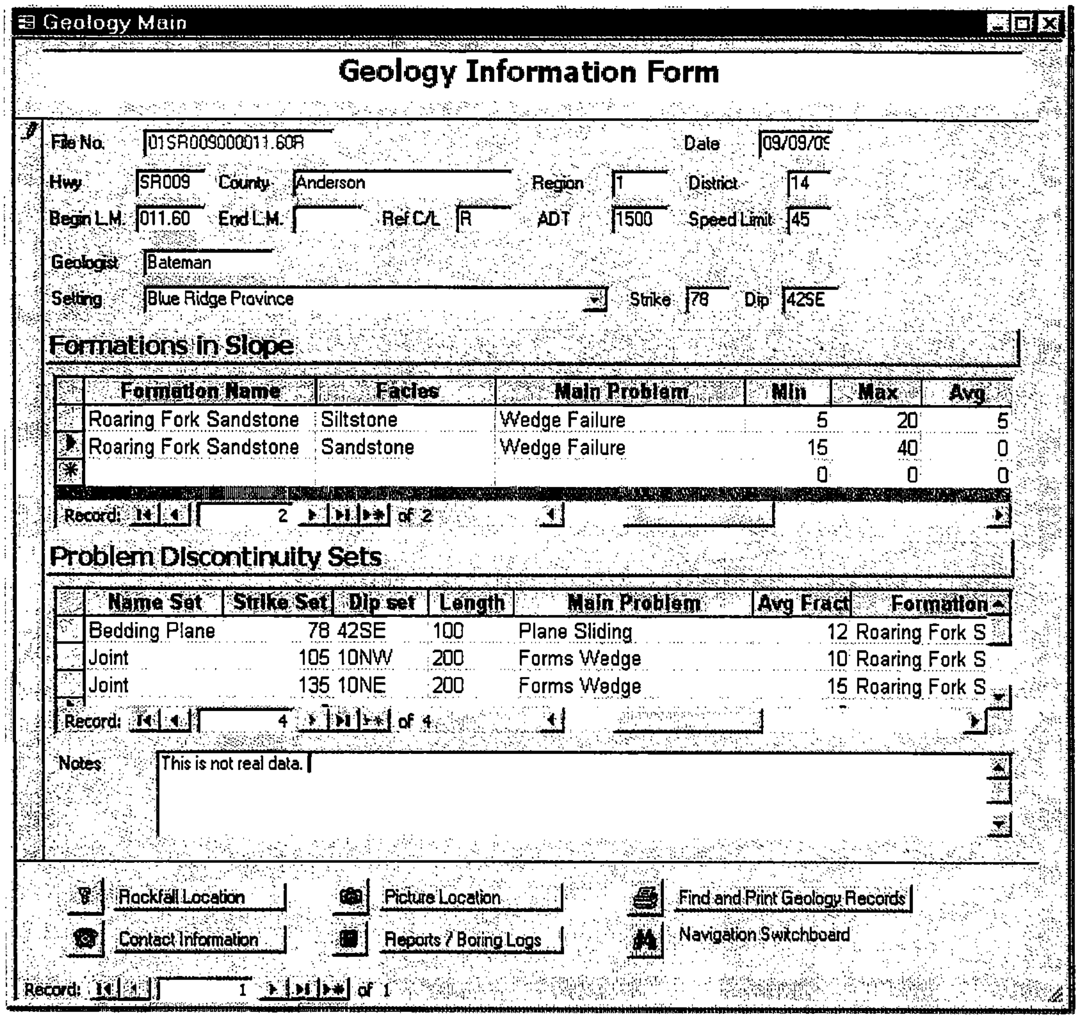
Contact Information | Picture Location | Navigation Switchboard

Record: 1 of 1 (Filtered)

Figure 3. Rockfall Questionnaire Form

Geology and Structural Information

The RHR gives a good method to compare sites and to assess a level of risk for a site. However, it does not take into account what kinds of problems occur in what kinds of geologic settings. The RHR does not record much information on slopes with unfavorable structural features. Often these sites can be the most expensive to repair and the most hazardous. The large rockslides that caused the closing of I-40 at the North Carolina and Tennessee border were caused by unfavorable structural conditions. These slides over the years on the Tennessee side have caused the interstate to be closed for up to two weeks at a time (6,7). The slide on the North Carolina side two years ago closed the interstate for approximately 2 months.



Geology Main

Geology Information Form

File No. 01SR009000011.60R Date 09/09/09

Hwy SR009 County Anderson Region 1 District 14

Begin L.M. 011.60 End L.M. Ref C/L R ADT 1500 Speed Limit 45

Geologist Bateman

Setting Blue Ridge Province Strike 78 Dip 42SE

Formations in Slope

Formation Name	Facies	Main Problem	Min	Max	Avg
Roaring Fork Sandstone	Siltstone	Wedge Failure	5	20	5
Roaring Fork Sandstone	Sandstone	Wedge Failure	15	40	0
			0	0	0

Record: 2 of 2

Problem Discontinuity Sets

Name Set	Strike Set	Dip set	Length	Main Problem	Avg Fract	Formation
Bedding Plane	78	42SE	100	Plane Sliding	12	Roaring Fork S
Joint	105	10NW	200	Forms Wedge	10	Roaring Fork S
Joint	135	10NE	200	Forms Wedge	15	Roaring Fork S

Record: 4 of 4

Notes: This is not real data.

Rockfall Location | Picture Location | Find and Print Geology Records

Contact Information | Reports / Boring Logs | Navigation Switchboard

Record: 1 of 1

Figure 4. Geology Information Form

Each site should have the formations included in the rock slope identified. Major problem discontinuity sets should be identified as well. This will preserve data that can be used in a steronet analysis to get a better grasp of the rock slope problem.

By gathering the general geologic province, the formations present and the problem discontinuity sets with will allow for long term correlation of problem areas with geologic information. After enough data is gathered we may be able to correlate:

1. Certain formations that typically cause rock slope problems
2. "Suites" of formations that cause rock slope problems
3. Rock slope problems by geologic setting
4. Tracking of the success of mitigation methods by geologic setting.

All of these should contribute to better highway design. It allows for the identification of big picture trends, which may not be immediately apparent.

Follow Up

This is meant to be sent to maintenance forces after a stabilization method is implemented. It gives feedback for the success of the mitigation method. This prevents a problem recommendation from being used over and over again because of no feedback. It allows for refinement of mitigation methods.

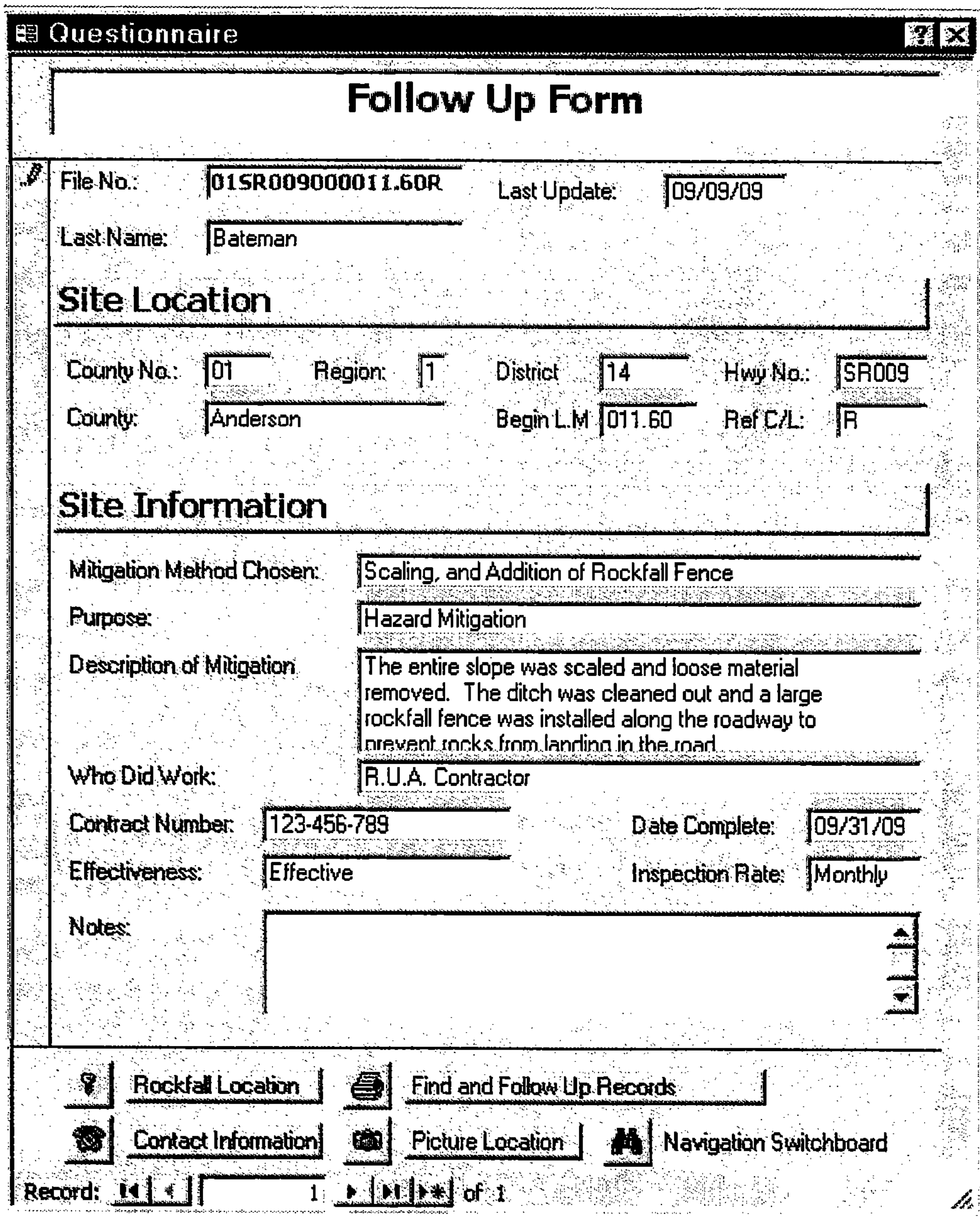


Figure 5. Follow Up Form

The other important aspect of this form is that it encouraged maintenance forces to communicate with others in the department regarding their rock slope problems.

Estimated Mitigation Cost

This form allows for a view of the cost of repair for a particular site. While it cannot replace a more comprehensive cost analysis, it does allow for some comparisons of mitigation costs for different methods at the same site and for different sites. More than one mitigation method can be listed for each project site. This will allow the development of cost benefit analyses to better allow TDOT to spend it's highway dollars wisely.

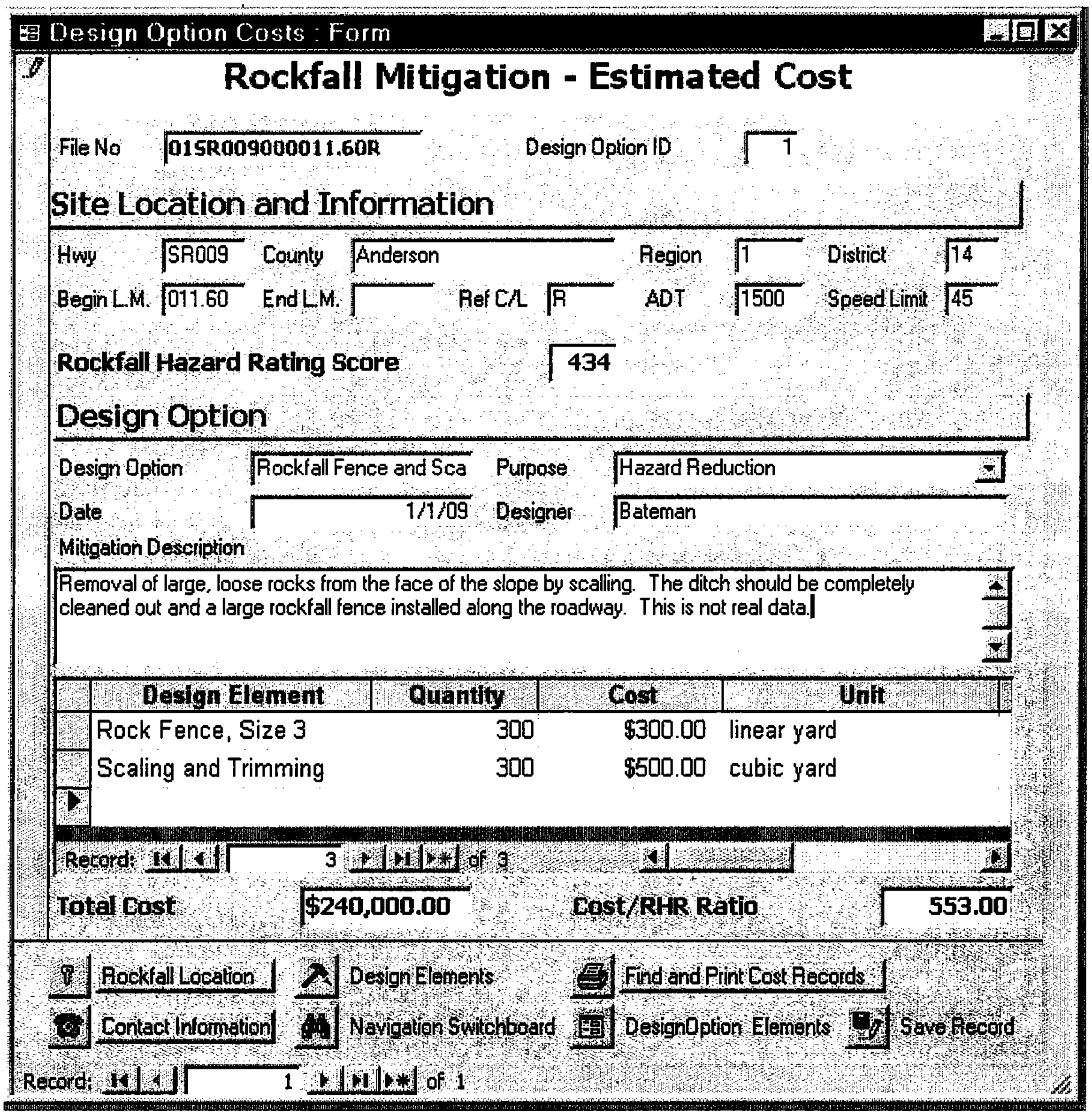


Figure 6. Rockfall Mitigation Estimated Cost Form

Photographs/Boring Logs and Report Location Forms

This allows for the tracking of past work. If a problem reappears, then we know the history of the site and what has already been tried. Also, the preservation of photographs gives geologists and engineers a first look at a site before they arrive. It gives some record of how a site has changed over time.

Contact

Tracking who is filling out the forms can be important if later clarification is required. This allows easy access to the people who are doing the work.

Reports

Various reports can be generated now that the basic structure of the database in place. Reports detailing location by various breakdowns such as County, Region, Maintenance District, Roadway etc. are all easily generated. Each of these can have lists sorted by criteria such as Rockfall Hazard Score, Cost of Mitigation or a Cost/Rockfall Hazard Score.

What we can do with the system after we have gathered data?

One of the first big benefits of the system after the data is gathered is analysis of risk. The most hazardous areas can be identified and ranked. This prioritization of the problem will give us a more systematic method to start mitigating rockfall problems. What is our exposure? What is the extent of our rock slope problems? What are the 10 most dangerous sites in Tennessee? Many questions such as this can be addressed.

We can estimate cost of repairs. This, over time should help with budgeting and cost forecasting. With some idea as to the extent of the problem, a program can be established for repair of these sites. The costs associated with such a program can more easily be quantified. A cost/benefit analysis can be performed to allow for better decision making.

The system should also allow for better "big picture" views of our rock slope problems. We may be able to correlate geologic settings and formations with certain kinds of rock slope problems more easily. We can get a better view on what kinds of repairs work and what kinds don't.

Problem geologic conditions can be correlated with rockfall hazard and with cost of mitigation. This should allow for better roadway design in the future as we are better able to identify and quantify rockfall hazard for problem geologic conditions.

The system will track historical data. We will be able to look and individual sites and see how they have changed over time. It will also preserve records of work that has been performed at an individual site. If that site later has a problem, this will make sure that all of the information we have on the site is located in one easily accessible place.

Maintenance forces will be more regularly involved as Questionnaires and Follow Up forms are sent on a periodic basis. While this will add some paperwork to their workload, it should encourage them to let the Geotechnical Section know when there is a problem.

The management system will also allow us to keep the data in one place so that we can later use other tools, such as statistics and probability analysis.

Where do we go from here?

The next major phase of the project is the gathering of data and full-scale field testing. This field testing should gather data from several different geologic settings. It should also:

1. Assess the hazard rating system to make sure that it is appropriate for Tennessee and our unique geology.
2. Find out how difficult it is to acquire information. How much extra time is required to gather the basic geology and structural information?
3. Determine if we gathering the right geology/structural/hazard data to adequately describe our problem sites?

The additions of GIS to better analyze the spatial distribution of the more hazardous areas and for the production of a variety of maps. Maps can more easily convey to planners why a budget is needed for systematic rock slope mitigation. It can be used to encourage an active approach to the problem.

Tracking of maintenance costs for the clean up and effects of rock slope problems. How often is traffic diverted or lanes are closed due to rock problems? How much material is removed from rock slope sites and how frequently is clean out of ditches or roadway required?

Tracking of traffic data from the department of Safety. Where have accidents caused by rock fall occurred? How does the comparison with traffic accidents compare with the RHR scores of the site?

Integration of some of the data gathered into the TDOT TRIMS (Tennessee Roadway Information Management System) database. This is a database of roadway information available to everyone within TDOT. We want to include enough information to be useful to planners, but not include information that they will not need or might not understand.

Modification of the components based on field gathered data.

Get all of the reports, boring logs and photographs onto a central server so that they can be accessed via the intranet within TDOT. Hyperlinks would be included in the database that will link to these documents.

Conclusion

Because of the structure of the TDOT Rockfall Database, other forms and reports can be easily added as needed. We envision that this database will grow after implementation as we discover other useful information for the decision making process. While the number of users is likely to be limited in the beginning, we hope to make some of the information from the database such as location of rockfall hazards and rockfall hazard scores available to TDOT planners and decision-makers. This information will likely be imported into the TRIMS system and available over the intranet to anyone within the department.

Our purpose was to design a database that will allow TDOT to apply a more systematic approach to managing our rockfall issues here in Tennessee. The inclusion of historical data already gathered should make the database more immediately useful. As data comes in from a more systematic survey of rockfall sites, this database should be of increasing utility for decision making. This accommodates both the active and reactive decision making strategies for dealing with Rockfall issues.

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REDUCING ROCKFALL HAZARD THROUGH ROUTINE MAINTENANCE
A Comparative Approach Using Qualitative and Quantitative Methods

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ABSTRACT

Rockfall hazards occur when blocks of rock become detached from a rock mass, fall under gravity and come to rest in a location where there is high probability of being struck by a passing vehicle. In a highway situation this normally means that the falling rock reaches the travelled portion of the road and drivers must take evasive action to avoid a collision. In a railway situation this means that the falling rock is foul of the track and railroad engineers can do little to prevent contact between rock and train.

In Ontario, Canada, the development of the Rockfall Hazard Rating for Ontario (RHRON) has yielded a quasi-quantitative approach to evaluating the rockfall hazard and assessing the effectiveness of remedial action for highways. A less formal, but equally effective qualitative approach has been developed through work on Canadian Pacific Railway for evaluating and scheduling rock engineering contract stabilization work.

Both the Ministry of Transportation Ontario and CPR have developed maintenance programs to ensure that the hazards that exist (either naturally or through man-made excavation) are reduced as far as possible within appropriate engineering and budgetary constraints. The two approaches are compared and contrasted.

INTRODUCTION

Rockfall hazards are at the very least a headache for administrators trying to ensure safe passage for the travelling public. In the worst-case scenario, undetected rockfalls may lead to serious property damage and even loss of life. Different approaches have been adopted by different organizations in order to mitigate the potential problems that might develop. One obvious option is to carry out a program of routine maintenance at rock cut locations where the greatest rockfall hazard exists. It is apparent, however, that in a rocky terrain there may be hundreds or thousands of rock cuts requiring work. A logical process is therefore required in order to efficiently utilize what are usually limited resources.

Individual organizations may set up a standard approach that operates well for that particular organization. The details for one set of working arrangements may not be directly applicable to another set of circumstances, however, and at the present time there are no known national standards that must be followed for rockfall hazard reduction either in Canada or the USA. Further, different sets of industrial regulations may also affect the way in which work is carried out at different sites depending on the legislation in effect at the time and the jurisdiction of different agencies responsible for, say, occupational health and safety. Within Ontario, Canada, the work that is carried out on slopes adjacent to highways is essentially equivalent to the work required on slopes adjacent to railways. However, there are a number of distinct characteristics that differentiate the two.

This paper starts by describing the two maintenance programs currently in operation for rock cuts alongside highways under the jurisdiction of the Ministry of Transportation Ontario (MTO) and railways operated by Canadian Pacific Railway (CPR). A discussion of the two systems includes comparison and criticism where relevant. The paper continues with a review of some of the pertinent regulations controlling certain aspects of the work and closes with suggestions for creating a more uniform and 'exportable' approach that might be appropriate to other geographic areas.

Ministry of Transportation Ontario

During the latter part of the 1990s, MTO established a rigorous, semi-quantitative approach to rockfall hazard remediation along Ontario highways, Franklin (1997), Senior (1999). Senior described the approach in detail at the 50th Highway Geology Symposium in Roanoke, Virginia. This summary of the Rockfall Hazard Rating System for Ontario (RHRON) has been abstracted from that presentation:

> "MTO has developed a rockfall hazard rating system based on the evaluation of more than 200 high-risk sites in northern Ontario. The RHRON system included parameters that are sensitive to the physiography, geology, and blasting methods used in construction of highway rock cuts throughout the Canadian Shield. It is expected that this system will be applied over the entire province and used for long term rockfall hazard management decisions. Individual sites will always require detailed planning to meet both long and short term objectives." (Senior, 1999, page 283)

In general terms, RHRON combines a variety of parameters into four main factors representing the Magnitude of a rockfall event, the Instability of the rock slope in question, the Reach potential of rockfall material and the Consequences of a rockfall. Table 1 gives a summary of the RHRON factors, the basic questions they answer and the assessment criteria required to establish a formal value of RHRON. Cost estimation is also an integral part of the RHRON evaluation so that management decisions for maintenance can be made either according to the hazard rating itself (relative to the ratings for all other sites under review) or according to the anticipated costs for rockfall hazard reduction.

Table 1. RHRON system factors

Factor	Question	Assessment Criteria
Magnitude	How much rock is likely to become unstable? How much potential energy would be released?	Volume and mass of rock; height, velocity and energy of fall
Instability	How unstable is the material? How likely and how frequent are rockfall events?	Instability mechanisms; face looseness and irregularity' rock quality; shear strength; joint roughness, infilling, persistence and orientation; rock strength and durability; water pressure; seismic potential; freeze-thaw severity
Reach	What are the chances of this rock reaching the pavement? How much of the pavement would be blocked/	Kinetic energy, height of fall; size and shape of debris; presence of launch features; ditch width and depth; history and evidence of previous events
Consequences	How serious are the consequences of the blockage?	Traffic density, highway geometry, rockfall visibility, posted speed, stopping distance; pavement width

(After Senior, 1999)

The final value of RHRON is calculated from parameter 'ratings' (R_n) that have been derived from 'values' (V_n) or 'truncated values' (T_n) for the various 'parameters' (P_n). RHRON is referred to as a semi-quantitative method since although numerical values are attributed to every rating some of the parameters are rather qualitative in form. For example, Parameter 15 is Ditch Effectiveness. The value range is from 0 to 100% (effective) and the specific value is determined from "the probability of any largest potential fall reaching the pavement". The rating is then calculated from R15=0.09(V15). Another example would be Parameter 6, which is Joint Orientation/ Persistence. The rating is estimated directly from a table showing ranges of trace lengths, dip towards face and strike relative to crest. There are four rating values (0, 3, 6 or 9) to choose from and the value used in calculating RHRON is taken as the average of separate estimates for trace length, dip and strike. These parameters, then, are not truly quantitative.

It is not always straightforward to make an RHRON evaluation of a rock slope and considerable experience is necessary in order to achieve repeatable results, especially when considering consistency between evaluators. There are also some other 'quirks' of the system that may give rise to inconsistency, although always erring on the conservative side. For example, the height of the slope is part of two principal factors, magnitude and reach. It appears directly in the estimation of magnitude, and in combination with clear zone width at the site to estimate the crest angle in the estimation of reach. The crest angle is defined as the average angle from the edge of travelled portion of the road to the crest of the slope and it conservatively assumes that the rock face is vertical from the clear zone to the crest. Clear zone width in this case is determined as the width of the shoulder plus the ditch to the base of the rock face.

It is apparent that there are many rock slopes, some excavated, some partially natural, that ascend to considerable heights but set back a distance from the edge of the roadway. These are not nearly as hazardous as a slope of a similar height that rises vertically from the inside edge of the ditch. It appears that RHRON is unable to differentiate between these two situations and a site would be rated as cause for concern based on total height to a potential hazard although loose rocks at that location may not reach the travelled portion of the road.

Regardless of the apparent difficulties in determining RHRON, it has found widespread acceptance by MTO engineers who now have 'a number' to deal with rather than a geologists' description. There are no guidelines as to 'what to do' about particular sites with particular ratings – RHRON lets you know where to put your efforts in reducing rockfall hazard. It still comes down to the experience of a limited number of consultants with 'high complexity in rockfall hazard' competency to design remedial works.

However, the Ontario system has allowed a formal and traceable procedure to be adopted that can be checked against a baseline standard to confirm the veracity of the result. In keeping with good engineering geological design methods, RHRON can be derived if the following steps are observed:

- Detailed Rock Mass Description
- Empirical Rock Mass Classification (in this case RHRON)
- Rockfall Hazard Evaluation
- Rockfall Hazard Reduction

This form of semi-quantitative design process, essentially based on the empirical approach is now being successfully applied by MTO to reduce rockfall hazard on Ontario highways.

CANADIAN PACIFIC RAILWAY

Back in the mid-1970s, a fatal accident in the Thompson River canyon in British Columbia led to the introduction of what is arguably the first formal procedure in North America to assess and reduce rockfall hazard. A freight train ran into rocks, derailed and plunged from the track, across the Trans Canada Highway and into the Thompson River. The two CPR engineers in the lead unit were killed and the ensuing enquiry by the Canadian Transportation Board came up with the requirement that CPR adopt a priority evaluation to determine where rock slopes were located and which ones required stabilization work. At this time, the goal was stabilization of the rock slopes rather than reduction of rockfall hazard.

The process developed by Golder, Brawner and Associates[1] include a four-phase approach to identify Priority sites and the stabilization work required, as follows:

[1] Now Golder Associates Ltd.

- Stage 1 – initial rock slope inventory, system-wide
- Stage 2 – detailed stability assessments for each site, priorities established ranging from A, very high probability of undetected rock fall, to E, very low probability.
- Stage 3 – detailed site remediation specifications prepared
- Stage 4 – construction and associated completion reporting carried out.

In the early 1980s rock cuts at sites that had been constructed in 1870s and 1880s were worked at based on the priority system developed, but it became clear that the notion of 'A to E' priorities was a rather inflexible concept. Should the site retain the same priority after stabilization work? On what grounds would the priority rating change? How would an engineer logically modify the rating previously given? As more and more data were collected (particularly in Western Canada) a sophisticated database was developed to store, manage and assist in evaluating and interpreting rock slope information system-wide. The current system now used to establish priorities for annual construction programs combines an inspection rating with a required action.

Annual inspection forms are created from the database, which includes all of the parameters evaluated in the original Stage 2 reports. These include site identification, traffic volume, alignment and sight visibility, climatic conditions, and past stability record. The site is described in terms of type, height, length, geological description, evidence of water and workspace available. The type, magnitude and seriousness of the potential instability are recorded. In the original Stage 2 reports, the site sheets also included details of recommended stabilization and a photograph of the site. This information has not been transferred to the database.

The engineering and management of CPR's rock slopes now follows procedures established in their 1997 Directive, which is described in detail in Morris & Wood, 1999. Of particular note are the four categories of methods used to describe "rock slope mitigative measures"[2]:

- Stabilization – methods based on preventing rockfalls from taking place
- Protection – methods based on protecting the railway and its equipment from damage from rockfalls
- Warning – methods based on detection and notification that a rockfall has occurred or is on progress
- Observation – methods based on visual detection.

Mitigation is implemented on both a short- and long-term strategy. The short-term strategy involves a limited range of methods designed to respond to impending hazards. The long-term strategy is implemented to meet longer-term objectives related to hazard reduction and service requirements.

Perhaps the most important development of the CPR system is the use of the inspection rating/required action table. Observations made during annual inspections of rock slopes adjacent to the track across the country and in upstate New York are used to assign an inspection rating, which describes the potential for a rockfall to occur. A required action is also specified, which defines the time frame in which remedial work should be carried out. The relationship between the two is presented below as Table 2. It can be seen that although the process is essentially qualitative, the ability to provide a consistent approach is maintained since the sites

[2] CPR, Engineering and Management of Rock Slopes, 01/97, foreword

are reviewed on a regular basis and previous years' reports and ratings are consulted in establishing annual evaluations.

Table 2. Inspection Ratings and Required Actions

		REQUIRED ACTIONS						
		Limit service Work within 1 month	Follow up inspection	Lockblocks or clean ditch	Work in current year	Work in 1-2 years	Long term	No action
INSPECTION RATING	Urgent	X	X	X				
	Priority		X	X	X	X	X	
	Observe						X	X
	OK							X

So, in contrast to the more formal procedures used in MTO projects, the process developed for use on CPR rock slopes has the following principal steps:

- Rock slope inventory
- Rockfall history
- Annual rock slope inspection
- Planned rock slope remediation

Comparison of approaches

The main differences between the two approaches relate to the management practices in the two organizations and the regulatory framework within which each operates. MTO is responsible for over fifteen thousand kilometres of highway in the province summarized in Table 3 below. The predominant classification is two-lane, rural highway with a posted speed of 80 km/hr (50 mph).

Table 3 Ontario highway classification

Number of Lanes			Posted Speed		
Number	Length	Percentage	Speed	Length	Percentage
8+	90 km	.5 %	50	80 km	0.5 %
6	345 km	2.1 %	60	85 km	0.5 %
4	1,635 km	9.9 %	70	30 km	0.2 %
2	14,390 km	87.5 %	80	10,780 km	65.4 %
			90	3,800 km	23.2 %
			100	1,680 km	10.2 %

In recent years there has been a shift away from engineering and maintenance work being carried out by MTO personnel. Most design work is now contracted out to consulting engineers under a prime engineer responsible for "Total Project Management". Geotechnical work is commonly sub-contracted, as is foundation engineering, under which rockfall hazard evaluation falls. Companies proposing to carry out work for the Ministry are required to be registered and are only allowed to carry out certain aspects of the work program depending on the specific skill set of the company, or employees. There are approximately 15 registered prime consultants involved in highway reconstruction and rehabilitation. They bid on refurbishing contracts for sections of highway in the 20 to 30 km length range that require resurfacing every 20 to 25 years or so. The geotechnical work associated with frost heaves, drainage, embankment stability, culverts, etc. is sub contracted to one of about 10 registered geotechnical companies. Rockfall hazard evaluation and remedial design is sub contracted to one of 4 registered professional engineers with expertise in this area.

The management of projects is coordinated through the MTO's regional offices in Northwestern, Northern, Eastern, Central, Southern and Southwestern Ontario. There are so many different engineers potentially working on any particular project that the Ministry has elected to follow procedures that are clearly auditable and as rigorous as possible. This led to the requirement for a semi-quantitative Rockfall Hazard Rating for Ontario. There are just too many different workers involved in projects to allow subjectivity to enter into the design process. The whole system has to be transferable from one engineering group to another without risking loss in 'due diligence'.

In comparison, CPR operates an interprovincial, linear transportation corridor entirely run under its own auspices. The thousands of individual rock slopes identified and described in the original Stage 2 reports are situated alongside track in parts of British Columbia, Alberta, Manitoba, Ontario, Upstate New York and Pennsylvania. The management of the annual program to review and remediate rock slopes is undertaken by the Geotechnical Engineering group based in Calgary, Alberta – a team of three engineers under an engineering manager. Assistance in all areas of engineering, inspections, rock slope remediation design, contract preparation and administration, and planning is provided by a very limited number of technical specialists with post-graduate training in rock mechanics and rock engineering. Contract stabilization work is carried out by one of only four, pre-qualified contractors. There are three from British Columbia and one from Quebec. They work across the CPR system.

For the railway work, then, there is a small, close-knit group of managers, engineers and contractors who have highly specialized experience in dealing with rock slope stability issues for CPR. Using well-trained and experienced personnel who have worked together over much of the last 20 to 25 years ensures the quality of the program. The members of the team have a series of formal steps to follow that have been refined over a period of at least two decades, so there is less need for a more rigorous semi-quantitative process to be implemented. The rock engineering program is 'fine tuned' on an annual basis – funding is made available for one calendar year only and priorities have to be set within the constraints of engineering need as well as finances – within the overall planning of approximately five year increments.

Audits are carried out each December to determine how effective that year's program has been and what the highest priorities should be for the upcoming year. Final decisions for stabilization work are commonly left until the spring inspections have been carried out to allow inclusion of sites that may have changed over the winter period. The four methods (Stabilization, Protection, Warning and Observation) are again mentioned as appropriate methods for the removal of rockfall hazard on the railway.

Practical Comparisons

What is most interesting in considering the differences between the two programs is the way in which contract construction work is carried out. For MTO projects the work is referred to as "Rockfall Hazard Reduction", while for CPR projects "Rock Slope Stabilization" is usually undertaken. For MTO, "a rockfall hazard is defined as any potentially unstable rock mass that, upon failure, is likely to come to rest on the trafficked portion of the highway pavement."[3] On CPR projects, contracts are let for "rock scaling and stabilization" at various locations.

There are many cases, particularly in mountainous terrain in British Columbia and northern Ontario where highway and railway alignments run on adjoining rights of way. Indeed, there are frequent instances where a rock mass has an excavation for a highway on one side and a railway on the other. Under these circumstances the rock mass, the climatic, and the water table conditions are identical. Original construction practices are likely to have been different since the highways were commonly constructed up to 50 years after the railways were pushed across Canada. But in many cases the intrinsic conditions are similar. Present-day construction practices, however, are quite different.

Rockfall Hazard Reduction

On highway reconstruction projects, rockfall hazard reduction measures generally comprise removal of potentially unstable rock blocks and rock masses from the rock face (what would be described as 'stabilization' in CPR work). As noted above, the predominant highway classification is two-lane rural highway with a posted speed of 80 km/hr. Full highway closures are not normally permitted, so rehabilitation work must be carried out during lane closures only. This usually means that a highly mobile workforce is needed. In addition, the regulations concerning construction activities currently in force[4] are highly restrictive when dealing with workers actually climbing a rock face. Thus, the commonest forms of rock removal are:

- Mechanical Scaling – using either a large excavator (CAT 235 or equivalent) or a hoe ram (550 kg-m or 4,000 ft-lb impact hammer)
- Trim Blasting – using a two-person man lift, crane or other suitable equipment capable of accessing rock faces up to about 25 metres (80 feet) high
- Manual Scaling – also using a two-person man lift as above, and occasionally
- Bulk Blasting – to remove significant volumes of overhung material, for example.

'Protection' of the highway is carried out at specific sites where other forms of rock removal are unfeasible. These forms of construction may involve:

- Rock bolting,
- Shotcreting,
- Concrete buttress construction,
- Draping of wire mesh,

[3] Quoted from "Request for Proposals for Total Project Management (TPM) Detailed Design and Construction Contract Administration Services, Agreement Number PO-5005-A-000039" June 29, 2000, Ministry of Transportation Ontario, page 34.

[4] "Occupational Health and Safety Act and Regulations for Construction Projects" June 2000, Ministry of Labour Ontario.

- Installation of protective barriers (Jersey barriers, Lockblock™ walls, Brugg® fences), and the
- Installation of drains – many of which require a two-person man lift, crane or other suitable equipment capable of accessing rock faces up to about 25 metres (80 feet) high, or
- Excavation to clear zone – creating a catchment zone wide enough to give 95% retention of an unanticipated rockfall event. Full clear zone for the majority of the highways in Ontario is at least 7 m (23 feet) on tangent.

Warning fences are not commonly used on highway projects, and there are none known in Ontario. Observations of rockfall events are not commonly recorded in a formal way anywhere in Ontario within the highway network, so future planning cannot rely on past events to assist in rational design.

The conclusions that may be drawn from the MTO work is that the work force is equipment-oriented, using lift vehicles (such as JLG, or Snorkelift) and heavy equipment to carry out rockfall hazard reduction. As such, there are a limited number of highly experienced contractors and the degree of expertise on these projects is limited. Very careful contract administration is required to ensure that the contractor is able to carry out the required work safely and efficiently.

Rock Slope Stabilization

Perhaps the most significant physical difference between the two working environments is the basic right-of-way. On a highway, the traffic can manoeuvre to a considerable degree and there is the chance that vehicles may avoid obstructions by steering around them or stopping. This is far from the case with railways; swerving is an impossibility and stopping a long freight train from 45 mph can take almost half a mile. Thus, if a rock does become detached, fall to the track level and end up 'foul of the track' there is little that can be done to avoid a collision. The right-of-way is usually about 50 feet to either side of centreline, so widening cuts is rarely an option, even if it were economic. The original construction was largely completed by the driving of the last spikes in 1885, so most of the slopes are well over a hundred years old, were constructed using antiquated blasting methods, and have been exposed to thousands of freeze-thaw cycles.

Many of these rock masses have considerable volumes of rock that could be removed but with limited available finances, the selection of what actually needs to be done has to be considered very carefully. Since the vast majority of the territory is crossed by single-track line with sidings, it is most common to "occupy the track" during construction and integrate stabilization works through the rail traffic controller to interrupt train traffic as little as possible. For these varied reasons, men working from ropes with climbing harnesses has developed into the preferred *modus operandi* in stabilization work. A combination of manual scaling, rock bolting, dowelling and ditch excavation has become the standard approach at many sites, while shotcrete, Lockblock walls, Brugg fences, cable bolting, concrete buttresses, etc. have been used where required.

The standard work crew is five men (one superintendent/backhoe operator, and four scalers) together with a CPR flagman to arrange track time and maintain radio contact with rail traffic controllers and others. Since the railway is interprovincial, the company that operates it and other corporations whose main role is to support railway operations fall under Federal jurisdiction. The Federal regulations for work requiring 'fall arrest systems' or 'fall restraint devices' are less specific than those given in the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. This is the principal reason that railway stabilization work does not use cranes, etc., which have been found to restrict the ability of a scaler to carry out his work effectively. Indeed, only in the Province of British Columbia has the legislation dealt specifically with rock scaling. The current regulations from the Workers Compensation Board of B.C. allows the use of a single rope rappelling system, provided that the rope remains under tension at all times.

The stabilization construction work is therefore handled by a highly mobile crew working from ropes, with specialized suspended staging (known as Spider cages) to carry out al work above track level. Limited working space commonly restricts the size of equipment that can be used, since the tack has to be completely cleared for the passage of trains. Large equipment is rarely employed except for large-scale excavations where a considerable volume of material must be removed. Even in these cases a small excavator-backhoe may be the most useful piece of equipment.

Although the workers are not 'protected' be a second means of fall arrest (as appears to be required by interpretation of Ontario legislation) accidents involving scalers using ropes and climbing harnesses are more rare than they are when using boatswain's chairs and lifelines.

Conclusions

Reducing the hazard to the public when travelling on Ontario highways, or reducing risk to successful operations of Canadian Pacific Railway involves carefully planned and implemented routine maintenance. For the highways it requires evaluating the potential hazard using the Rockfall Hazard Rating for Ontario, then establishing a remediation program with heavy equipment and operators with limited experience under the watchful eye of a construction contract administrator. Provincial occupational health and safety regulations apply. For the railways it requires establishing priorities for rehabilitation based on routine inspections by a limited number of experienced and qualified specialists, followed by remedial construction using pre-qualified contractors experienced in scaling and other rock stabilization techniques and working from ropes using climbing harnesses and other related equipment. This may appear to be labour intensive, but the successful results are a testament to the efficiency with which this method can be applied. Federal occupational health and safety regulations apply.

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GEOPHYSICS BEFORE HIGHWAY CONSTRUCTION

by

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Abstract

Many problems in highway construction can be avoided if certain subsurface conditions are identified before bid requests. Important questions that can be answered easily using near surface geophysics are depth to water table, depth to rock, rippability of rock and the existence of sink holes, caves or old mines. This paper discusses the value of seismic refraction and gravity to enhance the knowledge of the subsurface before bids are requested. The cost of a geophysical survey would be recovered by the reduction in required borings and possibly lower bids if some potential hazards are ruled out.

Seismic refraction will determine depth to water table and bedrock and the degree of rippability of the rock. Both seismic refraction and gravity surveys will indicate sink hole, buried valley or collapse areas. In addition a gravity survey will locate caves or other voids that are not indicated by the seismic survey. Drilling frequently misses enough subsurface problem zones to cause major errors in highway or construction planning. These errors either require additional work or the highway fails after completion. Either way, both the owner and the state or federal highway department suffer financial loss.

Introduction

Frequently, companies bidding on highway construction contracts have little detailed information about the geology along the route. The cost of the necessary prebid geologic study would generally be a part of the company overhead. Unknown geologic problems have frequently been very costly. To safeguard against these unanticipated problems the company's bid is higher than may be necessary if the route is known to be free of major geologic problems.

General geologic maps and information are available for most of North America but specific information is commonly lacking. Questions of importance for highway construction that are not usually answered by the available information are:

- What is the depth to water table?
- What is the depth to rock?
- How irregular is the water table surface, the bedrock surface?
- Can the rock be removed with heavy equipment or must it be blasted first?

- Is the highway going over old land slides? If so, will construction activate the old slides?
- Are caves or sink holes present? If so, how big and where?

To our knowledge these questions are frequently answered by the construction company after the contract is awarded but the bid has frequently been inflated to cover unforeseen problems. The answers are obtained by site geologic studies and borings. Both of these techniques, though necessary may miss features that could greatly increase the cost of construction. Some simple near-surface geophysical surveys can greatly reduce the number of required borings and increase the confidence of the knowledge of the subsurface. It is our belief that the cost of near surface geophysics will be offset by the reduction in the number of borings required. One need only list a few of the problems that have occurred in the past to emphasize this. Some representative situations where inadequate subsurface knowledge has led to severe construction problems are:

- Failure of a highway caused by a mine collapse,
- Activation of an old landslide,
- Finding bedrock depth that was not adequately shown by borings,
- Expecting to be able to remove rock with heavy equipment but had to blast to remove,
- Local fill material lacking adequate strength,
- Inadequate drainage of the substrate.

We can characterize most of North America by five types of geologic terrain:

1. soluble rock,
2. insoluble sedimentary rock,
3. igneous and metamorphic rock,
4. glacial sediments,
5. coastal sediments.

The location of each of these terrains are fairly well known but the details of specific sites are not. Later in this paper we look at problems common to each terrain.

This paper discusses two near surface techniques that aid in the engineering design and can be useful in almost all conditions. We assume that the location of the highway to be built and the maximum allowable grade is known. If so, seismic refraction and gravity surveys will greatly improve the subsurface knowledge and thus allow the construction company to more carefully evaluate the project cost. Ground penetrating radar, resistivity, electromagnetics, and magnetics, though useful, are more subject to local limitations.

Seismic Refraction

The instrumentation for seismic refraction is readily available and companies that can perform this survey are widely available. The method entails putting vibrational energy into the ground (a hammer or weight drop device) and measuring the arrival time of the first waves to reach geophones (motion sensors) at increasing distances from the source.

Many books describe this technique. We suggest Burger (1992) as a beginning discussion of the technique.

The first wave to arrive at a particular geophone will either be the direct ray or a refracted ray. Figure 1 shows the paths of the direct and the refracted ray. The energy travels through the ground obeying the laws of geometric optics. Thus the ray, when it hits an interface, will be refracted toward the interface, if going into a higher velocity media. At the critical angle the wave will be refracted along the interface. As it moves along the interface, this wave will generate waves that travel back to the surface. Because this refracted ray is traveling much of its path in the higher velocity media, the ray reaching distant geophones will arrive before the direct ray even though the refracted ray travels a greater distance. Plotting the arrival of the first energy on a graph of time vs. distance from the energy source will produce straight lines if the material is homogeneous and the interface is planar (Figure 1). Interpretation of the time-distance graphs will produce useful information to the engineer. It is important to know that it is necessary to gather data in both a forward and reverse direction. These time-distance graphs can then be used to determine true velocity of the direct and refracted rays and the degree of slope of the interface between layers of material in the subsurface. Because the plot is time vs. distance the inverse of the slope is the wave velocity of the media. In general the following can be assumed. Unsaturated material will have a velocity of less than 5,000 ft/sec, saturated unconsolidated material will have velocities from 5,000 to about 6,500 ft/sec and most consolidated material have velocities above 6,500 ft/sec. This velocity information can be used to determine how difficult it will be to remove the material. If it is above 9,000 ft/sec it generally cannot be ripped without blasting. See Figure 2 for rippability information. If the arrival times of the refracted ray produce straight lines, it means the refracting interface is planar. If the plotted curve is irregular, it means that either the upper layer is heterogeneous or the refracting surface is irregular. In addition, depth to water table, depth to consolidated rock and information on the character of the materials can be determined from the plotted curves. This low cost technique does not eliminate drilling test holes but does greatly reduce the number necessary.

Gravity

The instruments needed for a gravity survey are readily available. A gravity meter and careful elevation control are needed to gather the field data. Many near surface geophysics companies can do gravity surveys. The survey entails measuring the acceleration due to gravity at a series of stations along the highway route. The elevation of each station must be surveyed to a 0.1 ft accuracy

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Acceleration due to gravity is controlled by elevation, latitude, position of the sun and the moon, and density of the material beneath the survey line. The effects on gravity of all of these factors except the last can be removed from the data by careful collection techniques and mathematical correction in the office. A low or a high acceleration due to gravity at stations along the line will indicate density differences in the subsurface materials. For example a sink hole filled with soil will produce a density deficiency

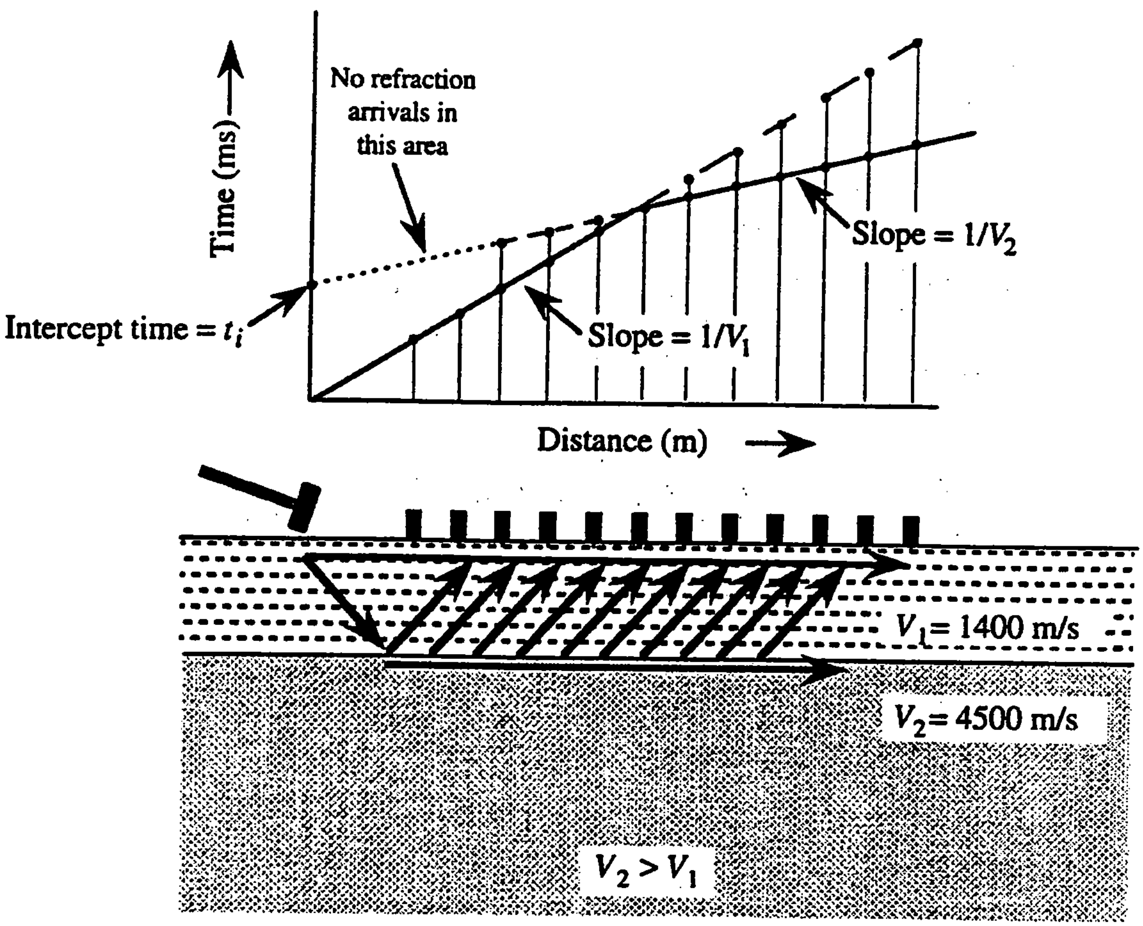

Figure 1. Seismic refraction basics. The lower portion is a schematic representation of the ray paths for a simple case of soil over a horizontal bedrock surface. The upper portion is the time-distance graph for this configuration (Burger, 1992).

relative to the surrounding limestone and thus a gravity low. This low will be greatest over the center of the sink hole. All density variations beneath the station will contribute to the anomaly. Therefore, the effects of deep earth materials must be compensated. There are a number of standard ways for the interpreter to accomplish this. The degree of confidence in the near surface interpretation is high, particularly with a few borings for verification. This technique is good in locating features such as sink holes , caves, buried valleys, faults and fault zones.

Interpretation

Geophysics surveys require interpretation by a person who is familiar with the limitations as well as the capabilities of the techniques. The interpretation have a greater uncertainty in terms of specific quantitative results than borings that provide the familiar subsurface information used by most construction projects. Geophysical results provide more continuous but less precise information on the subsurface than borings. The boring may provide precise value but only at a specific spot. Geophysics provides the extended view of the subsurface conditions that can easily be missed between borings. A combination of more widely spaced borings with nearly continuous geophysical information between provides a cost effective means for optimizing the trade off between precision and thorough coverage. Many times geophysics has provided valuable subsurface information but has been applied without a proper understanding of the limitations of the methods. Mechanical application of the techniques without proper consideration of local conditions and limitation of the interpretation can lead to bad results. In this paper we are attempting to show that the most cost effective way to construct a highway is to use geophysical survey data along with geological studies and borings. If this combination of data is given to the construction firm, that intends to bid, they will know better the problems associated with the project and will be able to produce a more efficient bid. It will not be necessary to build as much contingency cost into the contract to cover unknown conditions. The bidder will know better answers to questions such as:

- Can the cut materials be used as fill material?
- What slopes are necessary to be stable?

The geophysical survey will be a small fraction of the construction costs and we contend the total cost will be less if a survey is properly done and interpreted because other costs or uncertainty about potential costs will be reduced. The particular features of geophysical surveys and the information they can provide in each of the five geologic terrains are discussed in the following sections.

<u>Soluble rock</u>

In soluble rock regions(Figure 2), where carbonate rock is the dominant bedrock and characteristic karst terrain may develop, drilling and sampling frequently misses many important anomalies. Karst terrain is characterized by lack of surface drainage patterns and the existence of sink holes, caves, and a very irregular bedrock surface covered with highly variable thicknesses of red clay soil. Seismic profiles will indicate the irregularity of the bedrock surface. This irregularity will include buried valleys, sink holes, and fault

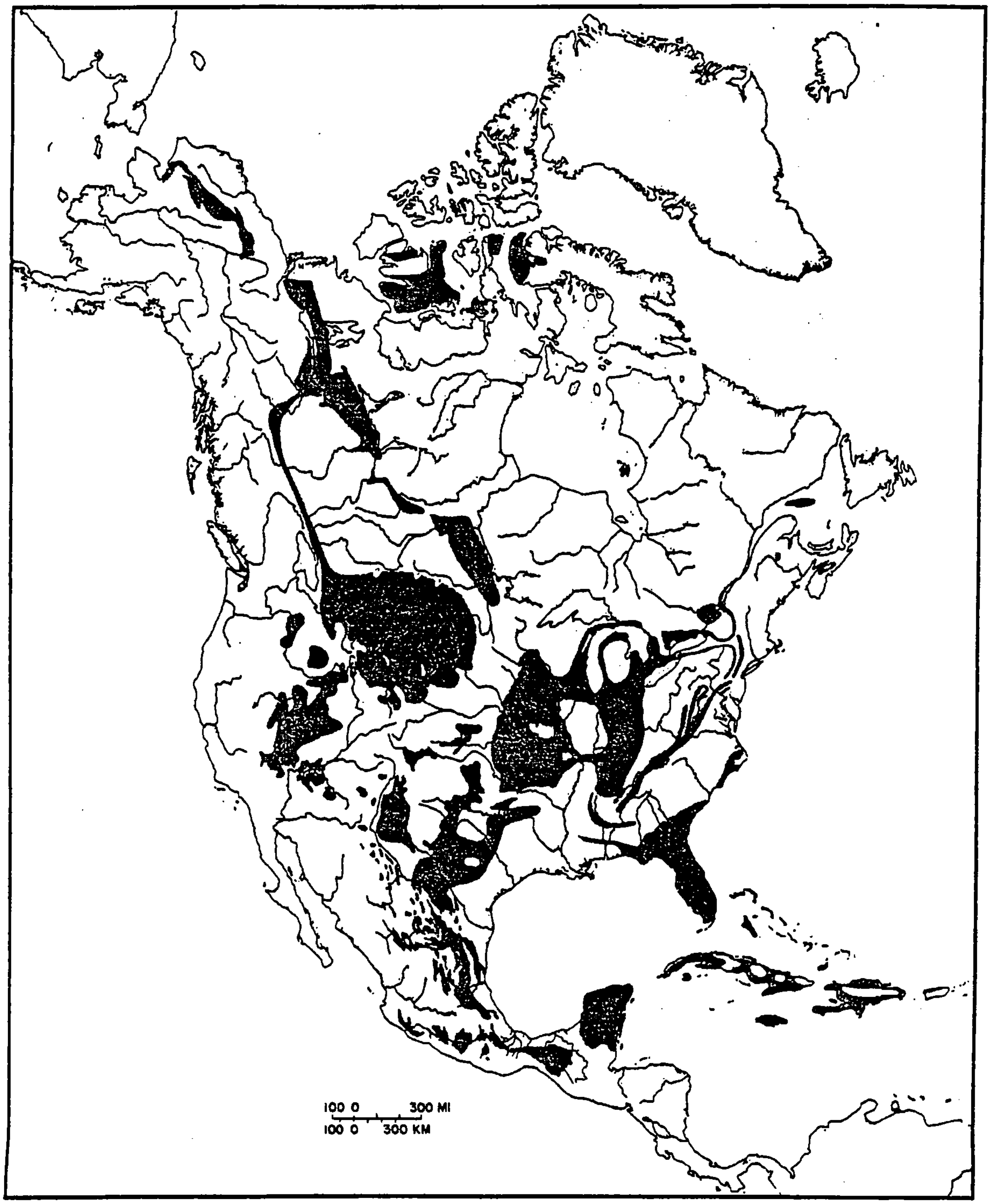

Figure 2. Map of North America with regions of soluble rock (carbonate and gypsum) indicated in gray (Heath, 1988).

zones. Gravity will identify sink holes, buried valleys and caves. If only gravity is used, it will be difficult to tell whether the anomaly is a cave, sink hole, buried valley, or fault. But if both gravity and seismic refraction are employed, the ambiguity is greatly reduced.. Other geophysical techniques can be quite valuable at some locations but these methods are more subject to difficulties produced by development such as fences, buried pipes etc.

Insoluble sedimentary rock

Sedimentary rocks form the bedrock of most of the United States. These rocks range in hardness and consequently in difficulty to excavate. Softer rocks can be removed by ripping equipment. If blasting is necessary, the cost will be considerably higher. Seismic wave velocity directly relates to the ease of ripping (Figure 3). A seismic refraction survey will indicate depth to rock, slope of the bedrock surface, how badly weathered the bedrock surface is, and the irregularity of the surface. It is also important in this terrain to located buried valleys and fault zones prior to construction. These features can be determined with a combination of seismic refraction and gravity surveys. Geophysical surveys will greatly reduce the number of borings necessary to characterize the bedrock surface and will give important information for construction design.

Igneous and metamorphic terrain

Igneous and metamorphic rocks tend to be harder then sedimentary rock and more frequently need to be blasted for a road cut. Seismic refraction will, however, indicate locations of rock that can be ripped with heavy equipment. Other subsurface features of interest are similar to those we covered under insoluble sedimentary rocks. The same geophysical techniques apply.

Glacial terrain

In areas covered by glacial deposits, the topography of bedrock beneath these deposits and their soil characteristics are usually not well known. Most the United States north of the Missouri and Ohio Rivers contain glacial deposits which range from inches to hundreds of feet thick. In these areas depth to water and rock as well as the irregularity of the bedrock surface greatly affect the construction costs. In areas where freezing conditions exist during part of the year it is important to locate highway subgrade material that will drain well. Seismic and gravity techniques can be used to explore for gravel deposits. Again geophysics is useful in defining buried valleys, depth to water and rock, and the character of the bedrock surface which results in reducing the amount of borings necessary.

Coastal sediments

The greatest population densities, hence the greatest numbers of highways are near the coast. These areas have some unique problems such as weak subgrade and areas of potential landslides. In coastal areas repairing failures has required much needless expense. Many of the highway failures were caused by inadequate knowledge of subsurface conditions or inadequate money to construct the highway properly. One critical issue is how well the subgrade will drain.

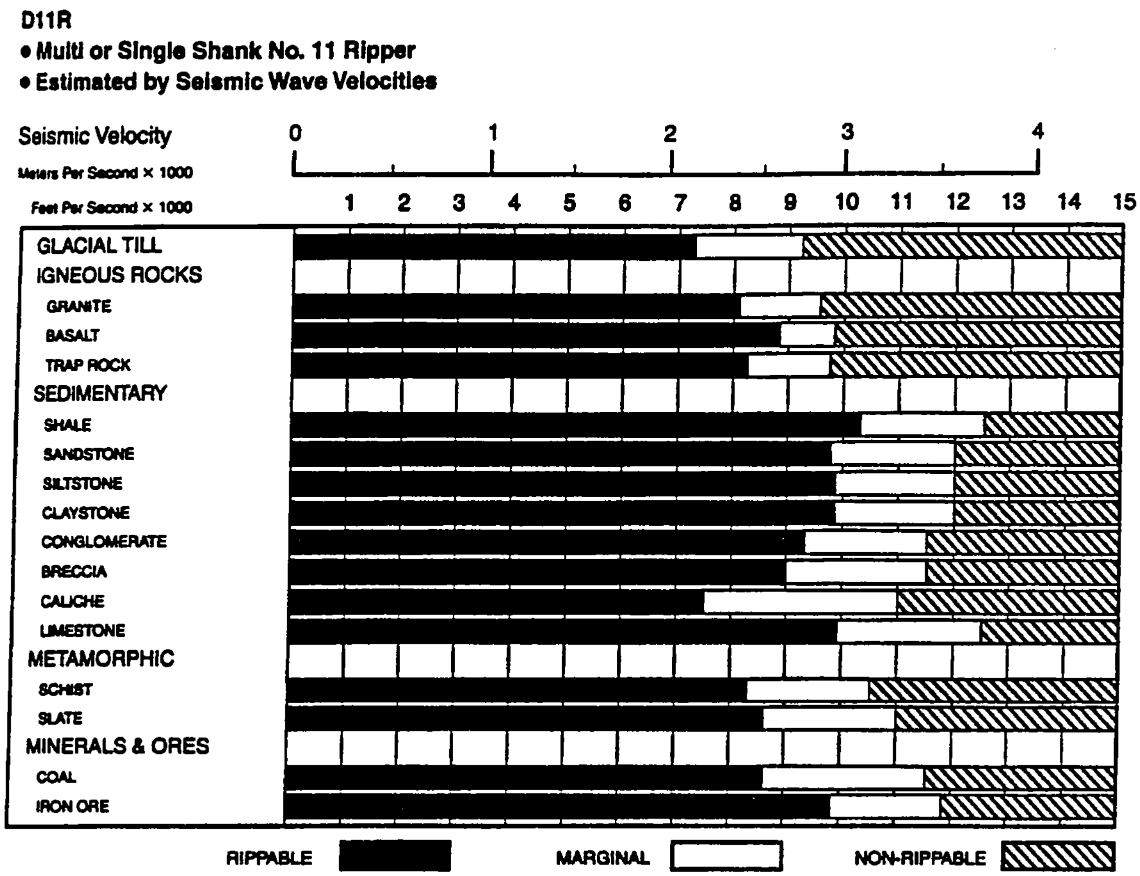

Figure 3. The relation of rippability with a D11R Caterpillar to the seismic wave velocity for various rock types.

Another is the effect construction will have on slope stability. Geophysical techniques can provide information on soil saturation and what affect construction will have on slope stability. Gravity surveys will also locate low density and incompetent subsurface materials and determine their lateral extent.. Test borings are always necessary but the degree of uncertainty and therefore costs is greatly reduced by geophysics.

Conclusion

Case histories of highway construction problems show that any technique that aids the engineer to better understand the subsurface should be employed. Some cases come to mind that may not have occurred if proper preliminary work had been performed. A few examples are: collapse over caves and mine tunnels, activation of landslides, slopes in a highway cut too steep to be stable, using incompetent material, and constructing a highway over peat or other incompetent material. If geology and geophysics are completed before requests for bids, major problems caused by lack of subsurface knowledge can be greatly reduced and contract costs will more closely match real project estimates.

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SHEAR WAVE VELOCITY FIELD TO DETECT ANOMALIES UNDER ASPHALT

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ABSTRACT

Non-invasive mapping of anomalies beneath asphalt at depths from 2 m to as deep as 50 m has been successful using MASW in a variety of near-surface settings. Anomalies that include fracture zones within bedrock, dissolution/potential subsidence features, voids associated with old mine works, and erosional channels etched into the bedrock surface have been effectively identified in the shear wave velocity field calculated by the Multichannel Analysis of Surface Waves (MASW) method. By acquiring many individual multichannel surface wave data gathers on even spacings along a continuous transect, a series of 1-D shear wave profiles obtained by inverting surface wave dispersion curves can be generated that form a 2-D shear wave velocity field beneath the transect. The cell size of the shear wave velocity field depends on the frequency range of the data and source spacing. By contouring the shear wave velocity field, variations representative of anomalous subsurface can easily be interpreted. The method itself focuses on surface wave data with frequencies ranging from 2 to over 60 Hz, which can be directly correlated to depth of investigation and is completely insensitive to cultural noise and surface conditions (e.g., asphalt, gravel, cement, etc.). By incorporating CMP style roll-along acquisition with multichannel acquisition, sufficient redundancy and smoothing exists to confidently interpret anomalies that are evident across several 1-D profiles. Case histories from several uniquely different sites with uniquely different problems provide empirical evidence supporting the utility of this method. Mapping bedrock beneath an asphalt parking lot at depth from 2 to 7 m was successfully accomplished at a site in Olathe, Kansas. Preliminary analysis of this site's hydrologic characteristics, based primarily on borehole data, suggested that fractures and/or an unmapped buried stream channel was influencing fluid movement along the drill-defined bedrock surface. High velocity gradients within the shear wave velocity field were used as diagnostic of the bedrock surface, while localized lateral decreases in the shear wave velocity below the bedrock surface were considered characteristic of fracture zones or erosional channels. Delineating voids resulting from extensive underground mining of lead and zinc in southeastern Kansas was critical to pavement evaluations and determinations of road stability. Void features interpreted on shear wave velocity profiles were consistent with extensive borehole data collected at this site. Subsidence features obscured by development and masked from other geophysical methods by power line noise, mechanical noise, reinforced concrete, and requirements for non-invasive methods were distinguishable on 2-D shear wave velocity field data. Subsidence features interpreted on data acquired through occupied houses in western Florida were correlated with existing drill data and verified by drilling based on interpretations of those data. Pits and trenches were located beneath asphalt surfacing at an old refinery site in eastern Illinois through coincident analysis of phase and amplitude distortions on surface wave data with the 2-D shear wave velocity field. Pipes placed 3 to 5 ft deep in trenches, infilled with native soils, and then covered with asphalt produced a distinctive signature on surface wave data. Advantages of using the shear wave velocity field, calculated from surface waves to detect, delineate, and/or map anomalous subsurface materials include the insensitivity of MASW to velocity inversions and cultural noise, ease of generating and propagating surface wave energy in comparison to body wave energy, and its sensitivity to changes in velocity.

INTRODUCTION

Surface waves traditionally have been viewed as noise on multichannel seismic data designed to image environmental, engineering, and groundwater targets (Steeples and Miller, 1990). A recent development incorporating concepts from spectral analysis of surface waves (SASW) developed for civil engineering applications (Nazarian et al., 1983) with multitrace seismic acquisition methods commonly used for petroleum applications (Glover, 1959) shows great potential for detecting, and in some cases delineating, anomalous subsurface materials. Extending the common use of surface wave analysis techniques from estimating 1-D shear wave velocities to detection and/or imaging required a multichannel approach to data acquisition and processing. Integrating the multichannel analysis of surface waves (MASW) method with a common mid-point (CMP)-style data acquisition permits the generation of a laterally continuous 2-D shear wave velocity field cross-section (Park et al., 1996; Xia et al., 1997; Xia et al., 1998; Park et al., 1999; Xia et al., 1999). The MASW method as used here requires minimal processing and is relatively insensitive to cultural interference. Mating MASW with the redundant sampling approach used in CMP data acquisition provides a non-invasive method of delineating horizontal and vertical variations in near-surface material properties.

Continuous acquisition of multichannel surface wave data along linear transects has recently shown great promise in detecting shallow voids and tunnels (Park et al., 1998), mapping the bedrock surface (Xia et al., 1998; Miller and Xia, 1999a), locating remnants of underground mines (Park et al., 1999), and delineating fracture systems (Park et al., 1997; Miller and Xia, 1999b). Extending this technology from sporadic sampling to continuous imaging required the incorporation of MASW with concepts from the CDP method (Mayne, 1962). Integrating these two methodologies resulted in the generation of a laterally continuous 2-D cross-section of the shear wave velocity field. Cross-sections generated in this fashion contain specific information about the horizontal and vertical continuity and physical properties of materials as shallow as a few inches to over 300 ft BGS in some settings.

Seismic reflection surveys are generally designed to image structural and stratigraphic features with a high degree of resolution and accuracy. On such surveys, surface waves are considered noise. For our application, however, it is possible to exploit the sensitivity of the surface wave to changes in material velocities that make up the half-space it travels through. Surface wave propagation depends on frequency (depth of penetration), phase velocity (compressional and shear), and density. Each of these properties will affect the surface wave dispersion curve (phase velocity vs. frequency) in a predictable fashion. Since shear wave velocity has the greatest impact on the properties of a surface wave, we can invert the dispersion curve in such a way as to obtain the shear wave velocity as a function of depth (Xia et al., 1999). Disturbances in the shear wave velocity field will show up as anomalies in the otherwise uniform contours of the shear wave velocity field for a layered earth.

Advantages

Several key characteristics of surface waves and surface wave imaging make application of this technique possible in areas and at sites where other geophysical tools have failed or provided inadequate results. First and probably foremost is the ease with which surface waves can be generated. The relative high amplitude nature of surface waves (in comparison to body waves) makes possible their application in areas with elevated levels of mechanic/acoustic noise. A layer over half space is all that is necessary to propagate surface waves. It is one of the few acoustic methods not requiring velocity to increase with depth and/or a contrast (i.e., velocity, density, or combination [acoustic impedance]). Conductivity of soils, electrical noise, conductive structures, and buried utilities all represent significant problems or at least important considerations for electrical or EM methods. These have little or no impact on the generation or propagation and generally have no influence on the processing or interpretation of surface wave data. This flexibility in acquisition and insensitivity to environmental noise allows successful use of shear wave velocity profiling in areas where other geophysical methods might be limited.

Surface waves, when used to image the earth, provide a rapid and relatively straightforward method of examining the shallow subsurface. Unfortunately, interpretations of the two-dimensional shear wave velocity field derived from the inversion of the surface wave dispersion curve are much lower resolution than seismic reflection sections. However, the shear wave velocity field derived in this fashion is quite sensitive to abrupt changes in shear wave velocity. In this setting, it is reasonable to expect voids, caverns, or collapse features to be associated with an abrupt change in shear wave velocity.

Applications

Areas with pits, trenches, or underground utilities are likely targets for this type of imaging. Decreases in the shear wave velocity related to reduced soil compaction or localized increases in shear wave velocity associated with compacted caps overlying buried trenches or pits dramatically affect the dispersive character of surface wave energy. Key to exploiting surface waves as a site characterization tool is their sensitivity to shear wave velocity, compressional wave velocity, density, and layering in the half space.

Subsidence-prone areas are likely targets for this type of imaging. Decreases in the shear wave velocity related to decreases in compaction or localized increases in shear wave velocity associated with the tension dome surrounding subsurface cavities are key indicators of either subsidence activity or areas with a strong potential for roof collapse. In situations where gradual subsidence is or has been active, a dramatic drop in shear wave velocity will be characteristic of earth materials that have begun to collapse into voids formed at depth. This low velocity zone generally produces a characteristic signature in the shear wave velocity field. Since the shear wave velocity of earth materials changes when the strain on those materials becomes "large," load-bearing roof rock above mines or dissolution voids may experience elevated shear wave velocities due to loading between pillars, or, in the case of voids, loading between supporting sidewalls. Key to exploiting surface waves as a site characterization tool is their sensitivity to shear wave velocity, compressional wave velocity, density, and layering in the half space.

DATA ACQUISITION

Each site will have specific characteristics effecting data properties. Optimizing parameters and equipment is critical to maximizing the accuracy, analysis format, and potential of the resultant processed sections. Data acquired for surface wave analysis using the MASW technique are generally broadband (i.e., 4 Hz to 64 Hz; four octaves), with offset designs based on target dimensions and depths. Standard CMP roll-along techniques are used in conjunction with 30- to 60-channel recording systems. Shot and receiver spacings as well as near and far source offsets depend on number of recording channels and maximum and minimum depth of interest. Ground cover (asphalt, cement, gravel, grass, etc) has no significant influence on the accuracy of the recorded surface wave energy. Generation of surface waves is quite easily accomplished with weight drop style sources, with the particular specifications of the source only limited by the dominant frequency band of interest. For deeper penetration a large and heavy source is optimum. Receivers need to be low frequency (< 8Hz) and broadband. With cost consideration, the optimum geophone has a natural frequency of around 4.5 Hz and can be outfitted with either flat baseplates or short spikes. Recording geometries and frequency ranges of data examples presented here provided optimum data characteristics for examining earth materials in the depth range from about 3 to over 150 ft below ground surface (BGS).

Recording acoustic data on asphalt or cement surfaces generally comes with coupling problems, limited amounts of vertically propagating body waves, and complex high-frequency trapped and guided waves. Many studies have shown that receiver-ground coupling is critical for high-resolution body wave surveys (Hewitt, 1980). Maximizing frequency response and recorded body waves normally requires longer spikes, well seated into competent earth. Coupling experiments at various sites have suggested receivers only require simple ground contact to record broad-spectrum surface wave energy. Little or no improvement is evident in response

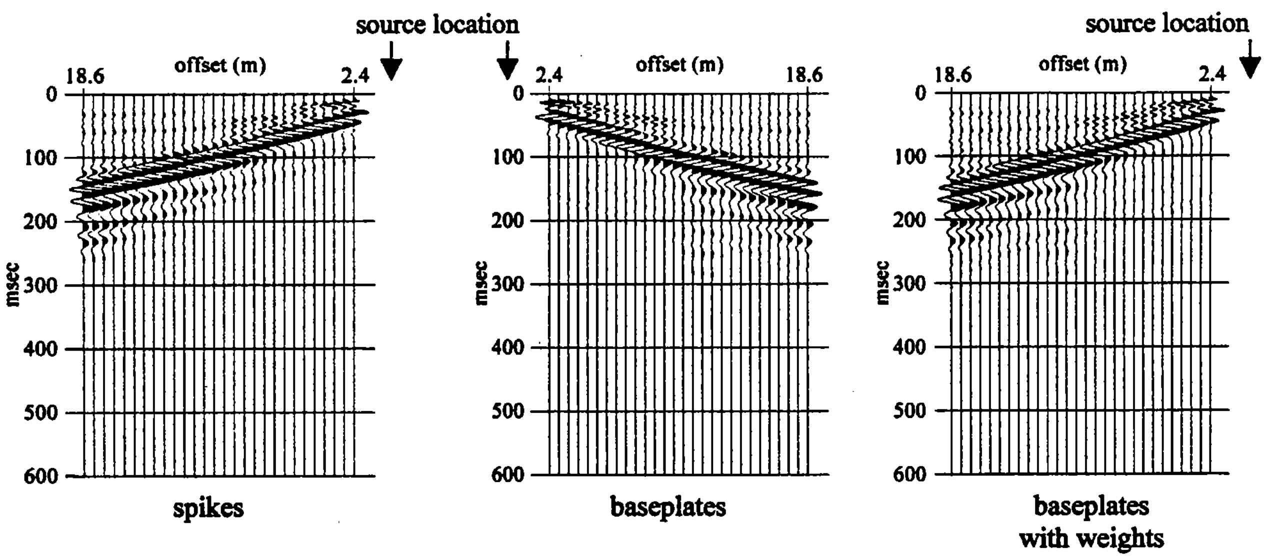

Figure 1. Comparison of geophone deployment methods.

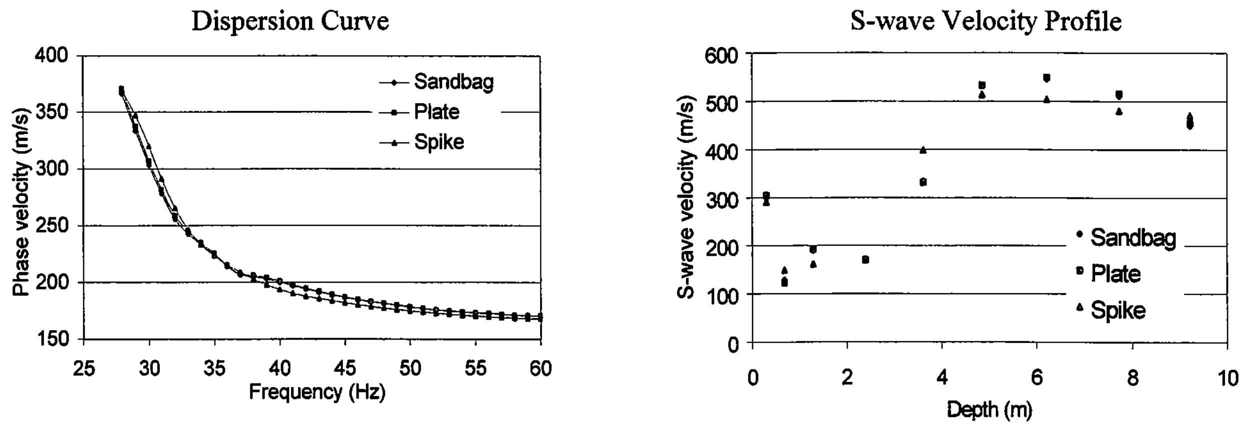

Figure 2. Comparison of geophone coupling.

(frequency vs. amplitude) when geophones are "planted" using spikes, placed on the ground using plates, or held to the ground with sandbags (Figure 1). This observation continues to fuel research into the use of land streamers, continuous recording techniques, and real-time data processing.

DATA PROCESSING

Each multi-channel shot gather was recorded with all live receivers within the optimum offset window for surface wave sampling of subsurface materials within the target window. Multichannel records were analyzed with *SurfSeis* (a proprietary software package of the Kansas Geological Survey), which facilitates use of MASW under continuous profiling configurations. During processing, each shot gather will generate one dispersion curve (Figure 2). Care must be taken to insure the spectral properties of the t-x data (shot gathers) are consistent with the maximum and minimum f-v_c values (v_c is phase velocities of surface waves) contained in the dispersion curve. Each dispersion curve is individually inverted into an x-v_s trace. Gathering all x-v_s traces into shot station sequential order forms a 2-D grid of the shear wave velocity field. The shear wave velocity field generated in this fashion does "smear," to a limited extent, the velocity value for a specific point in space. This distortion requires a good understanding of the resolution and accuracy of the specific data set analyzed before making any interpretation.

CASE STUDIES

Olathe, Kansas

Continuous 2-D profiles of the subsurface provide vital information in areas like this where, due to the sporadic nature and non-uniformity in drill hole spacing, drill data alone does not allow subtle and in many cases extremely significant bedrock features to be extended, or in some cases even detected. The Olathe case study delineated a buried channel and provided a map of the bedrock surface in an area near a building used to manufacture electronic components where contaminant loss was possible (Figure 3). Industrial fluids essential to the manufacturing process were routinely used and stored in and around this building. If these fluids were to leak from containment vessels or plumbing, a detailed transport and fate model would be imperative to rapid isolation and extraction of these hazardous fluids. This is a scenario not unlike thousands currently under investigation

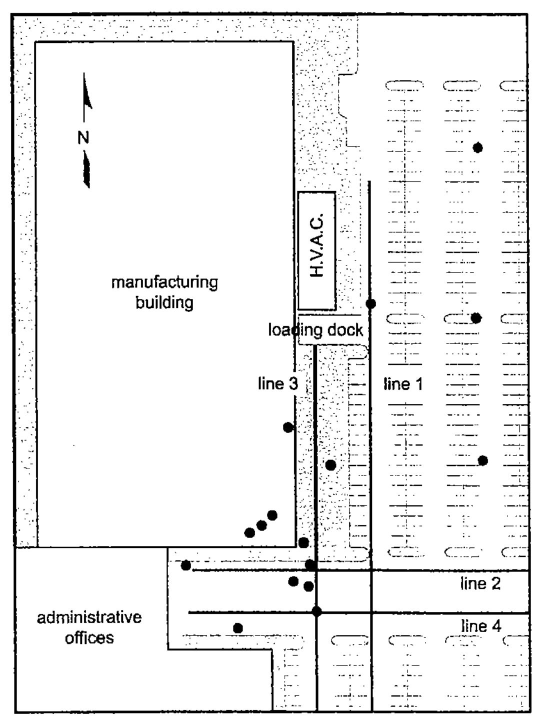

Figure 3. Site map at Olathe, Kansas.

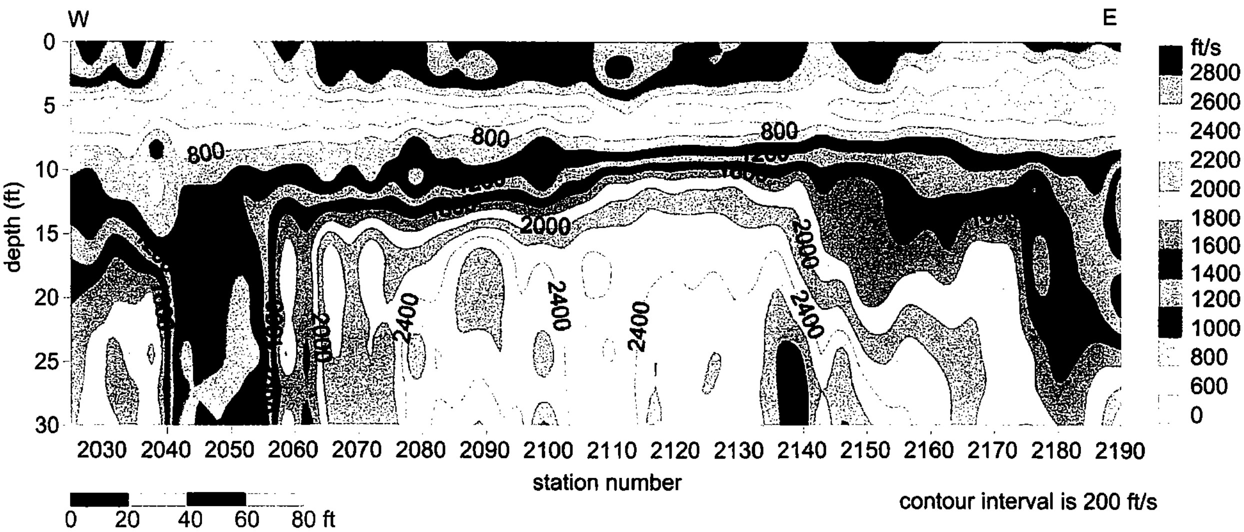

Figure 4. S-wave velocity contours along line 2 at the Olathe, Kansas, site.

around the country. For the study described here, two sets of parallel intersecting profile lines were located as near the building as practical and in close proximity to existing borings. Existing borings and monitor wells were drilled and completed to define bedrock and/or monitor groundwater. Acquisition parameters and geometry for MASW were selected to optimize the imaging of near-surface unconsolidated materials above bedrock, the bedrock surface, and several feet into bedrock. Depths of interest ranged from about 2 ft to 35 ft below the ground surface. Improving the bedrock surface map and delineating any potential contaminant pathways on or into bedrock were the primary objectives of this survey.

There are two features on line 2 with the potential to affect fluid movement along the surface of bedrock (Figure 4). An extreme drop in shear wave velocity beneath station 2050 is either a paleochannel infilled with weathered bedrock material or a fracture/fault zone. On the western flank of this abrupt low-velocity zone is a very localized velocity low beneath station 2040. This feature is pronounced and topographically the lowest point along this line on the bedrock surface. Immediately beneath station 2050 a drop in the shear wave velocity is evident from the ground surface to about 5 ft or so. This shallow low-velocity zone correlates with the known location of a sewer line buried along the eastern side of the building. A borehole was drilled to confirm that the lower velocity channel in bedrock between stations 4075 and 4088 was real and not an artifact of the sewer trench and methodology. The second noteworthy feature on this line is the broad channel feature on the east end of the line, defined by the gradual drop in shear wave velocity beyond station 2140. This bedrock channel could be the result of cut-and-fill with the infill material having properties distinctly different from the low-velocity unconsolidated sediments above bedrock.

Data resolution is an issue that must be addressed when using this technique. Surface wave imaging techniques involve the inversion of a wave that has sampled an area nearly as wide as deep. As well, the sampling depth is generally considered to be one-half the wavelength. Assuming the wave is limited to the 2-D plane, the velocity value assigned to a single sample point in the subsurface has been calculated using a wave that has sampled an area several times the square of the sample point depth. Therefore, structures observed on shear wave cross-sections are likely smoothed, subdued, and/or a sculpted version of what really exists in the subsurface.

Resolution of the drill-defined bedrock surface map improves significantly after incorporation of shear wave velocity data (Figure 5). Depth to bedrock contours based on drill data alone grossly defined the configuration of bedrock in proximity to the boreholes. As well, the bedrock contour map from shear wave data alone lacks the necessary off-line control to constrain depth contours of the bedrock surface. Incorporating the drill data and shear wave data greatly improved the detail and sitewide resolution of the depth-to-bedrock map as compared to either data set individually. Adding a few more seismic lines could noticeably improve the 3-D aspects of the bedrock contours.

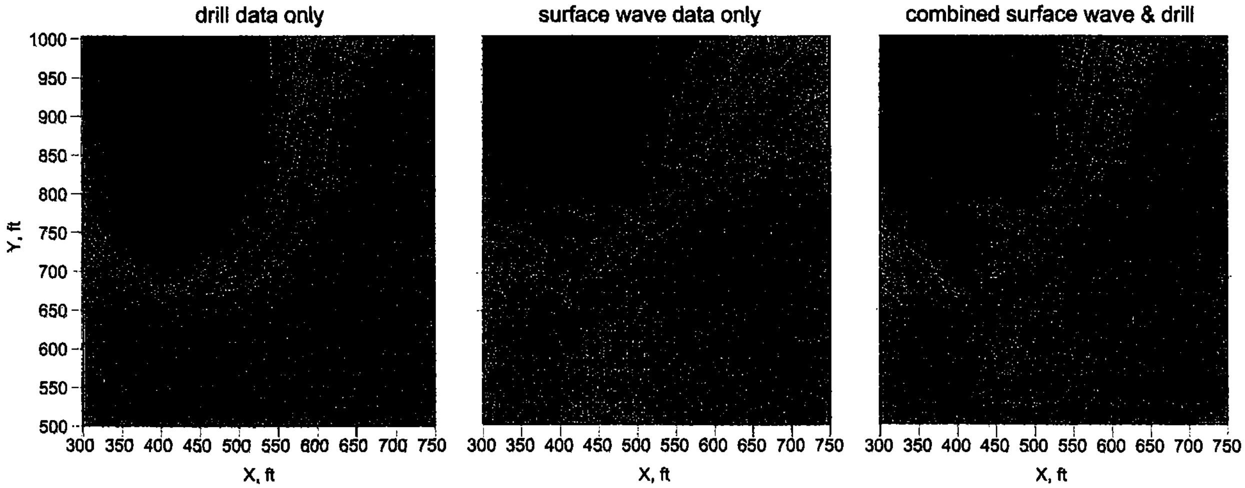

Figure 5. Depth-to-bedrock contour map based on (a) drilling alone, (b) seismic data alone, and (c) a combination of both drilling and seismic data.

Baxter Springs, Kansas

Historical lead/zinc mining activities have left surface scars and underground hazards across a portion of southeastern Kansas, southwestern Missouri, and northeastern Oklahoma, known as the Tri-State Lead/Zinc Mining District. Fractures and voids in otherwise competent near-surface rock layers can pose a stability risk to any overlying surface structure. Confident detection and 3-D mapping of voids or open fractures prior to surface expression permits the evaluation and possible reduction in the risk such features pose to people and property. Surface growth associated with a sinkhole located within 100 ft of State Line Road and a gas metering station south of Baxter Springs, Kansas, raised concerns for public safety and prompted an extensive drilling and geophysical investigation. The mine workings of interest ranged in depth from 100 to over 150 ft below ground surface and are in an area where Mississippian Limestone acts as both host and roof rock. Overlying the limestone is a soil and clay layer varying in thickness from 10 to 20 ft. Subsidence in this area has a historical precedent dating back prior to the mid-twentieth century.

Spectral properties of shot gathers changed abruptly when the surface material beneath the source went from a grassy soil to asphalt. This change in bandwidth occurred as a result of the source's inability to produce and propagate the lower frequency components of the signal. Producing low frequency surface waves requires a large energy source to deform a large volume of earth in a very plastic fashion. The asphalt represents a relatively rigid surface which does not easily deform over a large area. Inversion of dispersion curves when the source was on the asphalt produced a shear wave velocity profile with a penetration depth around 50 ft with a reliable velocity function down to about 40 ft.

Clearly, several significant anomalies can be interpreted along the profile (Figure 6). Of particular interest: the anomaly beneath station 2040 as well as the low velocity pattern observed between station 2065 and 2085. The low velocity closure beneath station 2075 has a well-defined high velocity halo. A final feature, physically the smallest of the anomalies noted but probably the feature of most concern, is the low velocity closure at about 50 ft BGS capped by an extremely high velocity geometrically erratic zone beneath station 2090.

The feature located beneath station 2040 is the largest and probably the best target for a confirmation drill hole targeting mine workings. This anomaly has all the physical characteristics of a mine drift. Of significant interest is the lack of any kind of a high velocity halo above this feature. It is very unlikely this feature is building strain in a fashion that would be considered a precursor to roof failure and collapse.

The low velocity closure directly beneath the high velocity closure at station 2090 extends from 20 ft to 50 ft BGS and is of particular interest. In this case the high velocity halo over the low velocity closure does not possess a geometry consistent with what has previously been observed in subsidence-prone areas. Considering the cyclic non-dipping geology in this area, it would not be unreasonable to expect strain to build along intact layers which act as the roof for lenticular voids. If roof strain can be directly correlated to localized, high

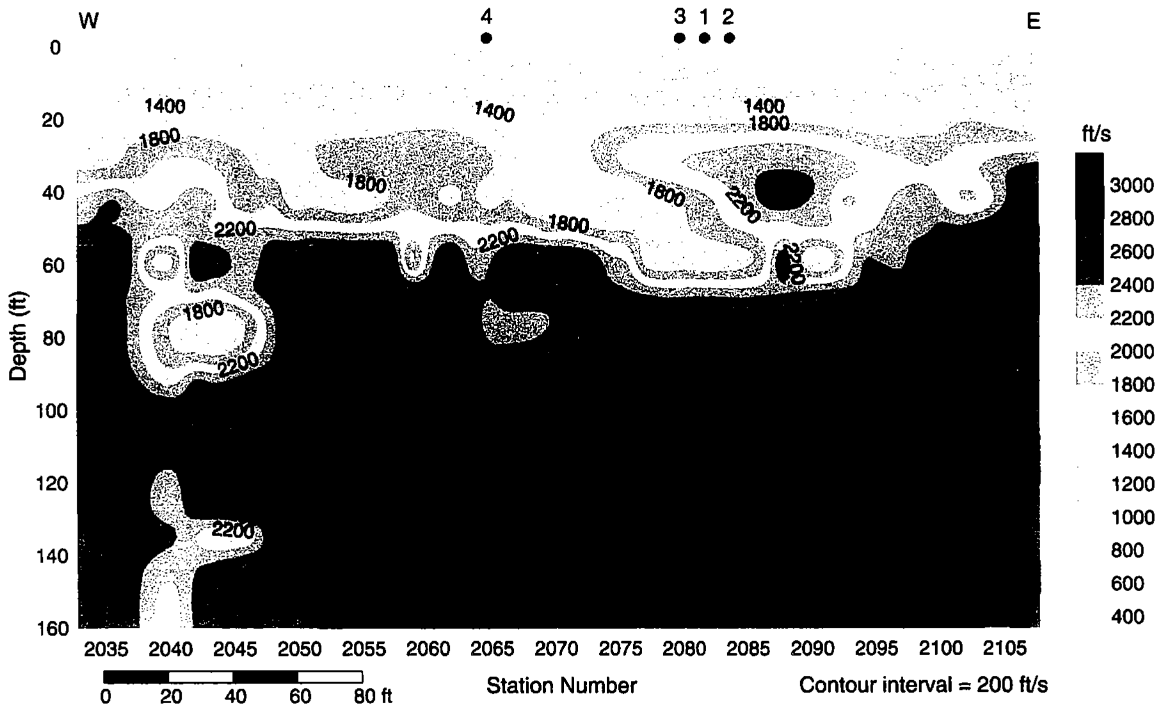

Figure 6. S-wave velocity field of line 2, Baxter Springs, Kansas.

velocity anomalies, sediments above 20 ft at station 2090 appear to have little risk of imminent collapse. With the bedrock surface defined by flat and uniform velocity contours, it is doubtful the bedrock surface was under stress below station 2090 at the time this profile was acquired. A final troubling correlation is proximity of this high over low velocity feature at 2090 in comparison to the subsurface extension of the surface elongation of the sinkhole.

The goal of this study was to detect void features and appraise their potential of developing a surface expression (sinkhole). Of primary concern was identifying low velocity anomalies, in otherwise laterally continuous materials, that can be correlated to the current sinkhole. Of particular interest was the sinkhole's relationship to old mine works or fractures inferred from mine maps and detected by drilling. The surface wave imaging technique clearly possesses excellent potential as a monitoring tool in this setting. It was not possible to correlate velocity anomalies on shear-wave profiles to drill-encountered voids with sufficient confidence to map individual voids and their extent. An additionally complicating factor at this site is discriminating the geometrically complex geology of this unique setting from void signatures.

Considering the complex geometry of the fracture and fault systems in this area it is not surprising this method does not possess the resolution necessary to discriminate each void encountered during drilling. The method does possess vast potential as a long-term monitoring tool in this geologic setting. Geophysical methods provide a much better understanding of how things are changing as opposed to providing unique correlations to geologic units or features. In that regard, by using the method in a time lapse format, changes associated with increased strain or vertically migrating roof failure should be easily detected. A great deal of confidence can be placed in determinations of subsidence potential if changes in the shear wave velocity field can be contrasted and compared across time.

Lawrenceville, Illinois

This study focused on acquiring data from several areas with uniquely different targets and near-surface conditions. These data are intended to provide a sampling of the shear wave velocity field (directly related to stiffness), to allow anomalous zones thought to be pits, trenches, or underground utilities to be identified as well as to evaluate the potential of this technique to map lithologic boundaries. It was our goal to identify characteristics unique to the various targets suspected to be present in the subsurface at each site in such a way

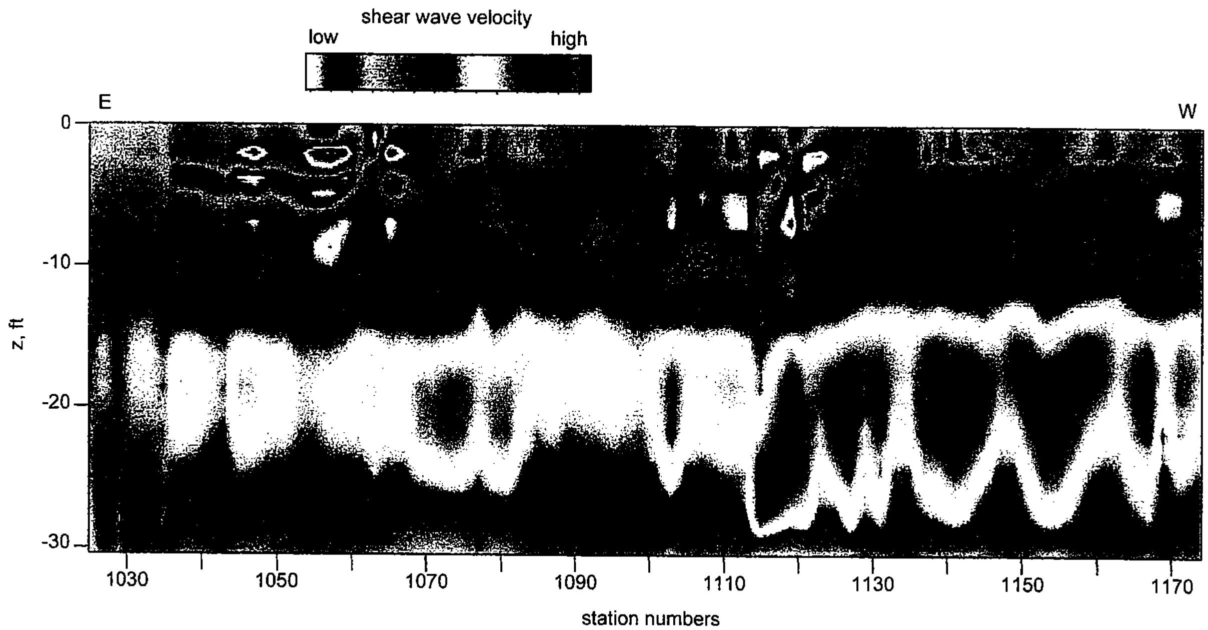

Figure 7. S-wave velocity contour color image from Lawrenceville, Illinois, site.

as to allow identification of similar anomalous features at other sites where their location is not nearly as well known. Lines were deployed after considering the optimum sampling of subsiding materials, depth of interest, and type of target. Targets included: a series of transfer pipes buried 3 to 5 ft beneath an asphalt road; continuity of alluvial layers from 10 to 100 ft BGS; mapping the base of a clay dike used as a retaining structure for tar sludge; surface remediated burn/burial pits; and top of bedrock approximately 150 ft deep at an old land farm site. The primary objective of this study was to evaluate the effectiveness of the method to locate, identify, and delineate unique subsurface targets at each of these five uniquely different sites with sufficient resolution to allow drill confirmation.

Data from the burn/burial pits provide an example of the effectiveness of the technique to delineate shallow anomalous areas in otherwise relatively uniform materials. These pits were located within several hundred feet of the Wabash River and over a 150 ft thick section of alluvial sediments. Usable bandwidths ranged from 10 Hz to 70 Hz, equating to a maximum and minimum depth of penetration from around 25 ft to 2 ft, respectively. Based on error analysis and previous studies designed to evaluate measurement accuracy (Xia et al., 2000), anomalies with closure on two or more contour lines (i.e., approx. 15%) are related to measurable changes in subsurface properties. Localized closures defined by a single contour may be related to geology, but are below the required level of certainty for confident identification on this study. Velocity trends along this line are laterally uniform, and in general, consistent with a very homogeneous subsurface across the profile. Of particular interest is the presence of localized high and low velocity closures near the eastern end of the profile (Figure 7).

Anomalous zones, such as those observed at the eastern end of the velocity-contoured section, are conceptually consistent with materials in a compacted (capped) landfill or an infilled burial pit (Figure 7). Based on the gradient and absolute value of the anomalies on the east end of the profile, it is reasonable to suggest natural sediments beneath stations 1035 to 1060 at 2 to 8 ft BGS have been altered or replaced. It is not possible to speculate on the nature of the disturbed area, or even if it is natural (river channel) or manmade (landfill), with confidence. It is, however, quite likely considering the character, gradient, and location of this feature, that it is an acoustic expression of an old trench. Looking deeper into the section, a high-velocity zone can be interpreted beginning at a depth of around 15 ft and extending below the imageable depth of this survey. As stated previously, the maximum depth interpretations can be made with any confidence is around 20 ft. The apparent velocity inversion below about 25 ft results from insufficient low frequency energy penetration to that depth. The high-velocity layer is likely a relatively tight clay or silty-clay, or possibly the boundary between alluvial sediments and glacial materials.

Tampa, Florida

Non-uniform foundation settling is responsible for millions of dollars in structural damage in the Tampa, Florida area alone. MASW data were used to study variations in and correlate to the structural properties of the shallow subsurface materials as they relate to shear wave velocity. Characterizing the near-surface materials supporting these structures is critical to determining the mechanism responsible for the physical damage. Materials present above "bedrock" are classified by most borehole lithologic descriptions as combinations of silts, sands, and clay in differing concentrations and degrees of cementation. Most boreholes in and around the areas investigated reach refusal in stiff clays or silts. Based on correlation with shear wave velocity profiles, it is very likely the silt or stiff clay is at or very near the top of bedrock (likely limestone). These data provided a sampling of the shear wave velocity field (directly related to stiffness), which in turn allows laterally anomalous zones, possibly indicative of subsidence activity, to be identified and drill investigated.

Several structures were studied with varying degrees of structural damage to foundations and walls. Seismic lines were deployed around and through these structures after surficial investigations of apparent subsidence areas, determinations of maximum line lengths (considering the density of residential structures), and consideration of line-to-line correlations. Data presented here was from a profile across the front yard over the driveway and through the garden of a residence (Figure 8). Based on a few scattered borings, depth to bedrock was expected to be around 40 ft. Damage to the foundation and walls of the residence was evidenced by cracks in the walls and offset in the cement slab floor. The overall objective of this study across the Tampa, Florida, area was to determine if it was possible to improve the general understanding of the subsidence mechanism at each of these uniquely different sites with sufficient resolution to allow drill confirmation and then establish acoustic characteristics and criteria for detection.

Oscillations observed in the shear wave velocity field at depths greater than 50 ft are consistent with changes observed in other areas around the southeast United States with active dissolution of limestone (Miller and Xia, 1999b). The frequency of the highs relative to lows on this south-to-north line is about 30 ft per wavelength (Figure 8). Considering a residence immediately south of this profile has confirmed dissolution subsidence damage, it is reasonable to suggest the low velocity zone at the extreme southern end of the line is related to this confirmed subsidence. Based on that correlation, the features at stations 115 and 140 are strong candidates for dissolution anomalies. Little can be deduced from changes in the shear wave velocity at depths

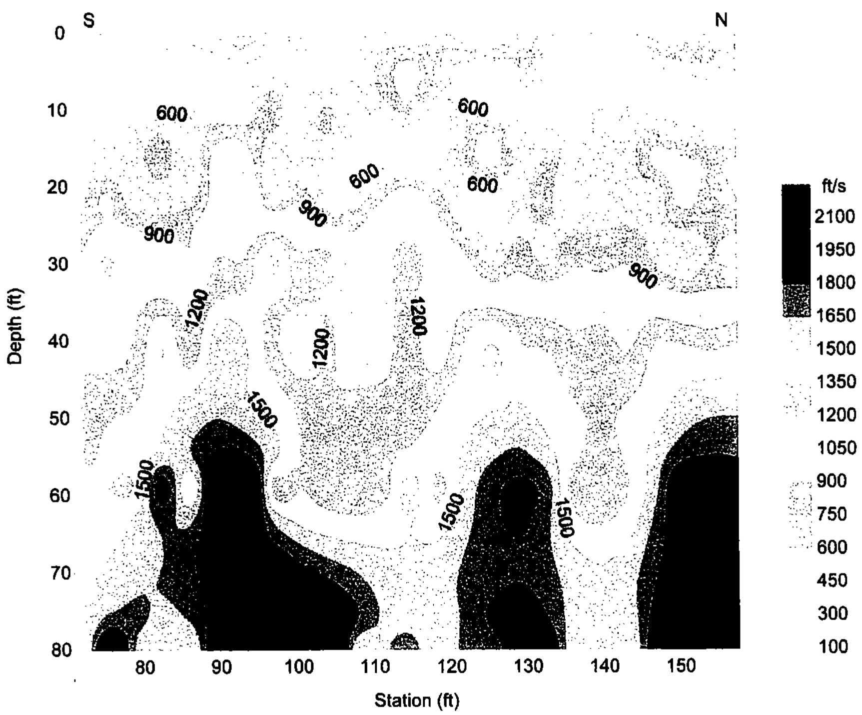

Figure 8. S-wave velocity contours at a site near Tampa, Florida.

less than about 25 ft. It is not possible with this data alone to infer a migration path between the two "bedrock" dissolution features at stations 115 and 140 with surface subsidence and the resulting documented damage. Without a doubt, these 50% drops in the shear wave velocity field across lateral distances on the order of 15 ft are related to changes in the structural integrity of the rock units present below 25 ft.

A boring located at approximately station 140 encountered lithologic units and measured blow counts that are consistent with the shear wave data through the depth range of the boring log (30 ft). Shallow sediment velocities along line 2 provide little in the way of characteristics consistent with chimneys or conduits for sub-siding materials working their way downward from the surface or very shallow near surface. A couple higher velocity zones present north of station 120 and at depths less than 3 ft are likely related to the concrete slabs that make up the driveway and sidewalk.

CONCLUSIONS

Extending surface wave analysis techniques from use predominantly by the engineering community for estimation of the shear wave velocity as a function of depth, to a tool for detecting shallow subsurface anomalies has met with consistent success. High velocity gradients within the shear wave velocity field are generally consistent with drill-defined bedrock and can be considered diagnostic of the bedrock surface. Localized anomalies in the shear wave velocity field manifested as low- and/or high-velocity closures have been correlated to voids, caverns, rubble zones, and backfilled pits/trenches in a variety of near-surface settings. Localized lateral decreases in the shear wave velocity below the bedrock surface were classified as fracture zones or erosional channels. Mapping layer geometry in a gross sense is possible, but care must be taken to insure interpretations do not surpass the resolution limits of the method.

Calculating the shear wave velocity field from surface wave arrivals can be accomplished with a high degree of accuracy regardless of cultural noise. The insensitivity of MASW to cultural obstacles and noise was demonstrated at several distinctly different sites with uniquely different geologies. Depth sections produced using the shear wave velocity and drill data possesses significantly higher resolution than maps produced using drilling or shear wave velocity data individually. In many cases, less than 1 ft of difference can be observed in the depth-to-bedrock interpreted from surface wave data compared to the depths determined through drilling.

Improved resolution on the surface of the bedrock provides insight into the texture of bedrock and permits identification and appraisal of short wavelength variations in the bedrock surface. The goals and objectives of each of the previously described surveys were met. Advantages of mapping the bedrock surface with the shear wave velocity field calculated from surface waves include the insensitivity of MASW to velocity inversions, ease of generating and propagating surface wave energy in comparison to body wave energy, and sensitivity to lateral changes in velocity.

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Characterization of Karst Systems for a Highway Widening Study in Warren County, Virginia

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ABSTRACT

Multiple surface and subsurface reconnaissance and analytical techniques were performed to characterize karst geologic conditions as part of a comprehensive geotechnical engineering design study. The planned project consists of the widening of 4.6 kilometers of Virginia Route 340 south of Front Royal, Virginia. The existing two-lane highway traverses rugged terrain between the South Fork of the Shenandoah River and the western flank of the Blue Ridge Mountains. It is also located near several cave systems of historic and commercial interest.

The geotechnical field investigation for the highway alignment widening included mapping of surface geology, sinkholes and cave entrances, rock mass structure mapping and analysis, resistivity imaging surveys in two and three dimensions, and conventional soil borings and rock coring. In addition to the highway alignment field investigation, mapping and numerical modeling of stalactites and rare anthodite formations in the commercially operated Skyline Caverns were performed to assess the potential impact of construction blast vibrations on these features. The vibrations were assumed to be generated by blasting planned within rock cuts located less than 20 meters from the nearest commercially maintained room in the caverns.

The integration of surface mapping data, geophysical imaging, and boring data was essential in optimizing slope designs, estimating the extent of rock excavations, and estimating the extent of suspected subsurface cavities and sinkholes beneath the planned road alignment and cut slopes. Design guidelines were also developed from the results of the numercial models of delicate cave structures for controlling construction vibration impacts to the Skyline Caverns during construction of the highway widening.

INTRODUCTION

U.S. Route 340 (Rt. 340) in western Virginia meanders for about 250 scenic kilometers near the base of the western slope of the Blue Ridge Mountains from Lexington, Virginia north to the West Virginia state line. Significant stretches of the highway are underlain by Paleozoic age limestone and dolostone formations prone to solution weathering and karstification. The region is home to some of the largest and most significant, commercially developed cavern systems in the eastern United States, and attracts cave enthusiasts worldwide. Many of these attractions are located along Rt. 340, including the Luray Caverns and the Skyline Caverns. Unfortunately, the karst geologic conditions that make this region unique also contribute to engineering challenges in designing and maintaining highways.

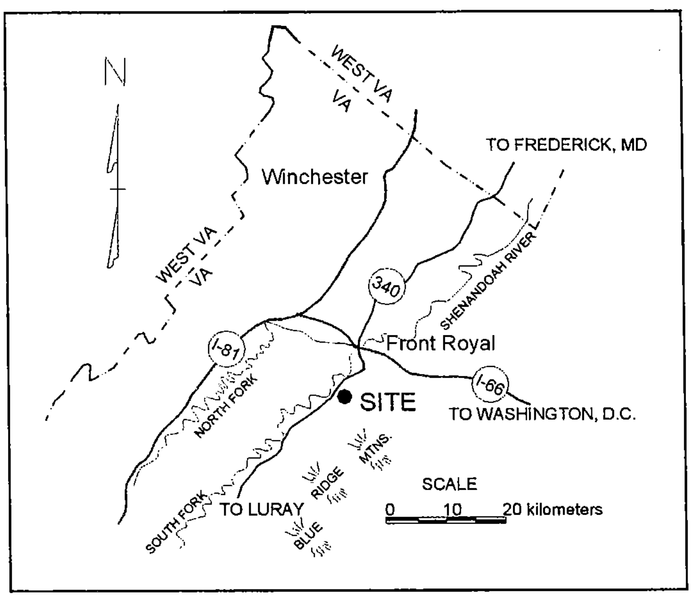

Figure 1. Rt. 340 Project Location Map.

A geotechnical engineering study was performed by Schnabel Engineering Associates (Schnabel) for the Virginia Department of

Transportation (VDOT) and designers Hayes, Seay, Mattern & Mattern, Inc. (HSMM) along a 4.6 kilometer stretch of Rt. 340 just south of the town of Front Royal in Warren County. The purpose of the study was to characterize subsurface conditions within the karst geology and provide geotechnical recommendations for planned widening of the highway from two lanes to a four-lane divided highway. The location of the study area is shown in Figure 1.

The existing highway within the study area is located in extremely rugged terrain, situated at the base of the western flank of the Blue Ridge Mountains and above the eastern bank of the South Fork of the Shenandoah River (South Fork). The highway at this location is characterized by a narrow roadway corridor with little or no shoulder, short sighting distances, steep rock cuts, and an ever increasing daily traffic volume. Numerous cave openings and sinkholes associated with the solution weathering of underlying carbonate rock formations are present adjacent to the existing highway. The commercially operated Skyline Caverns, which averages about 180,000 visitors annually, is located adjacent to the highway at the northern end of the study area. The sensitive nature of the natural karst systems from a ground water protection, recreational, and engineering standpoint demanded a high level of detail and sophistication in the subsurface investigation for the design of the highway widening.

GEOLOGIC SETTING

The project site is located in the easternmost portion of the Valley and Ridge physiographic province of Virginia, at the base of the western flank of the Blue Ridge Mountains. The Front Royal Fault and Happy Creek Fault thrust complex, located within the hill slopes east of the project study area, mark the eastern boundary of the Valley and Ridge province with the Blue Ridge province. Due to extensive folding and faulting in the region, the underlying rocks have been deformed into a series of anticlines and synclines, many of which are overturned. Fracturing associated with the extensive folding and faulting has provided the mechanism by which surface water runoff from acidic igneous and metamorphic rocks of the Blue Ridge uplands can infiltrate through the underlying, soluble carbonate rocks as groundwater. This has resulted in a complex and ecologically sensitive karst geologic terrain that includes locally solutioned limestone bedrock, sinking streams, caverns, and sinkholes. Existing and potential karst features are of primary concern for the proposed realignment and widening of the roadway due to their potential to adversely impact construction activities and future performance of the pavements and associated structures.

Figure 2. Existing Roadcut in Limestone

Most of the study site is underlain by Ordovician age carbonate rocks of the Rockdale Run Formation. The Rockdale Run Formation, which outcrops prominently within ridges and existing rock cuts in the area, consists primarily of gray, variably fractured, thick-bedded limestone and dolomite. Numerous caves, sinkholes and other karst related features have formed across most of the project area as a result of the dissolutioning of the underlying carbonate rocks over time. A thick, deeply weathered fault breccia deposit, associated with past faulting along the Front Royal Fault and possibly the Happy Creek Fault complex, was identified in one of the southernmost realignment areas. Very deep residual soils (up to at least 23 meters deep) with occasional boulders were encountered in the test borings performed in areas underlain by the breccia deposit.

An extensive and complicated underground drainage network has formed in the study area and previous hydrogeologic studies indicate that groundwater flow within the area is primarily toward the northwest, toward the South Fork, through interconnected natural cave systems and bedrock fractures. This karst aquifer system is recharged by natural sinking streams that drain the Blue Ridge uplands to the east, and by surface water runoff that enters the karst aquifer network through sinkholes and bedrock fractures. Residences and businesses along the Rt. 340 corridor within the project study area derive most of their water supplies from domestic wells that have been drilled into the underlying carbonate bedrock and as such protection of the karst aquifer system is also of significant concern for design of the roadway.

PROTECTION OF SKYLINE CAVERNS

The commercially operated Skyline Caverns is located adjacent to Rt. 340 within the study area. The Skyline Caverns cave system was discovered in 1939 and is considered the largest documented cave system in Warren County with over 1,689 meters of mapped cave passage (Front Royal Grotto, 1997; Holsinger, 1975). The commercially maintained portion of the cave accessible to tourists includes approximately 500 meters of passage within the northwestern portion of the cave system. This portion of the cave system is also located at the margin of the planned construction limits of the planned Rt. 340 highway widening. Protection of the Skyline Caverns from effects of roadway construction, including vibrations and rock fragmentation from excavation and blasting activities in the vicinity of the caves, is a primary concern of VDOT and the roadway designers (HSMM). Maintaining safe operation of cavern tours for the duration of the construction was an obvious concern of the Skyline Caverns commercial operation.

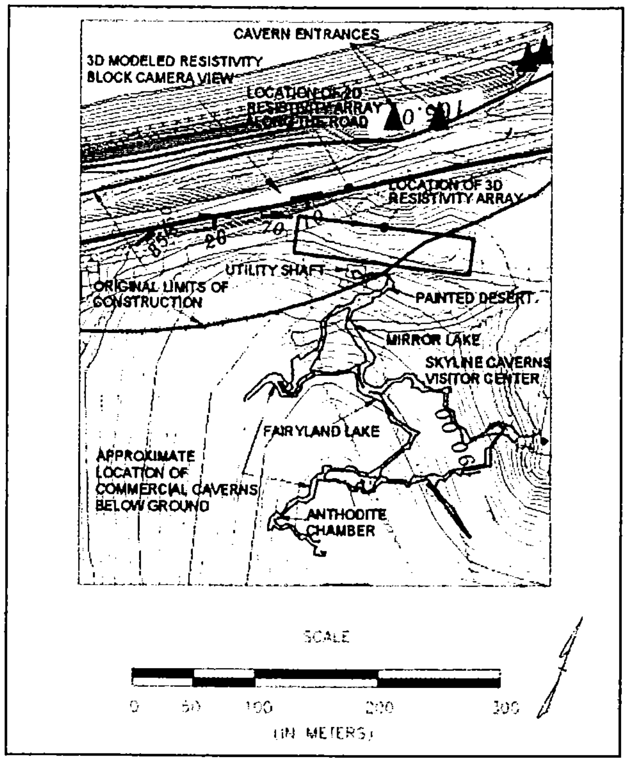

Figure 3. Location of Skyline Caverns and Rt. 340 Construction.

A map overlay of a portion of the Skyline Cavern system in relation to the original construction right of way is indicated in Figure 3. Road cuts into mixed residual soil and bedrock materials up to 18 meters high with 2H:1V slopes were originally planned in the vicinity of Skyline Caverns. A utility shaft servicing the Skyline Caverns at its northwestern limit is shown on the cavern map and is located within the planned construction limits. At its closest point, the roof and floor of the "Painted Desert" (see Figure 3) region of the Skyline Caverns are about 17 meters and 20 meters, respectively, below the originally planned right-of-way construction scenarios.

The Skyline Caverns attraction has been developed around a natural limestone cave system that contains underground streams and ponds, as well as unique cave formations, including flowstone, stalactites, stalagmites, and anthodites. Examples of the delicate anthodite formations, composed of calcite and aragonite, are shown Figure 4. The anthodite features are typically only a few millimeters in diameter and up to 400 millimeters long.

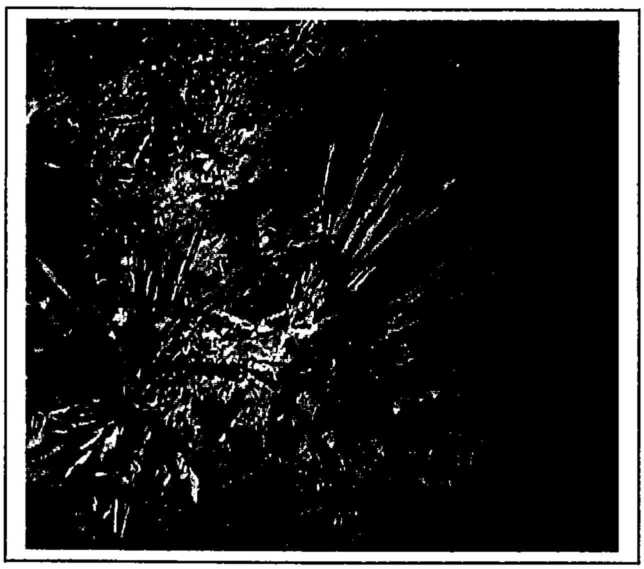

Figure 4. Anthodite Formation in Skyline Caverns.

Ground water supplying underground streams and pools within the caverns is fed by sinking streams located a few hundred meters east of the cavern within the slopes of the Blue Ridge. Discharge of ground water occurs within the river bed of the South Fork through bedrock fractures and faults (VDCR, NPS, FRGA, memoranda dated 1994 through 1999), well below the maximum depths of planned roadway construction excavations.

FIELD INVESTIGATION METHODS

Geologic Mapping and Cavern Investigations

Prior to beginning geologic mapping and cavern investigations, existing geologic and hydrogeologic information and stereoscopic aerial photographs were reviewed in an effort to identify fracture/fault traces and specific geologic structures within the study area. These structures were mapped to further evaluate their likely influence on the stability of rock slopes, as well as to evaluate the potential for sinkhole and cave formation along these features.

Upon completion of the literature review, field mapping of exposed geologic features, including detailed structure mapping of exposed rock cuts along the proposed alignment, identification of alluvial/colluvial deposits, and mapping of bedrock outcrops, sinkholes, and cave entrances was conducted. The detailed mapping characterized ground surface conditions across the area and supplemented subsurface data gathered during the resistivity surveys and drilling program. The surface and subsurface characterizations were used in conjunction to identify environmentally sensitive areas that could be adversely impacted by the proposed construction, as well as to provide mitigation recommendations for construction.

Figure 5. Open Cavity in Bedrock

Mapping of existing and potential cave entrances and sinkholes along the proposed roadway alignment was carried out with the assistance of a knowledgeable member of the Front Royal Grotto Association who had extensive knowledge of documented cave and sinkhole locations, as well as historical and environmental knowledge of the general area. The assistance of the Front Royal Grotto was also instrumental in obtaining 100 percent landowner cooperation in gaining access to private property for cavern and sinkhole identification, as well as for drill rig access. Numerous caves, as well as suspected cave entrances, were identified and mapped along the proposed roadway alignment. In addition to the commercially developed Skyline Caverns and other well documented caves such as Allen's Cave and Horseshoe Cave, numerous smaller caves and potential cave entrances were documented along the alignment.

In addition to identification of caves, numerous sinkholes, some of which had been in-filled by landowners, were identified along the proposed alignment. Of particular interest during cavern and sinkhole mapping activities was an actively subsiding sinkhole located on Skyline Caverns property. Several past attempts to fill the sinkhole with soil had failed due channeling of stormwater runoff from the parking area into the sinkhole. During the mapping reconnaissance, a cave opening located approximately 50 to 60 meters west of the active parking area sinkhole was discovered to be in-filled with soil. It was inferred that the soil infilling within the cave opening was washed from the sinkhole, through openings in the bedrock and out onto a limestone bluff above the South Fork.

Drilling Program

A comprehensive drilling program including soil borings and rock coring was developed based on the geologic reconnaissance and the two-dimensional resistivity imaging. The drilling program targeted possible karst features identified during the geologic reconnaissance and resistivity imaging, and also provided information on subgrade soil conditions and rock quality and structure information. In general, the borings correlated well with the geologic mapping and resistivity profiles.

Resisitivity Imaging Methods (2D and 3D)

Surface geophysical techniques are commonly used in subsurface characterizations of karst conditions to help select locations for more invasive investigative techniques such as drilling, or to supplement existing information on subsurface conditions from existing drilling data and observations of surface features such as sinkholes, springs, fracture trends and lineaments. Features commonly associated with karst geologies that are often the target of geophysical investigations include depth to bedrock and pinnacled bedrock profile conditions, ledge rock conditions, and location and size of voids or cavities in soil overburden and rock.

Development of relatively fast, resistivity imaging techniques has enabled the engineering geophysicist and geologist to map relatively complex subsurface geologic conditions in karst in greater detail and depth than what could be reasonably assessed using more traditional geophysical techniques, such as one-dimensional resisitivity profiling, resistivity sounding, or microgravity methods. The resistivity imaging method involves field measurements made using a computer-controlled data acquisition system connected to a linear or grid array of electrodes inserted in the ground at a constant spacing. Inversion modeling of the resulting data set is carried out using a finite difference or finite element subroutine that provides an approximation of the true earth resistivity distribution (Loke, 1997; Loke, 1998). Anomalies within the modeled data may then be used to help identify voids, rock ledges, bedrock fractures and thickness of soil overburden in karst terrain.

For the Rt. 340 study, two-dimensional (2D) and three-dimensional (3D) resistivity imaging arrays were used to characterize subsurface conditions and supplement boring data along the entire 4.6 kilometer alignment study area, and where deep cuts are planned for the highway widening in the vicinity of the Skyline Caverns. Examples of modeled 2D resistivity image cross sections for selected sections of the proposed highway centerline are provided in Figure 6, indicating shallow and solution-weathered bedrock conditions near the Skyline Caverns, and in Figure 7, indicating a deeper residual soil profile with overhanging bedrock ledge conditions at another location. The distribution of modeled resistivity values in the subsurface are indicated in cross sections by colorized contours. Coutour labels indicate the resistivity values for specific contour intervals in the model.

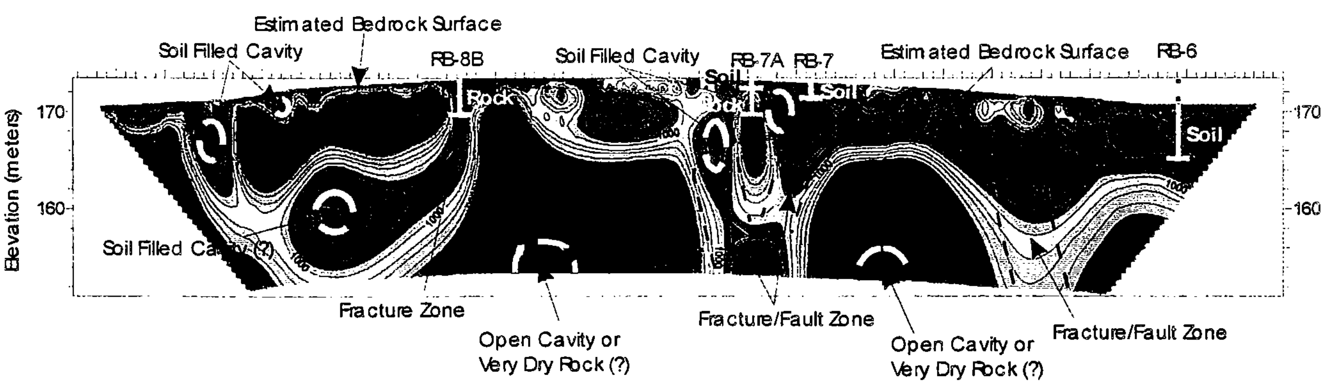

Figure 6. Modeled Resistivity Cross Section in Shallow Rock Near Skyline Caverns.

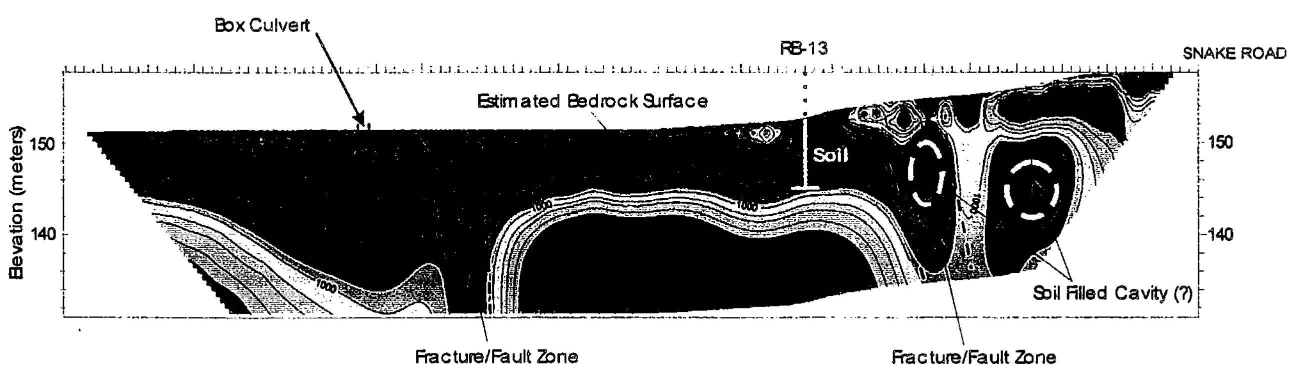

Figure 7. Modeled Resistivity Cross Section in Deeper Residual Soil Profile with Overhanging Rock.

The 2D resistivity images were used in this investigation to estimate the distribution of earth materials between boring locations, including estimating the bedrock surface profile and the location of possible air-filled or mud-filled bedrock cavities. The image cross sections have been annotated to indicate the estimated subsurface conditions beneath the array. Borings located on or near the array have been projected to the cross section and conditions encountered in the borings are indicated. In general, the brighter colored resistivity contours shown on the cross sections, such as red, yellow, and darker green, indicate materials of higher resistivity, including relatively dry limestone bedrock and weathered bedrock materials in the 300 ohm-meter to 100,000 ohm-meter range. More conductive (lower resistive) lighter green- and blue-colored resistivity contours less than about 300 ohm-meters, indicate softer, wetter residual soil and disintegrated rock conditions, including soil and mud-filled bedrock cavities and fractures.

Regions of resistivity highs greater than about 100,000 ohm-meters were considered as possible open air-filled cavities. These anomalies, as well as anomalies associated with potential soil or mud-filled bedrock cavities were targeted for additional borings where these features were observed in resistivity images near planned highway subgrade elevations. Although subsurface conditions from the resistivity cross sections have been estimated using the general criteria described above, it should be pointed out that the resistivity values associated with the interpreted anomalies are not unique to these materials and should be considered as a qualitative estimate of subsurface conditions.

To provide a broader representation of subsurface conditions at the construction limits adjacent to the Skyline Caverns, a 3D resistivity survey was performed near the top of the planned road cut in front of the utility shaft opening at the ground surface (see Figure 3). The 3D array grid measured 15 meters wide by 39 meters long. The location of the 3D array is indicated in Figure 3. Figure 8 provides a 3D resistivity model image of subsurface conditions. The utility shaft is provided for reference in the 3D image. Selected ranges of resistivity values have been removed from the model to highlight the estimated bedrock surface and the bedrock fractures associated with weak zones and possible cavern development at depth that are expected to impact design and construction of highway cut slopes at this location.

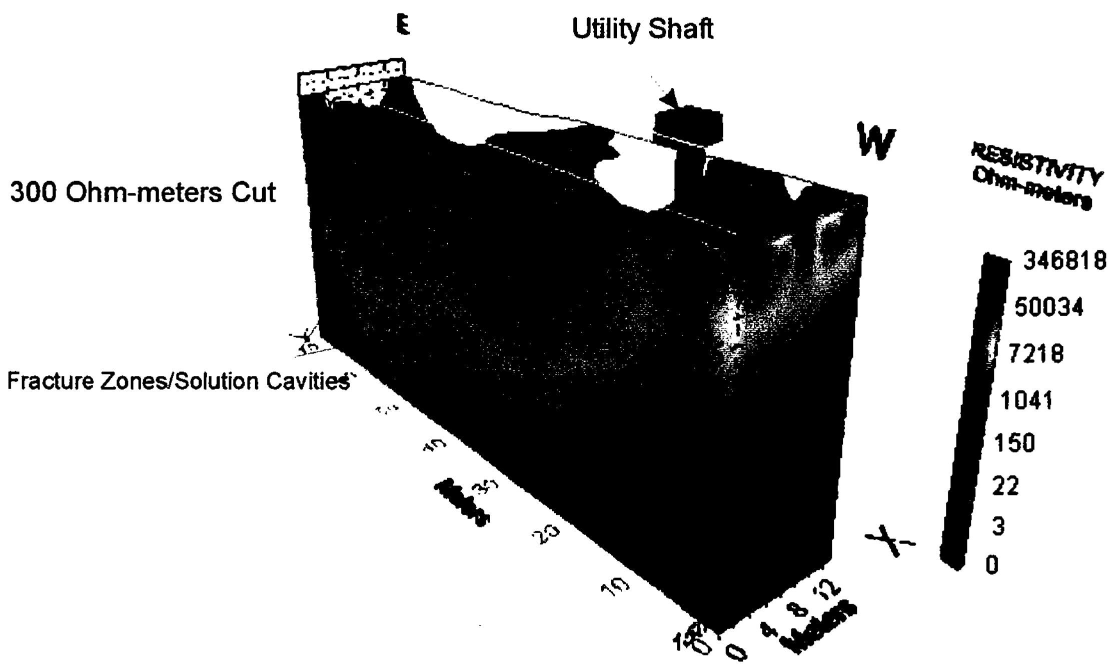

Figure 8. 3D Resisitivity Block Image Showing Estimated Bedrock Surface in Front of Skyline Caverns Utility Shaft (resistivity contours less than 300 ohm-meters removed).

Due to the number of calculations required to compute the 3D model, the length of time required to collect and process the resistivity data is several times greater than for 2D applications using single channel resistivity systems. As a result, the application of 3D models was limited to one location for this project. Multi-channel resistivity data collection units which have become available commercially within the past year have substantially increased the rate of data collection for 3D arrays and promise to make this method of subsurface modeling more feasible for future studies. In addition, the present limitation of 3D modeling algorithms to pole-pole electrode arrays also provides less resolution in the modeled image than the 2D models which use data collected from dipole-dipole electrode array geometries. Relatively small targets, such as discrete cave openings, typically cannot be resolved well within the 3D model, particularly as depth increases. Future improvements to the currently available model algorithms of 3D resistivity imaging data to increase model resolution will also encourage greater application of 3D resistivity image modeling in karst terrains.

<u>**Vibration Response of Cave Structures**</u>

Response of delicate cave formations within the Skyline Caverns to blast vibrations from nearby rock excavations for construction of the Rt. 340 widening was analyzed to establish recommended vibration control limits and recommended limits on rock excavation methods that could be used in the vicinity of the caverns. Sensitivity analyses were performed on delicate cave formations, including anthodites and stalactites using three-dimensional models of the anthodites developed in the numerical simulation computer program SAP2000.

Ranges of anthodite and stalactite size and estimated material properties, orientation of formations in space, distance to cave formation from the closest expected blast locations, and empirical blast vibration attenuation criteria were used in the SAP2000 sensitivity analyses to estimate tensile stress levels generated in the formations from construction blasting. These stresses were then compared to the tensile strength of the cave formations for a range of material properties and formation sizes (length and diameter), and orientations. Cracking failure of the cave formations from blast vibrations was estimated to occur where the tensile strength of the formation was exceeded by the tensile stress induced by the blast vibrations.

The orientation of stalactites for the analysis was considered to be uniformly downward and vertical. The orientation of the thin, individual spines of the anthodite "flower structures", as shown in Figure 4, were considered in the vertical downward, horizontal, as well as the inclined upward and inclined downward positions. Figure 9 indicates the range of anthodite orientations analyzed.

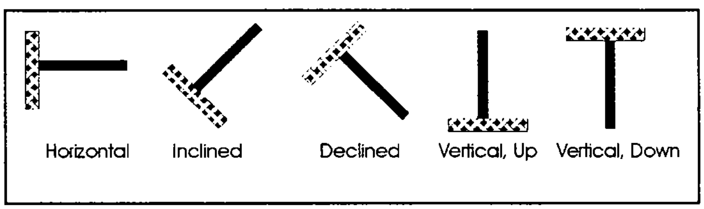

Figure 9. Orientation of Anthodite Spines Analyzed

The selection of a characteristic "blast" to use in the analysis was based on available ground motion time histories from quarry blasts in similar geologic conditions. Conventional production blast parameters were assumed in the analysis, including use of ANFO explosive in a 6 meter deep borehole with an estimated explosive charge weight per delay of 15 kg (31 pounds). Attenuation of ground vibrations from production blasting with distance, measured as peak particle velocity (PPV), was estimated using attenuation relationships developed from regression analyses of a suite of blast vibration time histories. The results of the regression analyses allowed for estimation of maximum blast vibration levels for specific locations within the Skyline Caverns where the cave structures of interest were analyzed. The blast time histories were then scaled such that the peak particle velocities matched the estimated maximum vibration levels for the analysis locations. To capture the response of the cave structures to a range of similar ground motions with different frequency and amplitude content, response spectra were developed scaled to the expected maximum peak ground motion components.

The regression analyses of peak particle velocity assumed "normal confinement" conditions during a production blast. Scaled distance (SD) values and distance from the blast to the point of interest were used in the regression equation to estimate peak particle velocity values at specific locations.

The scaled distance value is calculated using:

$$SD = D / (W)^{1/2}$$

where: **SD** = Scaled Distance
D = Distance from blast to point of interest (feet)
W = Maximum explosive charge weight per delay (pounds)

The estimated attenuation relation between the expected peak particle velocity from a blast to the scaled distance for that blast is approximated for the Skyline Caverns site as:

$$PPV = 160(SD)^{-1.6}$$

where: **PPV** = Predicted peak particle velocity (inches per second)

Using this relation, maximum peak particle velocities were estimated assuming a maximum charge weight per delay of 15 kg (31 pounds) for the locations indicated in Table 1:

Table 1: Skyline Caverns Blast Vibration Estimates

Cavern Location (formation type)	Estimated Closest Distance From Production Blast in meters (feet)	Scaled Distance	Maximum Estimated PPV in mm/second (ips)
Utility Shaft	10 (33)	6	230 (9)
Painted Desert (stalactites)	20 (66)	12	76 (3)
Anthodite Chambers (anthodites)	80 (262)	47	8 (0.3)

Results of the analysis of blast effects on cave structures indicated that tensile stresses generated within anthodites of various documented orientations and sizes, and within the documented range of sizes of stalactite structures would not exceed the estimated tensile strength of these structures. Therefore, from the analysis results and the estimated maximum production blast vibration levels indicated in Table 1, the cave formations should be stable for the blast parameters and blast distances assumed in the analysis. The vibration levels from "typical" construction blasting are also expected to be at levels low enough within the cavern not to produce fracturing and slabbing failure of the rock material in the roof and walls of the caverns and utility shaft (Atlas, 1987; Oriard and Coulson, 1980).

RESULTS OF INVESTIGATION

Highway Right of Way Study

Results of the field investigation using an integration of surface mapping, 2D and 3D resistivity imaging methods, and conventional soil boring and rock coring methods were crucial in providing the "big picture" defining the general subsurface conditions below the planned alignment for the Rt. 340 widening. Correlation of surface mapping and 2D resistivity imaging cross sections were useful in constraining areas of existing and potential sinkholes and bedrock cavities for discrete sections of the highway alignment. No significant open bedrock cavities were detected in borings or resistivity image cross sections immediately below planned roadway and slope subgrades. However, several existing and potential sinkholes and mud-filled bedrock cavities were encountered at the ground surface, in borings, and on resistivity image cross sections at or near subgrade elevations. Planning for mitigation of these features in the design phase of the project has been completed with the extent of recommended methods of mitigation to be incorporated into the construction bid documents.

Estimates of where bedrock and bedrock ledges will be encountered, and where rock excavation will be required to achieve roadway and slope subgrades is also provided with greater confidence using the resistivity images with verification of these conditions provided by widely spaced borings. This information will assist in developing more realistic estimates of the extent of rock excavation for the project.

Skyline Caverns Vibration Analysis

The results of the numerical simulation and sensitivity analysis of the stability of cave formations in the Skyline Caverns indicated that, in general, stalactite and anthodite formations should be stable when subjected to blast vibrations from "typical" construction blasting for the Rt. 340 widening. However, to further protect the integrity of the cave and minimize the human perception of the severity of the ground shock, controlled blasting and alternate non-explosive rock excavation techniques were recommended for a 110 meter section of planned roadway widening construction adjacent to the Skyline Caverns. Limitation of production blasting to no closer than 40 meters to existing Skyline Caverns facilities including the utility shaft and underground passages was also recommended. Recommendations were also made to minimize slope excavations adjacent to the Skyline Caverns utility shaft by designing a steeper soil nail wall system to support the planned roadcut, thereby reducing construction-related vibrations in the vicinity of the Skyline Caverns and associated facilities.

Video surveys of accessible portions of the Skyline Caverns were recommended to document the pre-existing condition of the cavern and cavern formations. Seismograph monitoring stations established at key locations within the Skyline Caverns and at the ground surface next to the utility shaft entry were recommended to monitor blast vibrations during construction. Recommended limitations on allowable peak particle velocity at specific sites inside and outside of the Skyline Caverns were also established, including the surface opening of the utility shaft (75 mm per second [3 ips]); the Painted Desert (25 mm per second [1 ips]), and the Anthodite Chamber (3 mm per second [0.1 ips]). Test blasts were also recommended prior to production blasting outside of the recommended controlled blast area adjacent to the Skyline Caverns. During the test blasting, key locations in and outside the Skyline Caverns would be monitored. Modifications to the blast plan would be made based on the results of the monitoring, if necessary.

CONCLUSIONS

Integration of conventional surface mapping and drilling techniques, combined with 2D and 3D resistivity imaging methods was found to be effective in characterizing subsurface conditions in complex karst geologic conditions for the Rt. 340 widening study. Resistivity imaging was found to be particularly effective in estimating the bedrock surface profile, locating anomalies in the subsurface characteristic of potential sinkholes, bedrock cavities, and assisted in planning locations and depths of additional borings to verify anomalous conditions detected in the resistivity images. Lateral changes in subsurface conditions between borings could be evaluated using the resistivity images and used for estimating subsurface conditions to be encountered during construction in slope and pavement subgrades where no boring information is available. Surface mapping of sinkholes and cavern entrances, in conjunction with estimation of sinkhole and cavern development below planned roadway subgrades using resistivity images, was useful in constraining the location of these features along the highway alignment and developing effective measures to remediate these features during construction.

Detailed evaluation of the response of cave formations in the Skyline Caverns was useful in providing blast vibration criteria for protection of sensitive cave structures during construction of the highway widening. Adjustments in the planned slope excavation and construction techniques near the Skyline Caverns can be made to limit the impact of construction activities on the stability of delicate cavern formations, and to allow for minimal impact to the present commercial use of the caverns.

Finally, it should be mentioned that VDOT's recognition of the sensitive nature of the karst environment and potential impacts to residences and businesses effected by the Rt. 340 widening construction, including the Skyline Caverns, allowed for the detailed level of site investigation study performed for this project. Cooperation with local citizens groups, commercial businesses, and the local grotto association, and use of their available resources and local knowledge, was encouraged by VDOT. Schnabel pursued this philosophy and was successful in obtaining greater insight into important geotechnical consideration for site improvements and protection of site resources by taking advantage of the combined local knowledge of area karst experts and business owners.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the local expertise and assistance of Ms. Janet Tinkham of the Front Royal Grotto Association in mapping and locating cavern openings within the alignment study area, as well as providing valuable historical insight on karst-related site features. The authors would also like to acknowledge the excellent assistance and cooperation provided by the Skyline Caverns staff during the many phases of our field investigation and cavern study. Finally, the authors would like to acknowledge the assistance and support of Mssrs. Frank Gilmer and Vince Ruark of VDOT, and Mssrs. Paul Anderson and Mark Jamison of HSMM, who provided Schnabel the opportunity to be involved in this very interesting and important project.

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Application of Innovative Geophysical Methods for Subgrade Investigations in Karst Terrain

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ABSTRACT

Geophysical surveys were conducted for the Missouri Department of Transportation (MoDOT) by the Department of Geology and Geophysics at the University of Missouri-Rolla to determine the most probable cause or causes of ongoing subsidence along a distressed section of Interstate 44 in Springfield, Missouri. The Springfield area is associated with sinkholes and karst terrain. This particular section of highway had experienced gradual, but continual, subsidence that was visually detectable on both the shoulders and median. A sudden 1-meter diameter, 2.5-meter deep collapse on the shoulder signified the need of a rapid subgrade assessment procedure to ensure the safety of the traveling public. Ground penetrating radar (GPR) and shallow reflection seismic technologies were applied to the site immediately.

GPR and reflection seismic quickly assessed roadway and subsurface conditions with nondestructive, continuous profiles. They expedited both the investigation and mitigation of karst related voids. The geophysical surveys were successful. The GPR proved to be of useful utility in defining upward-propagating voids in embankment fill material. On the basis of interpretation of these data, MoDOT personnel were able to drill into the voids that had developed beneath the pavement (as a result of washing out of the fine-grained material of the embankment fill), and to devise an effective grouting plan for stabilization of the roadway.

The reflection seismic survey established the presence of reactivated paleosinkholes in the area that had developed along essentially north-northwest trending fault/fracture zones. These were responsible for swallowing the fill material as water drained through the embankment. The site was later revisited for confirmation of the effectiveness of the stabilization of the grouting plan. Duplicate GPR profiles were acquired and indicated that the grouting program had been effective and that no substantial voids had developed in the interim.

Evaluation of Electrical Resistivity Tomography for Delineating Areas That Are Susceptible to Subsidence and Sinkhole Collapse Along I-70 near Frederick, Maryland

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Abstract

Electrical resistivity tomography (ERT) was used to locate and characterize areas that are susceptible to subsidence and sinkhole collapse along a section of I-70 near Frederick, Maryland. The investigation site has undergone numerous sinkhole collapse incidents. It is underlain by Ordovician Grove Limestone and Cambrian Lime Kiln Limestone. Because of the irregular distribution of cutters and pinnacles on the bedrock surface, uncertainties arise when "hit-or-miss" borehole drilling is used to locate potential collapse sites. The formulation of a drilling plan based on a surface geophysical investigation that produces continuous 2-dimensional subsurface images is probably the most cost-effective strategy for identifying subsurface features related to sinkhole development.

ERT is a subsurface imaging technique that employs a computer-controlled multi-electrode system to collect electrical resistivity data and an inversion modeling software to process the data. It provides two- or three-dimensional resistivity models that represent subsurface conditions in greater detail and depth than other traditional geophysical methods. However, its successful application to karst terranes requires considerable geologic insight based on experience, proper design of the transects and appropriate interpretation of the resistivity images. The resolution of three different electrode configurations (Wenner, Schlumberger, and dipole-dipole) at two electrode spacings (5 feet and 10 feet) was evaluated for detection of karst features in an area 500 feet north of I-70, where nine sinkholes have occurred. Forward modeling was also conducted to evaluate the reliability of the inversion program. The processed images from both forward modeling and inverse modeling were compared with detailed ground truth data from borings. The dipole-dipole array produces the most detailed picture of the subsurface conditions. It is also subject to the most severe interference from near-surface inhomogeneities, especially at an electrode spacing of 5 feet. The inversion program appears to overestimate the depth to bedrock, while the forward modeling underestimates the depth to bedrock. They both provide reasonable representations of the subsurface geology.

The dipole-dipole array at a 10-foot electrode spacing was used to investigate the area along I-70. The transects were conducted on the shoulders of the highway and in the median. The results show a highly irregular bedrock surface with sharp pinnacles and deep cutters, which is consistent with the subsurface condition where detailed geologic information is available. Areas that may undermine the safety of I-70 have been delineated; they need to be further investigated using exploratory boreholes.

THREE-DIMENSIONAL TOMOGRAPHIC IMAGING FOR DEEP FOUNDATION INTEGRITY TESTING

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ABSTRACT

The ability to accurately assess the structural integrity of deep foundation elements remains a challenge to geotechnical engineers.

Although technological advancements in electronic sensors and data acquisition systems have improved the application of non-destructive testing, software development has fallen short in providing true 3-D representation of deep foundation integrity.

A new high-tech testing method called CSL TOMO3D™ is a tomographic ground imaging system that uses a non-destructive technology similar to that used in the cross-hole sonic or seismic logging (CSL) technique. The system software provides three-dimensional, graphical representations of subsurface conditions with greater detail and accuracy than traditional seismic tomography and other available CSL testing methods. The system is designed for high-quality assurance assessment of the structure interior of deep shafts and foundations.

This paper presents the application of CSL TOMO3D™ tomographic imaging technology in geotechnical site characterization and assessment of deep foundation structural integrity. Three case studies are discussed in which cross-hole sonic or seismic tomography was used to...

1. detect structural defects in a deep drilled shaft pier;
2. locate the depth of piles used in the construction of a bridge;
3. map the karstic limestone and evaluate the effectiveness of grouting a bridge pier foundation.

THE CSL TOMO3D™ SYSTEM

The CSL TOMO3D™ system consists of commercially available hardware and proprietary software. The hardware is simple and comprised of a receiver array, seismic source, cables, and a seismograph data acquisition system. The seismic energy source could be a seismo-acoustic source such as the signal/sound produced by an air gun, or the frequency produced by a magnetostrictive unit. In practice, data sets are collected as full waveform signals (shear or S-wave, or compressional or P-wave) that are recorded on the seismograph using a sampling rate and duration appropriate for the signal source utilized, the earth materials anticipated, and the physical scale of the study. Higher frequencies, such as the signal produced by the traditional CSL sources, are used on smaller targets or small physical dimensions.

The CSL TOMO3D™ software is a complete tomography package, from signal collection to CAD output display. The software incorporates highly sophisticated algorithms and modules for automated data collection, signal arrival event picking, and mesh initialization, data filtering and processing, and reconstruction of tomographic images. The tomographic inversion algorithm models seismic energy transmission and dispersion using a variation of the eikonal equation, a finite-difference approximation of the wave equation. Mesh properties are iteratively adjusted until simulated energy transmission measurements match field measurements, using a variation of SIRT (Simultaneous Iterative Reconstruction Technique). SIRT uses the concept of a ray path region. Within this region, the paths followed by seismic waves from a source to individual receivers are represented as velocity and attenuation ray paths. The ray paths (in this region) are not normally straight, but rather bend (refract, curved ray) depending on the physical properties of the medium or the velocity contrasts between various material units. For example, in uniform rock or low-velocity contrast mediums, the velocity and attenuation ray paths are generally straight, and faster processing techniques are employed.

Curved-ray velocity produces better results for geotechnical applications, but the processing technique in determining the location of the ray path is very complex. The curved-ray technique used in the CSL TOMO3D™ software computes the location using steepest descent on the travel-time mesh, nearly matching the computational efficiency of the straight-ray technique. The seismic signals are then analyzed to derive primarily two types of tomographic images, velocity or attenuation, depending on the type desired. A velocity tomogram is based on the time it takes for each ray to travel through the medium from the source to the receiver. An attenuation tomogram is based on the reduction in power of the seismic signal as it travels from the source to the receiver. Areas of lower attenuation contrast or higher velocity contrast generally correspond to areas of more competent or consolidated material (rock, soil, etc.). On the other hand, areas of high attenuation or low velocity represent areas of less consolidated or medium to soft material (Westman, et al., 1996; Nur, 1987; Shea-Albin, et al., 1991; Yu, 1992). These two tomographic techniques can be used to distinguish important features in rock, soils, or other materials, including karst cavities, old mine workings, fault zones, joint and shear zones, man-made structures (piles, drill shafts, foundations, etc.), soil or weak rock interfaces, slope failure zones, and sinkholes (Hanna, et al., 2000). This paper addresses applications where the velocity and attenuation tomographic techniques were used depending on specific site conditions.

CSL TOMO3D™ Features

CSL TOMO3D™ is a high-tech tomographic imaging method that utilizes seismic energy and processes the information using similar techniques as those used in medical CAT scans (Hanna, et al., 1998; Rock, et al., 1977). With the information that CSL TOMO3D™ provides, geotechnical site characterization is dramatically improved, and the risks and expenses associated with "**changed conditions**" claims can be greatly reduced. Some of the key features within the system include...

- Ability to provide "**forward models**" – tomographic simulations of the planned survey prior to actually conducting the fieldwork, allowing the client to (1) determine the level of imaging performance to be expected, and (2) optimize required borehole placement and geotechnical mapping within the site being characterized.
- Use of three-dimensional B-Spline Interpolation Network for automated arrival event picking and mesh initialization. This application greatly expedites the solution of large, complicated imaging arrays. In addition, the interpolation network within CSL TOMO3D™ allows the available drill log information to be incorporated into the 3-D images to significantly improve the understanding of subsurface ground conditions.
- Ability to produce 2-D and 3-D color-coded velocity, attenuation, and "difference" images for improved assessment of the interior of deep foundation elements.
- Volumetric images to show velocity/energy variations for accurate detection of defects, their sizes and lateral extent, and defect classification.
- Compatibility with several variations of currently available non-destructive CSL equipment and techniques.

CSL TOMO3D™ APPLICATIONS

To date, numerous geotechnical ground-imaging projects have been successfully completed related to highways, bridges, slope excavation, retaining wall foundations, and tunnel constructions. The following section focuses on three case studies in the field of geotechnical engineering for deep foundations integrity testing.

Detection of Structural Defects in Deep Drilled Shaft Pier – Case Study 1

Generally, a forward model to simulate the planned survey is constructed prior to actually conducting the field survey. The purpose of this simulation is primarily to determine expected results or the level of accuracy for specific site conditions. Figure 1 shows forward velocity and inverse models of a planned cross-borehole survey to image concrete inclusions/defects within a 5-ft-square and 50-ft-deep bridge pier. An array of receivers and sources was placed in the pier using a configuration of four boreholes. The information on the concrete and the inclusion (defect) was used to assess the P-wave velocity. Figure 1A shows the velocity ray paths of the modeled pier. Figure 1B shows a 3-D forward velocity model representing a structural make-up of the pier. Figure 1C shows the reconstruction tomographic inversion. Comparison of the forward velocity model and its tomographic reconstruction demonstrates the average match for detecting defects within the bridge pier. This demonstrates the modeling capabilities in identifying structural defects and assessing the interior of the concrete structure.

Based on the above methodology, the integrity of a 3-ft-diameter drilled shaft foundation at a bridge construction project was evaluated using the non-destructive cross-hole sonic logging (CSL) technique. Three sonic logging tubes were installed in the shaft prior to construction and were used for testing. The tests were conducted eight days after the concrete was poured by placing the source and receiver at the bottom of each tube pair and retrieving the probes while the test equipment logged the probe location relative to the starting point at the bottom of the shaft. The cross-hole sonic logging technique is an indirect, low-strain, non-destructive method for evaluating drilled shaft and pile foundations. This method provides

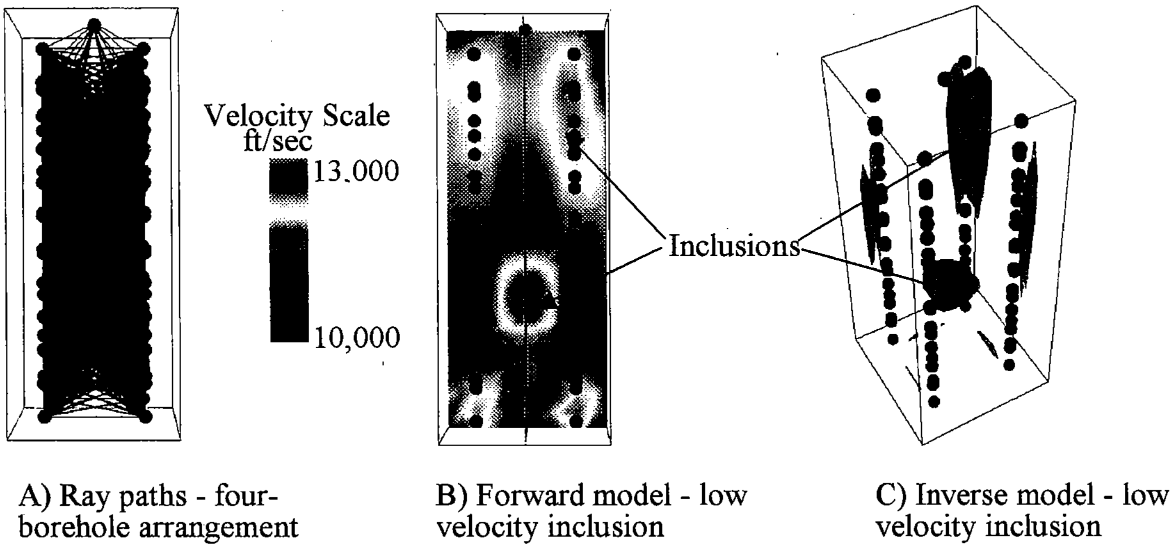


Figure 1. Three-dimensional tomographic simulation for imaging low velocity inclusions within a drill shaft pier.

information about the material in the zones directly between tested tube pairs, but does not provide information about material outside those zones or below depths at which the probes were lowered. Although it is typically assumed that changes in arrival times or amplitudes are attributed to changes in concrete quality, other factors such as change in the tube spacing along the shaft length or de-bonding of the tube from the surrounding concrete may also affect the results. All data in the tubes extended outside the shafts were discarded.

During and immediately after testing, the logs of arrival times were reviewed to assess signal uniformity. Knowing the distance between the tube pairs, the velocities were calculated. The results from one of the drilled shafts supporting the abutment were further analyzed using the CSL TOMO3D™ software to give a 3-D image of the interior of the shaft. Figure 2 shows the 3-D velocity image of the analyzed abutment shaft.

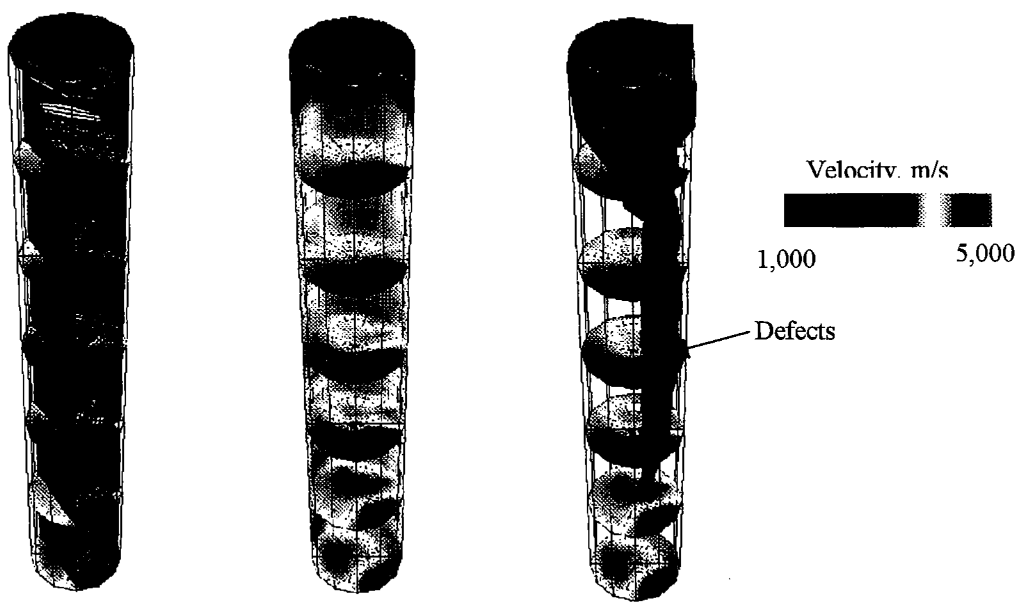


Figure 2. Three-dimensional images of the drilled shaft interior for a bridge foundation.

The image shows the ray path coverage within the shaft, cross-sections at 2-ft intervals, and 3-D contours of the areas below 2,000 m/s (6,560 ft/s). These areas indicated zones of possible defects. The tubes being placed outside the concrete cause the low-velocity zone shown in the upper 10 ft of the shaft.

The use of 3-D tomographic imaging for reproducing sonic data instead of the usual XY plots provides the site engineer with an intuitive picture for more accurate data interpretation, reduces error, and improves defect classification.

Location of Depth of Piles Underlying Bridge Piers – Case Study 2

The CSL TOMO3D™ was applied to two existing piles in support of the geotechnical subsurface investigation for the design and construction of Bridge No. 3 along S.R. 0028, Etna Interchange Project in Pittsburgh, Pennsylvania. The Pennsylvania Department of Transportation and their design consultants had some question as to whether or not the pilings had been driven to the top of the sandstone bedrock approximately 70 to 80 ft below the surface. A two-dimensional attenuation cross-hole tomography survey was performed to determine...

- **The tip elevation for the existing piling at Pier Nos. 1 and 2.**

<u>System Installation and Data Collection</u>

The tomography survey was performed pairing a magnetostrictive source and a string of hydrophones to propagate and capture seismic signals transmitted between source and receiver. Each source-receiver borehole pair results in data that form a vertical panel containing information on the velocity and/or attenuation structure between the holes. At each pier site, two vertical boreholes were drilled, one on either side of each pier (pile group). The boreholes were drilled 4 inches in diameter to a depth of approximately 100 to 120 ft below the surface. All four boreholes were cased with 2.5-in OD schedule 80 PVC solid pipe. Field logs of the boreholes were recorded for the tomographic survey. Figure 3 shows the plan view of the borehole location with respect to Bridge No. 3 and Pier Nos. 1 and 2, and the generalized geologic column. The lithology encountered in the boreholes consisted of 38 ft of clayey gravel and sand fill material, with a 6-ft-thick layer of cinders/slag fill. This was underlain by 30 to 40 ft of sand and gravel alluvium. The bedrock underlying this, at a depth of 70-80 ft below ground surface, was hard sandstone that was highly fractured near the surface.

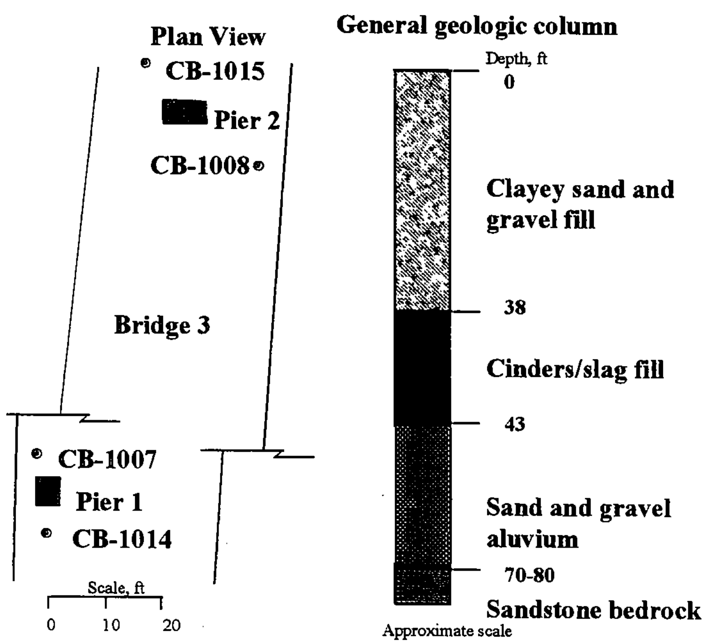


Figure 3. Plan view showing surface locations of Etna Bridge piers 1 and 2 and the boreholes surveyed and generalized geological column of the surveyed boreholes, Etna Interchange Bridge No. 3.

The survey was performed using a string of 12 hydrophones, a trigger, and amagnetostrictive source connected to a 24-channel seismograph. The hydrophone receivers were spaced at 6.6-ft intervals, with a total string length of 78 ft. These receivers have an internal preamplifier with a gain of 20 dB and operating frequency range of 7 Hz to 4.7 kHz. The source generated a sweep of frequencies from 200 – 2,000 Hz over a period of 764 ms. Data were collected at a sample rate of 8,000 samples per second for a time resolution of 0.25 msec and Nyquist frequency of 4,000 Hz. Data were recorded on the seismograph for 0.1524 seconds for each shot.

The survey procedures involved the following steps. All boreholes were filled with water to allow the use of the hydrophones and the source. For each borehole pair, the source was installed in one hole and the hydrophone string in the adjacent hole. The string was lowered into the hole until the lowest receiver was at the bottom of the hole. The source was moved along the depth of the hole, and seismic signals were generated at 3-ft intervals starting from the bottom of the hole up to 12 ft from the surface. The entire area for Pier Nos. 1 and 2 was surveyed in one day.

This process of moving sources and receivers was repeated for both borehole pairs until the data from all source and receiver locations in the vertical dimension had been acquired. This procedure allowed sufficient ray path coverage for the most accurate two-dimensional imaging of the area of interest. A total of 3,200 rays were measured between sources and receivers. Since hydrophones respond primarily to P-waves, the first arrivals (which would be P-waves) were used in the processing. The acquired data quality assessment, filtering, and analysis were performed to produce attenuation tomographic images using the CSL TOMO3D™ software discussed in the previous section.

Data Interpretation

Prior to the field survey, a velocity forward model was produced of the planned cross-hole survey to image the extent of the piles using the general geologic column depicted in Figure 3. As shown in Figure 4, the reconstruction two- and three-dimensional tomograms clearly indicate three distinct velocity contrast zones: (1) upper zone showing the extent of the piles; (2) middle zone indicating unconsolidated soft material; and (3) lower zone indicating highly consolidated bedrock. The results of the forward model simulation were used to evaluate the results of the field survey discussed below.

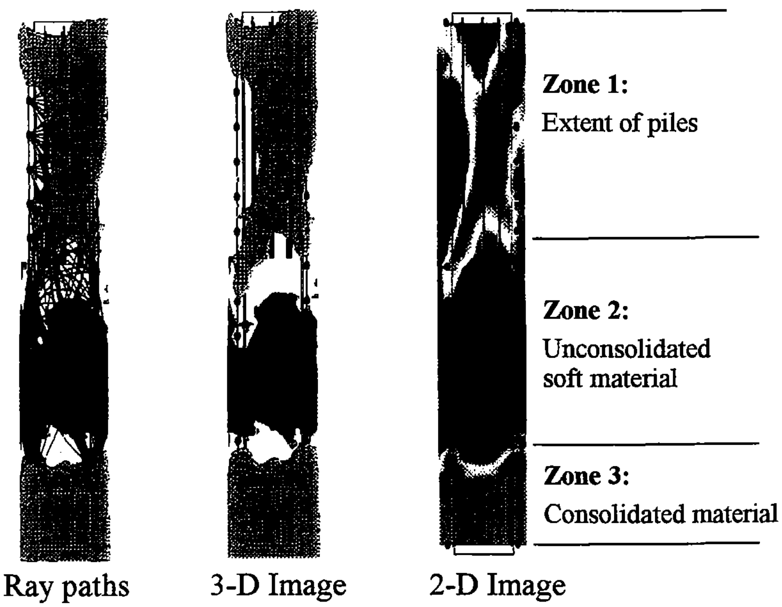


Figure 4. Inverse models for imaging the extent of bridge piers within the bridge foundation.

The actual representation of the survey area was constructed to produce a 2-D attenuation tomogram for both borehole pairs. A total of 520 mesh nodes and approximately 1,600 rays at 2-ft resolution for each panel were used in the tomogram computation. The tomogram plane is bounded by the borings on either side of the piers, as shown in Figure 5, and ranges in elevation from 759 ft to 664 ft ASL.

The tomograms in Figures 5A and 5B depict 2-D contours of the computed levels of attenuation through the planes between the two pairs of boreholes. The upper zones in each of the panels that are circled for Pier Nos. 1 and 2 are areas of high attenuation. These zones have been interpreted to be the extent of the pilings used in bridge construction. Below that layer is a zone of low attenuation that was interpreted to be a zone of saturated fill and sand and gravel. At the bottom of each of the tomograms is a zone of intermediate attenuation that is not as high as the upper layer. This zone was interpreted to represent bedrock. The depth range of these three zones is summarized in Table 1.

Table 1 – Depth ranges of interpreted attenuation zones.

Zone/Interpretation	**Depth Range, ft**	
	Pier 1	Pier 2
High attenuation zone – Piles length	759 – 712	759 – 710
Low attenuation zone – Saturated fill and sand and gravel	712 – 676	710 – 690
Intermediate attenuation zone – Sandstone bedrock	676 – Bottom of image	690 to Bottom of image

The rationale for this interpretation is that the signal will be highly attenuated when traveling through the medium where pilings are present. This attenuation will occur because of the scattering of seismic energy as the wave propagates through a zone of closely spaced piles. The piles have a significant acoustic impedance contrast with respect to the fill and would, therefore, reflect and refract the seismic energy, preventing it from traveling a straight path from source to receiver.

The layer just below the pilings (low attenuation zone) exhibits relatively lower attenuation, because the medium through which the seismic energy travels is uniform, and the soils are saturated. Initially, it was expected that even lower attenuation

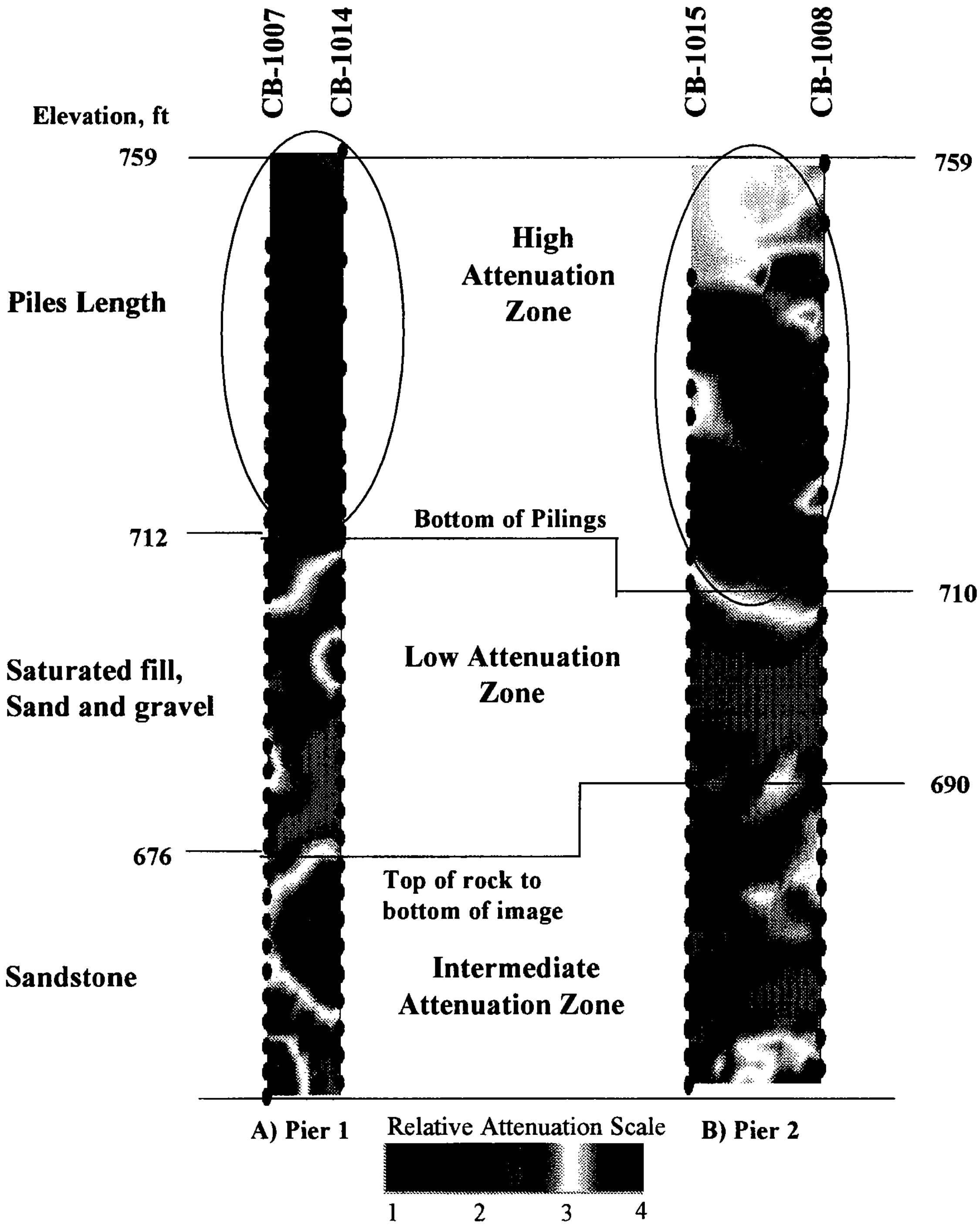


Figure 5. Two-dimensional tomographic images showing the zones of high attenuation as indicative of piles length for Piers 1 and 2.

would be seen in the bedrock. However, it is believed that the reason for the relatively higher attenuation is due to de-bonding of the casing from the surrounding rock that results in a high attenuation zone in the area right around the borehole. Alternatively, the high attenuation may be due to the presence of fracturing in the sandstone as noted in some borehole logs.

Based on the tomographic imaging results obtained from the CSL TOMO3D™ seismic survey and forward model, combined with on-site observations and geotechnical engineering experience, the survey was successful in determining the following:

- A zone in the upper 50 ft of the panel between each pair of boreholes showed anomalistically high attenuation. The high attenuation in this zone was interpreted to be due to the presence of piles between the boreholes. The lower attenuation below this zone was interpreted to mean that it did not contain piles.
- The length of the piles, as indicated by the high attenuation zones, is approximately 47 ft for Pier 1 and 49 ft for Pier 2.
- The piles did not extend to the bedrock.

Mapping Karstic Limestone and Evaluating Effectiveness of Grouting of Bridge Pier Foundation – Case Study 3

Project Background

The Bill Emerson Memorial Bridge is being constructed over the Mississippi River between Missouri and Illinois near Cape Girardeau, Missouri. The river bottom consists of karstic limestone underlying the foundation of Pier 3. During the geotechnical site characterization and subsequent investigation, it was determined that structural defects, such as voids or clay-filled spaces, in the limestone formation would require substantial reinforcement for the load-bearing portion of the Pier 3 foundation. High-pressure jet grouting was determined to be the most cost-effective remediation without compromising the original design criteria for the pier (Hanna, et al., 2000).

The degree of ground stability achieved by the jet grouting operations is largely dependent on knowledge of the site stratigraphy for correct placement and injection of grout. Seismic imaging techniques can be utilized to indirectly measure the degree of ground stability enhancement after grouting. Jet grouting increases the strength, density, and velocity of the rock mass by displacing water or clay within weak or fractured zones. Changes in velocity between the two phases can be used to indicate the degree of effectiveness of the grouting operations.

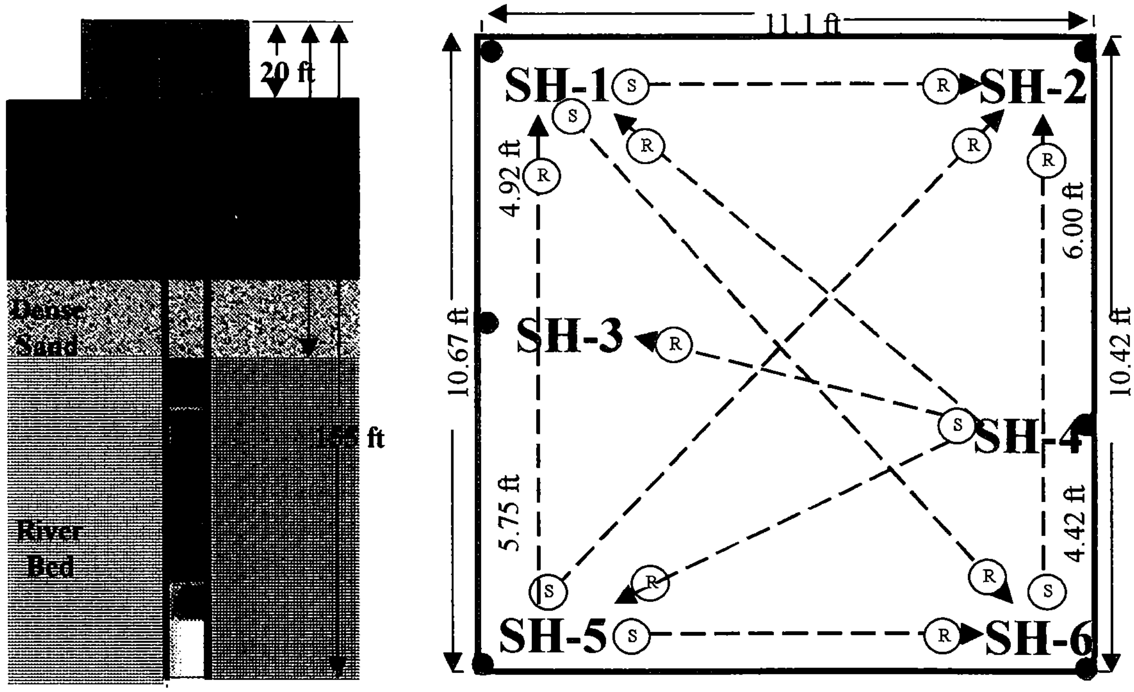
As part of the Missouri Department of Transportation (MODOT) compliance testing of the grouting program, a two-phase regime of tomographic imaging was proposed. The first phase of imaging was to be performed prior to the jet-grouting program, and the second phase was to be performed after jet grouting. Three-dimensional cross-borehole tomography surveys were performed using the CSL TOMO3D™ imaging system. The purpose of the investigation was twofold:

- Identification and location of karst features such as voids or clay-filled spaces within the geologic profile of the Pier 3 foundation – Phase 1: Pre-grouting.
- Evaluation of high-pressure jet-grouting effectiveness – Phase II: Post-grouting.

System Installation and Data Collection

The seismic surveys for the pre- and post-grouting phases were performed at two separate times. The equipment used and the data collection and processing procedures were identical for both surveys. Each survey was conducted at the bridge site from a construction platform in the river. As shown in Figure 6, a total of six boreholes (Figure 6A) were drilled to a depth of approximately 160 ft and cased with 6-in ID perforated schedule 40 PVC in two parallel rows of three holes each. Several pockets of clay, sand, and mud were encountered during drilling. The rectangular surveyed area bounded by the boreholes was 11 ft long by 10 ft wide and extended to about 160 ft (elevation 186) below the platform. The water level at the time of the survey was approximately 20 ft below the platform, and the river bottom was about 95 ft below the platform.

The survey was performed using the equipment and survey procedures discussed in Case Study 2, except that the seismic source utilized was an airgun. The airgun is operated at a chamber pressure of 1,000 psi. Compressed nitrogen cylinders charged to 3,000 psi were used to operate the airgun. The sudden release of the gas pressure results in a seismic pulse that is



A) Cross-section of the bridge construction site

B) Plan view of borehole locations on platform

Figure 6. Borehole locations and test procedures showing sources (S) to receivers (R) configuration for Pier 3 foundation, Bill Emerson Bridge.

detectable with the hydrophones. The airgun produces a very consistent, repeatable pulse with a fairly flat spectrum between approximately 10 Hz and 180 Hz. Transmitted frequencies decrease approximately linearly from 180 to 300 Hz. Data were collected at 16,000 samples/sec multiplexed over a 1/8-sec duration, with a 5-ms pre-trigger for each hydrophone receiver channel. Pairs of adjacent boreholes were surveyed in a manner that would permit three-dimensional imaging of the volume and/or two-dimensional images of cross-sections (panels) between any two boreholes in a configuration depicted in Figure 6B. The source and receiver locations were advanced in leapfrog fashion until all adjacent hole pairs were surveyed. Each survey involved lowering the hydrophone receivers string into the borehole until the lowermost hydrophone was close to the bottom of the interval to be surveyed.

Data Interpretation

Data processing and analysis were performed using the software package discussed earlier. A total of over 4,000 ray paths were collected in each phase. The curved-ray velocity method was applied to produce tomographic images. For comparison with the images obtained prior to grouting, tomograms were numbered using a similar numbering system used in Phase 1. The same velocity scale was also used. The tomograms were generated with velocity contour ranging from 5,000 to 20,000 ft/s on the velocity scales. The velocity values are relative to the rock type. A velocity of 5,000 ft/s indicates areas of lowest acoustic velocity or relatively weaker areas of dense sand or mud-filled zones. Over 10,000 ft/s indicates areas of grouted zones, and areas of over 15,000 ft/s indicate zones of possible concentrated grout.

The well-log interpolation was generated from material descriptions listed on borehole log sheets provided by the site engineer. The data were entered into the CSL TOMO3D™ software and well-log images were reconstructed using B-Spline Interpolation Network. This information was then compared with the tomographic imaging results to improve the understanding of the subsurface conditions and enhance the accuracy of the tomographic imaging process.

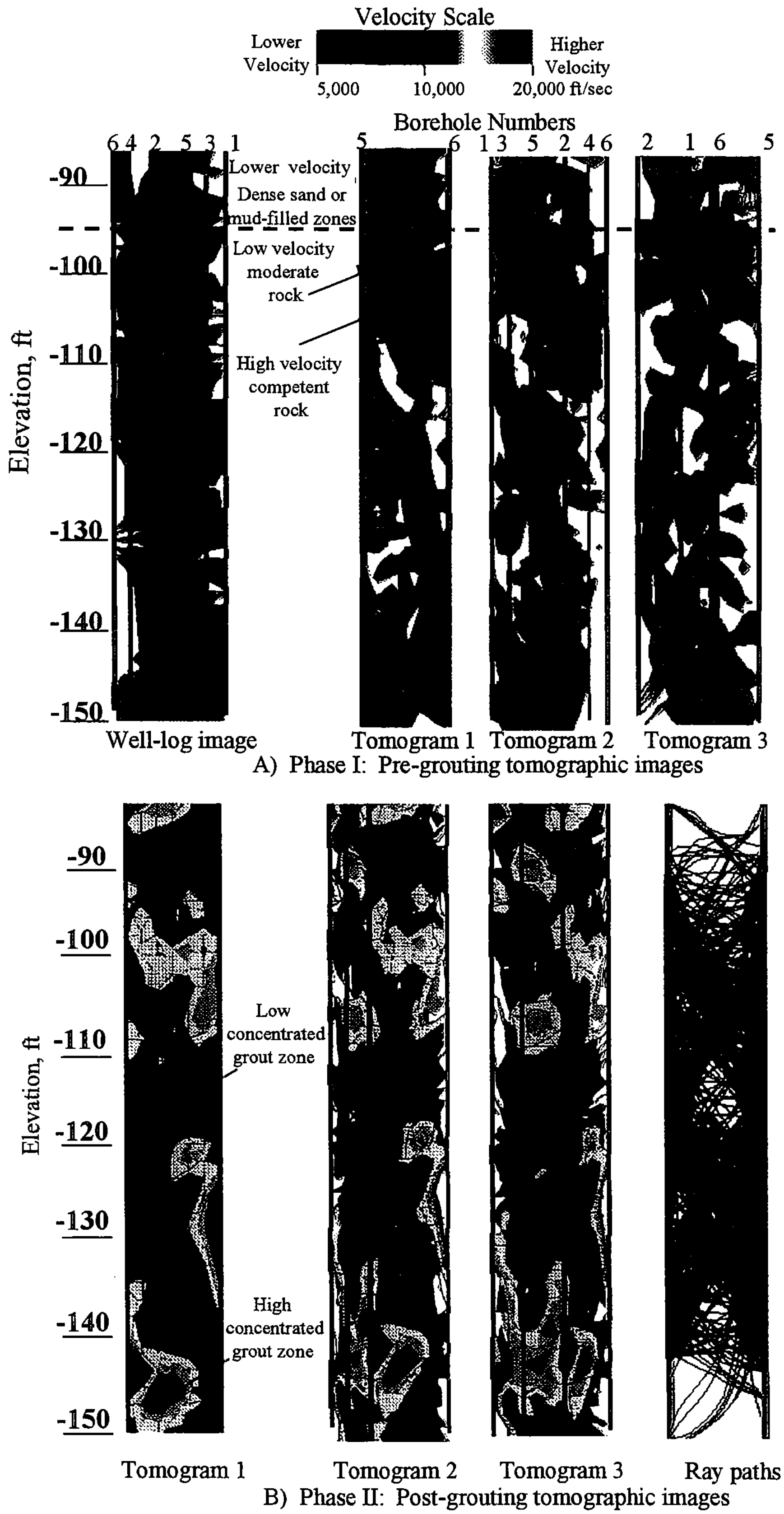


Figure 7. Pre- and Post-grout images beneath river bottom formation underlying the formation of the Pier 3 foundation.

Figure 7 shows the interpolated well-log images and similar sections of the resulting tomogram before and after grouting between -87 and –150-ft elevation. As shown in Figure 7a (pre-grouting survey), lower velocity zones were mapped between –87 and –95, at -110 ft and between -125 and -140 ft. These zones shown in purple are indicative of areas within the rock mass that have lower velocity and may infer less competent material such as dense sand, mud or clay-filled or fractured zones. The well-logs also identified weak zones occurring between -106 and -114 and between -130 and -138 ft. The areas in blue are indicative of zones of moderate rock competencies, and areas in green are indicative of the most competent rock mass. In comparison with the well-log geological data, these images show close correlation and can be adequately interpolated to provide an accurate description of ground conditions. Results of the Phase 2 post-grouting survey (Figure 7B) indicate that jet grouting significantly increased seismic velocities throughout the treated volume. The tomograms show few zones of lower velocities (8,000 ft/sec), with the exception of very small zones at about the -90-ft level and the -145-ft level. These small zones may not be representative of the area due to the limits of the survey. Another area also showed a relatively weaker zone (dark purple), which might indicate a zone with low grout. Most of the other areas indicated velocities higher than 10,000 ft/sec (blue-to-green), which indicates that the grouting was successful. Areas with a velocity of 15,000 ft/sec (yellow-to-red) may indicate zones of concentrated grout. The results of this investigation allowed MODOT to confirm that the jet grouting effort was adequate for the pier foundation as designed.

CONCLUSIONS

The traditional non-destructive methods of assessing the integrity of deep foundation elements have fallen short of providing true 3-D representation of the subsurface site conditions. CSL TOMO3D™ seismic tomography has evolved into a powerful geotechnical tool for high-quality assurance assessment of various constructed support systems.

The case studies for deep foundation integrity assessment presented here are but a few applications of the many projects where 3-D seismic tomography has been applied to highways, bridge piers, retaining wall foundations, sinkholes, old mine workings, slope excavation, and sewer and tunnel site characterization. To date, the use of the 3-D tomography technology in providing volumetric images has allowed the design engineers and contractors to better understand the complex geologic environment of the site, and the risks and expenses associated with "**changed conditions**" can be greatly reduced.

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Slake Durability of Deep River Triassic Basin Rock

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ABSTRACT: Rock from the Triassic Basin is primarily sedimentary in nature, exhibiting a wide range of behavioral responses following excavation. One aspect of this material is its' ability to slake. Defined as the short term physical disintegration of a geologic material following the removal of confining stresses (Shamberger, Patrick and Lutton, 1975), slaking changes the strength, durability and other characteristics of these materials by orders of magnitude, if the rocks are allowed to dry, shrink or swell (Franklin, 1981). The underlying cause of excessive settlement and slope failures in this region's highway embankments appears to be this deterioration or loss of strength. Previous studies on shale have attempted to correlate strength with slaking potential; however, relationships vary depending upon the material and the geologic location from which the samples were gathered. To determine the durability of the rock from the Deep River Triassic Basin of North Carolina, this paper examines the interdependency between slaking, plasticity and strength in an attempt to provide a qualitative correlation for this relationship. Strength parameters for 25 separate samples are analyzed using uniaxial compression and split-tensile strength tests. To ascertain the plasticity of the shales, standard Atterberg limits, as well as hydrometer analysis for clay fractions, were performed on the weaker material. Test results are presented as a comparison to the slake-durability indices (percentage of dry mass retained after two wetting and drying cycles incorporating abrasion) of the samples. A discussion and evaluation for determining potentially weak material by means of durability is proposed.

INTRODUCTION: Engineers have had a problem identifying non-durable, Triassic Basin rock since the distinction between weak and strong rock is difficult to ascertain. Researchers have used a variety of methods to determine the strength and durability of weak rock with wide ranging results. Underwood (1967) determined that the compressive strength of shale ranges from less than 25 psi for softer shales to more than 25,000 psi for those well cemented. This shows a wide variance in material that may look and feel similar to construction site personnel. In another study, Taylor (1988) developed a mudrock durability classification for British Coal Measures based on the unconfined compressive strength (UCS), third cycle slake durability values, and the composition and fabric of the rock samples. In a later paper, Shakoor and Sarman (1997) explored the swelling potential of mudrock and concluded that swell was based on geological characteristics of the rock itself.

In the study of classifications based entirely on compressive strengths, Afrouz (1992) summarized a number of systems proposed by various researchers and institutions. From the summarization, rock in general with compressive strengths exceeding 2900-11,600 psi are recognized as "hard" or "durable" while rock with compressive strengths below 725 psi is "soft" or "weak" or non-durable, Santi (1998). Previous research on the durability of shales has attempted to correlate strength or plasticity with slaking indices; however, relationships vary depending upon the type of material and the geologic location from which the samples were gathered. Walkinshaw and Santi, (1995) stated that transitional material classification appears to be subject to location rather than generic rock qualities. The problem, from an engineering point of view is the classification of the rock as far as a strength determination for construction purposes is concerned. It is understood that material in the range between soil (UCS = 145 psi) and hard rock (UCS > 2900 psi) is difficult to characterize since no sedimentary material could be perfectly homogenous in nature. It is also difficult to determine to what extent a material is considered durable and non-durable. Since the slake durability test appears to be the most universally accepted method to qualify weak shales, it must be accepted that compressive strengths below 1400 psi is not considered adequate for construction without incorporating additional treatment. Rock with a slake durability index (I_{d2}) less than 50% is considered unfit for construction rock purposes. It is easily recognized from its lack of physical integrity. Likewise, rock with an I_{d2} greater than 90% is identifiable by simple analysis. It does not require additional testing to determine its' potential for degradation. It is in the zone between these two values of slake durability that the visual properties are not evident by simple laboratory tests. Since potentially weak rock changes physically with local geologic variances, material from the Triassic Basin required further investigation both geologically and through engineering analysis.

GEOLOGY:

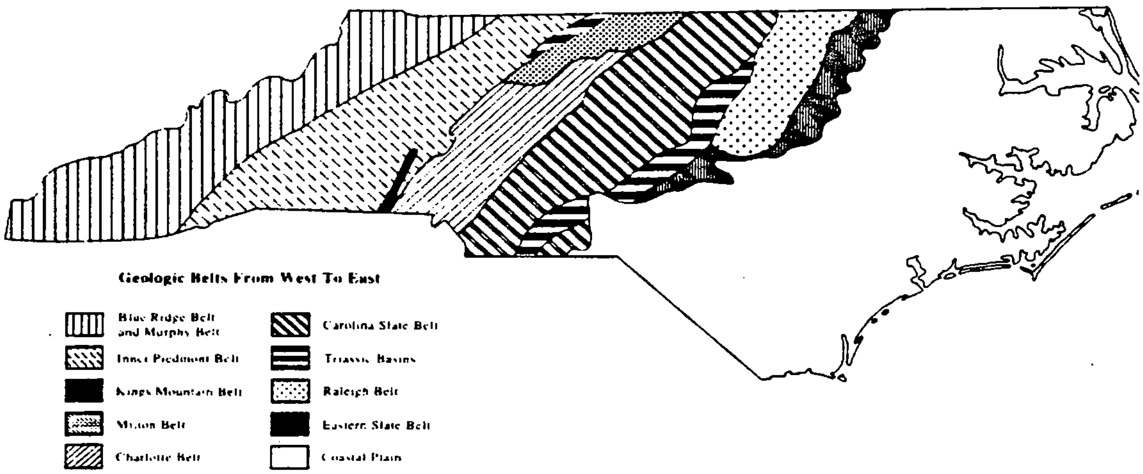


Figure 1. Geologic Basins of North Carolina

The Deep River Triassic Basin is part of a series of extensional basins in eastern North America that developed during rifting in the early Mesozoic and is known as the Newark Supergroup. The rifting event resulted in the separation of the North America and Africa plate. The Durham Basin and the Sanford and Wadesboro Basins to the south, form a 388-mile long by 42-mile wide structure. Figure 1 shows the geologic basins located in North Carolina. The individual sub-basins are separated by cross-trending structural highs. The Colon cross structure is located between the Durham and Sanford basins while the Pekin cross structure separated the Sanford and Wadesboro basins.

The Durham sub-basin is the source of our test sites and is a half-graben structure within crystalline rocks of the North Carolina Piedmont region. It is down faulted on the southeastern margin and bounded on the western border by an unconformity. The western border is formed by a minor southeast dipping high-angle normal fault while the eastern border fault is named the Jonesboro fault, which dips northwestward at a high angle. With this faulting, the Durham basin became a shallow, freshwater lake bounded by relatively higher Piedmont crystalline rocks. Erosion of these upland areas filled the basin with coarse granular materials at first then clay and silt sized particles as gradients from the sources diminished.

Sedimentary rocks deposited within the Durham Triassic basin are comprised of interbedded sandstone, siltstone and mudstone with areas of fanglomerates near the eastern fault border. Fining and coarsening trends within beds are usually subtle, although bedding may abruptly change within the fluvial depositional setting. The conglomerate and fanglomerate consist of angular to subangular rock fragments ranging from one inch to over one foot in diameter that were derived from crystalline rocks on the adjacent sides of the graben. These rock fragments are mixed with red-brown sands, silts and clays. Toward the center of the basin the sediments become finer grained with sandstone, siltstone and mudstone with clay particles predominate.

The powder diffraction graph shown in Figure 2 depicts the fine quartzitic and feldspathic grains of fine material found in many of these weak rocks. The matrix of this siltstone is composed of the clay mineral illite.

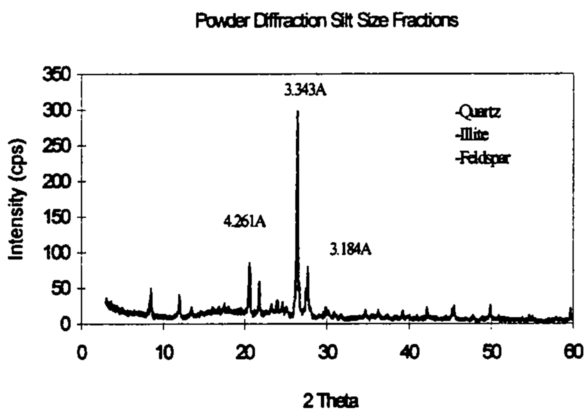


Figure 2. X-ray diffraction of a siltstone from the Deep River Basin.

METHODOLOGY:

Sampling:

Tan, pink, purple and red-brown, medium to very coarse feldspathic sandstone interbedded with red-brown to brown bioturbated siltstone were the subject of the investigation. Mudstone with whitish caliche and nodular limestone were also a part of the study. Engineering values for these sedimentary rocks yield Standard Penetration test refusals, yet are easily drilled with power augers. Materials excavated for roadway construction projects in the Durham sub-basin are removed by means of explosives. After placement in embankments, the siltstone and mudstone rock fragments begin to slake and reduce in size creating voids and slope instability.

To identify the potentially weak rock in the Triassic basin, twenty-five samples of sedimentary rock were gathered from the region. The Department of Transportation drilled pilot holes, using both air and water, to collect the cores from six different test sites known to contain unstable rock material. Each run was geologically characterized for Rock Quality Designation (RQD) and the percent recovered (REC) and then stored to maintain initial water contents.

All cores were subjected to jar slaking for the initial determination of durability. Jar slaking is the process of immersing a piece of rock in water and observing the degradation over a 24-hour period. After the allotted time, a jar-slaking index (I_J) is determined. Values range from 1 to 6 or from total degradation (sample turns to mud) to no change in physical integrity, respectively. Jar-slake testing was performed to qualify the material as very weak ($I_J = 1$ or 2) or semi durable ($I_J = 3$ or 4). Rock that was visually identifiable by jar slaking, such as coarse sandstone, was removed from the study. The softer, soil-like material such as mudrock and overconsolidated clays was also characterized easily. Since they degraded quickly when introduced to moisture ($I_J = 1$), these specimen were not maintained for further analysis. The rock of most concern was the semi-weak or possibly non-durable specimen such as the silty shales (siltstone and mudstone). These were not easily discernible from the other cores in either visual or initial durability testing. These shales, varying in color and texture, posed the biggest problem in classification since initial testing results showed that the rock varied from weak to semi-durable material ($I_J = 2$ through $I_J = 5$). Thus, the sampling techniques and testing methods to best evaluate and identify these particular weak rocks were the focus of the study.

Slake Durability:

For this series, the two-cycle slake durability test was considered the most useful parameter to determine the rock's integrity. Although jar-slake indexing was quicker, it was concluded that the slake durability method was far more qualitative in nature and a better representation of durability. Once the material was determined to be non-durable according to the quick slake method, the two-cycle slake durability index tests were then performed. Slaking was accomplished incorporating ASTM Method D4644-87, *Slake Durability of Shales and Similar Weak Rocks*. Sections of the rock specimen were broken into 10 separate, 40 gram to 60 gram pieces and subjected to a 10-minute water bath.

During the bath, the material is simultaneously abraded and screened through an ASTM No. 10 sieve. The material is removed and dried for 24 hours. The slake durability index (I_{d2}) is the percentage of dry mass retained after two of these wetting and drying cycles. Sediment that remained in the bath after testing was subjected to plasticity analysis.

Plasticity Testing:

All rock, which was identified as semi to non-durable from the jar-slaking process, was subjected to plasticity testing. Laboratory testing for Atterberg Limit indices were performed as described in ASTM Method D418-84 *Liquid Limit, Plastic Limit and Plastic Index of Soils*. Two separate locations were used to obtain plasticity samples: 1. gathered sediment that remained after slake durability testing and 2. pulverized rock that was passed through a standard ASTM No. 40 sieve.

For the purpose of rock identification and the measurement of the percent of fines, standard grain size distribution testing was performed. Hydrometer analysis was required to determine the percent passing the 2-micron grain size (clay boundary). The examination was conducted according to ASTM Method D 422-63, *Particle-Size Analysis of Soils*. From the tests, a determination for the activity of the clay matrix, as defined by Skempton, (1953) was calculated. The activity (A) of a substance is obtained by dividing the plasticity index (PI) with the percent finer than the 2μm–grain size. Activity values above the range of 1.25 are an indicator of active, swelling clays (montmorillonite). Material of this nature is conducive to rapid changes upon contact with water. It was deduced that activity values would not only be an indication of the type of clay but a possible identifier of potentially degradable material.

Strength Testing:

Test cores were approximately 2 inch in diameter (NX drill core size). Material was categorized for durability and subdivided into specific lengths for unconfined compression testing. A ratio of 2:1 (length to diameter) was measured and all specimen were then cut dry with a chop-saw to prevent any increase in moisture. Specimens were sanded smooth in preparation for test evaluations. All specimen adhered to ASTM D4543-85, *Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances*. Since much of the very soft mudstone and some of the coarser sandstone would flake and chip during the preparation, cored sections unable to produce 3 or more specimens were not used in the study. Jar-slaking and slake durability testing was still performed on those sections of cores, however, to analyze the relationship between plasticity and durability. Compression testing was performed in accordance with ASTM Method D2938-86, *Unconfined Compressive Strength of Intact Rock Core Specimen.* During testing, strain measurements were obtained in order to determine Young's modulus.

Cores were also cut and sanded into 1-inch length segments to be used for split-tensile testing. Recoverable cores did not always provide ample 2:1 (length to diameter) ratios for compression testing. However, much of this material was capable of being segmented and sanded into tensile specimens. Split-tensile strengths were obtained using ASTM Method D3967-92, *Splitting Tensile Strength of Intact Rock Core Specimen.* The majority of the rock pieces that were recovered from these failed specimens were incorporated into slake durability testing. This was to provide a better correlation between durability and strength within the sample geologic conditions.

RESULTS:

The majority of the rock from the recovered cores was determined to be a sandstone and siltstone mixture. The rock studied is primarily siltstone in nature with an ASTM classification, for the fine sediment, of low plasticity silt (ML) or clayey silt and sand (CL-ML). The grain size distribution shows little clay content, which greatly contributes to the lack of durability in most shale. Very fine-grained sand particles accounted for a large percentage of the rock. In the cores, softer layers of shale were intermingled between more durable and sometimes, non-durable laminations of sandstone. The weakest material was initially evident from the driller's log. Lower RQD and REC values were the first indication of poor material.

Color was also somewhat of a precursor to durability. Reddish-brown, purple and dark-brown siltstone had a majority of jar-slake indexes in the range of $I_J = 3$ or 4. The non-durable siltstones were more soil-like or dull brown in color. Much of the tan material was composed of fine-grained sand producing a high durability index. Clear, cemented sandstone had a medium durability but was difficult to shape into strength testing specimen due to their coarseness. Finally, the conglomerates tested were all considered durable with relatively no change when exposed to moisture.

Once the weak rock was identified, strength tests were administered. Results displayed in Figure 3 show that the correlation between the unconfined compressive strength and the slake durability index is difficult to predict. Compressive strengths for the non-durable material range from 1450 psi for claystone to 4350 psi for stronger siltstone with average slake durability indices from 59% to 83%, respectively. Compressive strength values as low 870 psi were obtained in a few samples; however most material of this nature were difficult to shape into reliable specimens.

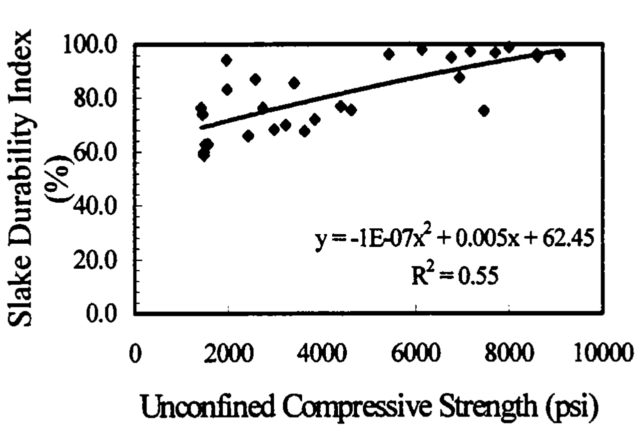
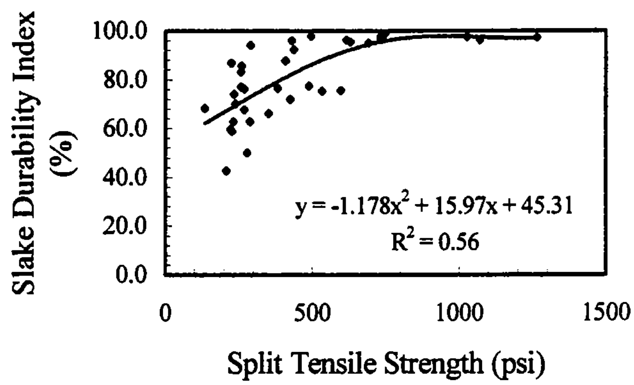


Figure 3. Unconfined compressive strength (UCS) results as compared to slake durability indices (I_{d2}).

On the graph, the upper end values, having UCS results from 5800 psi to 9425 psi, are the sandy siltstone specimen. These were more durable than the other cores and used for a comparison against the softer materials. Although the trend shows an upward increase in strength with the increase of durability, an R^2 value of 0.55 for this trendline equation is not a direct enough correlation to show a good relationship.

The determination of the slake durability index (I_{d2}) from the split-tensile strength is almost as variable as that of the compression results. Split-tensile data can be seen Figure 4. The rock ranges in strength from a low of 200 psi up to 1300 psi with the majority of the results in the range of 290 to 580 psi. The trend again shows an increasing rise in strength with increasing durability; however, the R^2 correlation value of 0.56 is only slightly better than with the compression results. This may be due to larger amount of specimen available for testing. A vast number of specimens with varying durability broke at an approximate



strength of 290 psi.

Figure 4. Split-tensile strength results as compared to slake durability indices (I_{d2}).

From ASTM standard methods, the soil classification of the sediment appears to be a low plasticity silt (ML) with a few samples ranging into silty clays or clayey silts and sands (CL-ML). This naturally leads to lower plasticity values and thus lesser clay contents as evidenced in the grain size distributions. The percent of fines passing the 2-micron grain size range from 10.7% for the very low plastic rock up to 26.6% for the higher liquid limit specimen. To check larger grain sizes, the 5-micron grain percentage was examined and varied accordingly from 13% to 41%. Much of the sample rock contains fine to very fine sand particles. High silt content dominates these Triassic Basin materials. Clay content thus appears to be relatively meager for troublesome shale. These rock samples are very silty with clay percentages averaging below

20%. Although it has been determined that clay contents as low as 15% may cause deterioration in rock, this material poses problems due to its silt-ized particles.

Mudstones, which could not be tested for strength, were still subjected to slake durability and plasticity tests. I_{D2} values between 0% and 40% were obtained from the more plastic material. Jar slake testing on these samples produced rapid results with indexes I_J = 1 or 2 within four hours. These were not part of the study, however, since they could not be correlated to strength values.

All material tested indicated a low plasticity index range from 1.5% to 7.5%. In a comparison of the plasticity index (PI) with slake durability, scattered results of the average PI were given a nonlinear correlation with slaking durability effects. These scattered results can be seen in Figure 5.

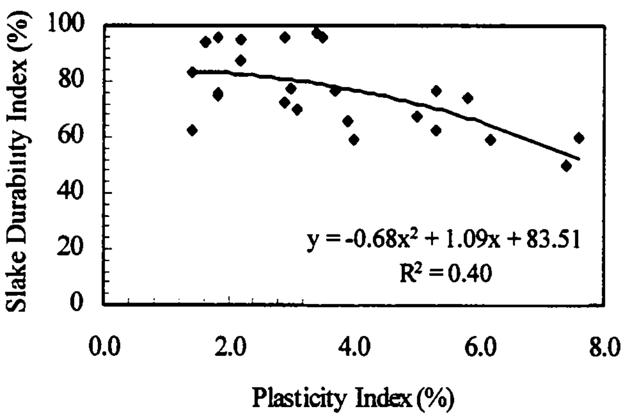


Figure 5. Plasticity Index (PI) values versus slake durability indices (I_{d2}).

Although the R^2 value for the trendline is less 50% it is evident that there is a loss in durability with the increase of plasticity. It should be noted that none of the fines from these samples were in the high plastic range, usually associated with clay-shale or mudshale. Due to the varying clay contents and the amount of sand and silt particles found in these specimen, it is difficult to predict the slake index directly from the Atterberg Limits. Samples that were tested for durability and plasticity alone are not shown in this graph. Those samples were considered easily identifiable by jar-slaking analysis.

The percentage of clay fines (<0.002) for these specimen averaged 17.5 percent by weight. From this 2µm–grain size distribution and the plasticity index values, the activity of the clays were determined for each specimen. It was determined that clay activity should be calculated without separating the clays from the rest of the rock sediment and performing plasticity tests separate from the entire specimen. In this manner, rocks with greater percentages of clay fines would exhibit larger activity values. According to Skempton, 1951, the range for normal clays should range between 1.00 and 1.25 with highly active clays exceeding these limits. For the rock used in this study, much lower results were produced. The activity of these samples varied from 0.1 to 0.5, well below the standard for any active clay. It should be noted that with low percentages of clay in these rocks, collecting an adequate amount of clay fines for plasticity testing was not possible. The test results reflect the values for all the material in a sample and not just the separated clays. This reflected the idea for rapid identification of non-durable material.

In Figure 6, it is evident that the correlation of durability with activity is very similar to that with plasticity. Again the tendency is to have decreasing slake durability with increasing activity; however, the relationship between these two variables is not indisputable. The average activity value was 0.23 while the average I_{d2} for this rock was 68.5 %.

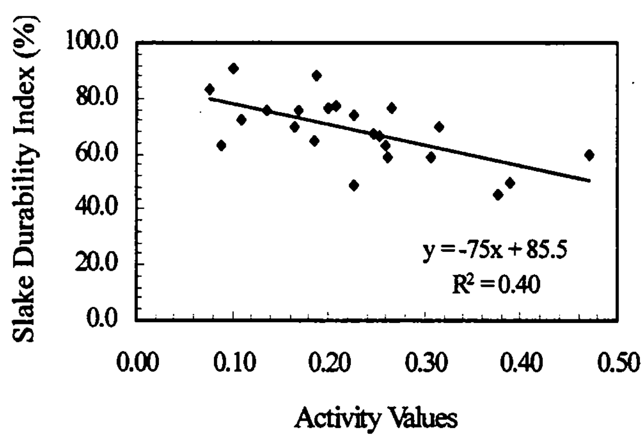


Figure 6. Clay activity values plotted against slake durability indices (I_{d2}).

With approximately 20% of the rock being comprised of clay, these specimens provided a value

20% of the range for normally active clays. In this manner, activity numbers in the range of 0.15 to 0.40 can be considered normally active clays while those samples with activities in the lower region would be determined to be inactive.

DISCUSSION:

The overall classification of the non-durable to semi-durable, Triassic rock samples gathered from the Durham Sub-basin of North Carolina appears to be argillaceous sediment formed into mudstone and siltstone. Although slake durability indices of a number of the samples is below 60%, excessive clay content did not appear to be the factor affecting the degradation. The sedimentary material naturally degraded with the introduction of moisture. However, deterioration of the rock does not appear to be due to the activity of the clays. In comparison to Gambles (1971) study of the degradation of soft rock, these shales are not necessarily plastic but can still cause problems in construction. It was evident from the graphs that a direct correlation between slaking, plasticity, clay content or geologic considerations (grain size distribution) can not be performed successfully between two independent variables. More than any two factors simultaneously affect the degradation of this type of rock. To prove this premise, UCS results were plotted against normalized slake durability values in Figure 7. I_{d2} results were normalized by the percent passing the 2μm–grain size.

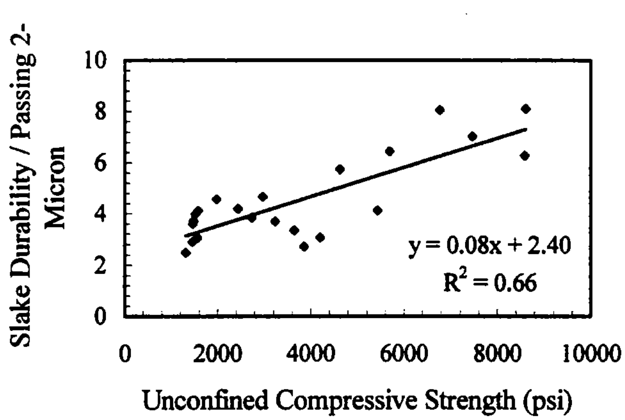


Figure 7. UCS plotted with I_{d2} normalized by percent clay fines.

Statistically, the correlation increases by more than 25% when an additional parameter was placed into the equation. To further relate the parameters, slake durability and passing 2μ, each was set as an independent variable and graphed against the dependent variable, UCS (Parish and Borden, 2001). The R^2 value for that trend improved to 0.69. This was also an increase in the correlation coefficient between results comparing any other two variables.

Using three separate variables resulted in a marked improvement in the prediction capability when compared to bivariate analysis. It was apparent that even greater statistical results could be obtained by increasing the amount of independent parameters. However, additional testing would not be conducive to better field engineering practice when determining the durability of a material.

It appears that no one single factor affects the loss of strength in this rock. The best relationship to determine the strength of these specimens exists between the durability and the 2um-grain size of the sedimentary material. With the lack of clay content and the fact that very fine sands and silt particles control plasticity, grain size is a large contributor to the durability and strength. In slaking effects of this nature, pore-air compression must dominate the degradation of the rock, not volumetric changes as seen in other, more clay-laden shales (mudrock).

The problem in determining the durability classification of this siltstone is the variability of the strength of this substance. From the regression trends, it appears that there is a region where the rock may or may not be considered durable. The region from a slake durability index of 50% to 85% has been suggested the rock be considered semi-durable (Gamble, 1971). In this research, some rock had compressive strengths as low as 870 psi but still possessed semi-durable qualities.

For the rock with recovery values less than 65%, it was apparent that durability questions were not arguable when the cores were obtained with care. Extremely soft shale was immediately identified from its lack of physical integrity. Correspondingly, coarse sandstone with high RQD and recovery values was also easily categorized by visible appearance.

Problems with this type of testing exist primarily in the collection of specimen. Weak rock is usually difficult to collect in samples adequate for unconfined compression testing. Although most samples that were suitable for strength studies are of a durable nature, softer samples of questionable durability could be prepared but only under extreme diligence.

CONCLUSIONS:

Geologically, the sedimentary rocks from the basin consist of red-brown sands, silts and clays. The fines are comprised of low plasticity silt or silty clays and fine sands. Based on the results from slake durability, plasticity, and strength testing of these Triassic Basin rocks the following conclusions can be drawn:

1. The lower strength values for weak rock is dictated by the lack of ability to collect adequate samples. Unconfined compressive strengths ranged from 1450 psi to 9425 psi. The upper boundary values near 8700 psi were sandier and more durable in nature. A large variation in the durability of the rock was seen in the lower strength range from 1450 psi to 4200 psi. Approximately 1450 psi is the strength boundary at which specimens can be sufficiently collected.

2. A direct bivariate correlation between any two tests parameters is not conclusive enough to formulate a useful design equation. Although the trends do show increases in UCS with increases in I_{d2} and with decreases in plasticity, the correlation coefficients range near 0.50 for all of the two-variable trendlines. By normalizing the slaking factor, an R^2 value of 0.66 was realized when comparing durability to UCS.

3. Activity results, using standard methods, revealed that active clay fines (A>1.25) do not dominate these samples. An activity between 0.11 and 0.27 would be representative of this material. This is in a very low range for the non-durable shale but the test was performed on the entire sample and not just separated clay fines. Thus, the slaking of these softer rock may be attributed to pore-air compression (a phenomena in the slaking mechanism of non-swelling clays) rather than the reaction of responsive fines.

In summary, determining the durability of Triassic Basin siltstone is difficult at best. Introduction of moisture to these materials will eventually begin the process of degradation. Thus, drilling methods are extremely important when gathering tests samples. Finally, the amount of deterioration involved for a total loss of strength is dependent on more than one factor and can not be concluded directly from any one series of tests.

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EVALUATION OF CARBONATE AGGREGATES FOR BITUMINOUS OVERLAYS IN INDIANA

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ABSTRACT

Indiana dolomite aggregates are widely used for bituminous wearing courses owing to abundance and greater frictional (skid) resistance. Surprisingly, recent research showed some limestones had higher frictional resistance than some dolomites.

Engineering tests including acid insoluble residue, elemental Mg content, freeze and thaw, sulfate soundness, Los Angels abrasion, absorption and specific gravity and British Pendulum testing were conducted on limestones and dolomites. Correlation and multiple regression analyses related Polished Values or PV (British Pendulum test) to these parameters. Results indicate that heterogeneity in mineral hardness and composition, creating an uneven texture during polishing, improves frictional properties.

Regarding geologic strata, for dolomites, impure Kokomo Member had the highest PV, whereas pure Huntington Dolomite showed the lowest. For limestones, impure Mississinewa Member showed the highest PV, whereas pure Brassfield Limestone showed the lowest.

PV of dolomites correlated well with absorption, specific gravity, sulfate soundness and elemental Mg content. However, PV of the limestone aggregates correlated well with acid insoluble residue.

Using multiple linear regression analysis, empirical equations were developed at the 5% significance level:

Dolomite aggregates: $PV = 46.0331 + 1.5645 \times (\text{Absorption}) - 1.7651 \times (\text{Mg content})$

Limestone aggregates: $PV = 21.7006 + 0.6029 \times (\text{Total insoluble residue})$

Carbonate aggregates combined:

$$PV = 19.5464 + 1.7129 \times (\text{Absorption}) - 0.0164 \times (\text{Mg content})^2 + 0.5189 \times (\text{Total insoluble residue})$$

As a starting point for further research using the British Polishing Wheel and Pendulum test, the following is proposed for bituminous surfaces: minimum PV=24 or less is poor resistance, PV=25-30, marginal, and PV=31 or more, good resistance.

Results will be used to develop procedures to identify aggregate sources for bituminous surfaces in Indiana.

INTRODUCTION

Bituminous courses are placed over concrete pavements after the concrete has experienced years of wear from highway traffic. The coarse aggregate in the bituminous overlay must supply the primary roughness to yield needed resistance for braking. Although most coarse aggregate types in new bituminous pavements initially provide high friction values, polishing of the coarse aggregate to an equilibrium level eventually occurs. The extent of polishing an aggregate undergoes is a function of rock type and gradation, as well as its physical and chemical properties.

For high vehicular traffic roads in Indiana, equal amounts of dolomite and blast furnace slag are used as the coarse aggregate in bituminous surfaces (INDOT, 1999). A minimum value of 10.3 % elemental Mg (78.1 % dolomite) is required for aggregates to qualify as an acceptable dolomite material. As high purity dolomites occur in only certain geologic formations in Indiana and therefore, in only certain locations, high purity dolomite aggregate must be shipped long distances to produce the bituminous surfaces for paving projects on Interstate highways. These greater transportation distances greatly increase the cost of construction.

The results of the study by Bruner, Choi and West (1995) suggest that some limestones may provide equal or better frictional resistance than do some dolomites that qualify because of their elemental Mg content. The frictional resistance of the limestone and dolomite aggregates is controlled by their physical and chemical properties.

The most important, textural parameters affecting frictional properties of aggregates are grain size and shape (West, 1995; West et al., 1970) and grain size, hardness and durability (Dierstien and LaCroix, 1984). Shupe (1960) noted that particle-by-particle type of wear in rocks such as sandstone consisting of hard quartz and weak calcite matrix was related to high friction values. In a study by Shakoor and West (1979), grain size and particle shape were found to affect polish and thus friction properties; however, they contributed to a lesser degree than did composition.

Therefore, in this follow-up study, major consideration is given to the compositional properties of dolomite and limestone aggregates to find the critical factors that provide some limestones with a better performance in skid resistance. Review of the study by Russell (1972), provides the reason why magnesium content was selected as the criterion for acceptance of dolomite aggregates in Indiana. A high percentage of MgO corresponds directly with a high dolomite concentration. Russell considered dolomite aggregate to consist of >50% dolomite mineral or 10.9% MgO. However, in the dolomite study by Bruner, Choi and West (1995), the higher elemental Mg content of dolomite correlates with smaller PV (Polished Value) and higher WI (Wear Index). This means that dolomites with a high elemental Mg content polish easier under traffic conditions.

The amount of dolomite can be calculated from the percent of elemental Mg or from the percent of MgO in the following way.

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

$\% \ Dolomite = \dfrac{\%\ Elemental\ Mg}{0.1318}$

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

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

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

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

A comparison of the representation of Mg and MgO in dolomite is illustrated in Figure 1. As shown in this figure, the acceptable dolomite amount for Illinois DOT use ranges from 50 to 100% dolomite (10.93 to 21.86% MgO), whereas for INDOT the acceptable dolomite amounts ranges from 78.1 to 100% dolomite (10.3 to 13.19% Mg).

Regarding compositional properties of carbonate aggregates, an increase in magnesium (impure limestone) content for Indiana limestone sources showed an increase in friction properties (Shupe, 1958, 1960).

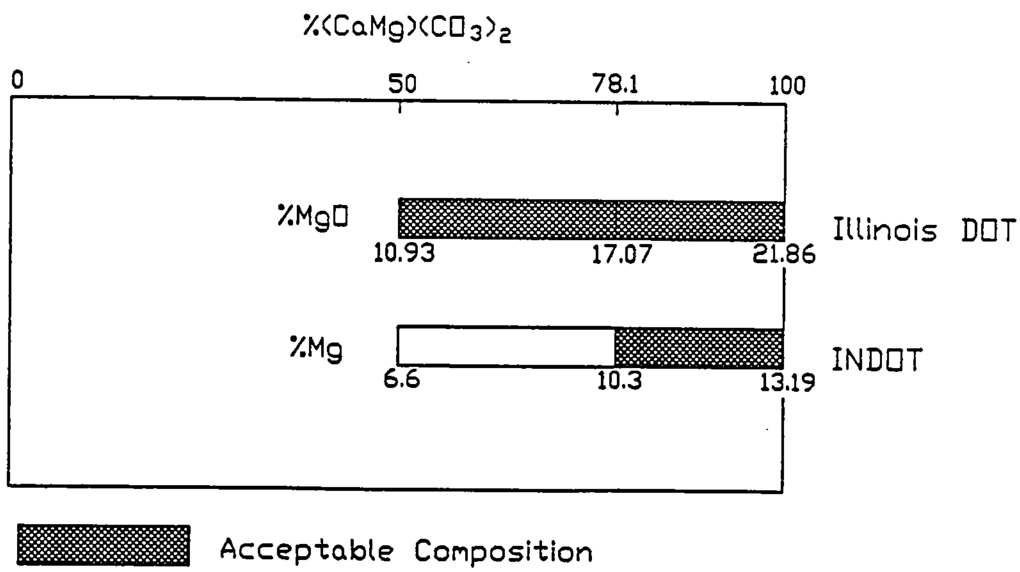

Figure 1. Percent Dolomite in Carbonate Aggregates related to Percent MgO and Mg Content for Bituminous Surface Courses

A positive correlation was found between acid-insoluble residues >75 microns in size and friction values for the dolomite aggregates (Russell, 1972). Also, A positive correlation was found between friction value for dolomite aggregates and the insoluble-residue retained on the No. 200 sieve, as well as with the ratio between the No. 30 and No. 200 sieves (Cummings, 1976). At greater than 25 percent, the sand size acid-insoluble residue generally accounts for higher friction values (Dierstein and LaCroix, 1984). Strong relationships were found between total insoluble residue and friction coefficient (Sherwood and Mahone, 1970). Dahir and Mullen (1971) observed that skid resistance increases with an increase in the amount of insoluble residue; sand size residue probably is more important than total residue and the presence of clay generally contributes to lower frictional resistance. They also mentioned that skid resistance was higher for aggregates having mixed composition of hard and soft minerals than for aggregates consisting mostly of minerals with the same hardness. Dolomite aggregates containing greater amounts of insoluble residue less than #200 size show a higher Polished Value (Bruner, Choi and West, 1995).

The common physical tests considered to have a major influence on aggregate performance are absorption, soundness, abrasion and specific gravity (Senior and Rogers, 1991). Aggregates with high absorption generally have lower freeze-thaw resistance. Specific gravity is related to composition and absorption. Increased compositional variation within an aggregate source yields increased differential polishing and a higher PV. In the Bruner, Choi and West study (1995), for dolomite aggregates, Los Angeles abrasion did not correlate with either WI or PV. However, dolomites with higher absorptions showed lower WI and higher PV. Also higher specific gravities correlated with lower PV and higher WI. The dolomites with a higher sulfate soundness loss also showed a higher PV and lower WI.

RESEARCH PROCEDURES

This study was a continuation of previous work by Bruner, Choi and West, 1995, which focused primarily on dolomite aggregates (20 sources). In the current study, 21 limestone sources were added to the analysis. Data from both studies were evaluated to provide an overall conclusion.

Following aggregate collection, megascopic and microscopic observations on the aggregate samples were accomplished. Twenty-four additional rock thin sections were examined adding to those from the first study. Laboratory tests including acid insoluble residue (ASTM, D3042), size distribution of the acid insoluble residue, elemental Mg content (ASTM, C602), Los Angels abrasion (ASTM, C131), sulfate soundness (ASTM, C88), freeze-thaw loss in water and brine solution (AASHTO, T103 Procedure A), absorption and specific gravity (ASTM, C127) were conducted at the INDOT, Division of Materials and Tests, and at Purdue University.

Aggregate coupons of limestones were made for the British Wheel test and British Pendulum test (ASTM, D3319, E303). The largest portion of No.11 INDOT gradation, 12.5mm to 9.5mm in size, was used. To construct the coupons the aggregates are embedded in epoxy resin and a hardener component added. These materials were different from those used in the 1995 study (Bruner, Choi and West, 1995). Coupons were polished using the British Wheel machine and friction resistances were measured with the British Pendulum tester after zero hours (Initial Friction Value, IFV), 1 hour, 3 hour, 6 hour and 10 hour (Polish Value, PV or BPN_{10}) during the polishing process.

Finally, analysis of all data from the current and the 1995 study were combined and a statistical evaluation performed. Based on this work, the PV of dolomite and limestone for bituminous surface courses required for heavy traffic highways can be recommended

RESULTS

Results of the current research on limestone aggregates along with those on dolomite aggregates by Bruner, Choi and West (1995) were analyzed collectively.

BPN (IFV and PV), physical properties, elemental Mg content and insoluble residue for limestone and dolomite samples are listed in Table 1A and 1B. As shown in these tables, the average friction resistance for dolomites (PV=28.50) is higher than that for limestones (PV=24.77).

<u>Aggregate Lithology and Frictional Resistance</u>

The BPN of carbonate aggregates with regard to aggregate lithology for Indiana sources was evaluated. Geologic details on stratigraphy for Indiana quarries are provided by Carr et al., 1971, and Shaver and Alt, 1986.

As shown in Table 2, for dolomite aggregates, the Kokomo Member (Upper Silurian) shows the highest PV (average 33.3) and the Huntington Dolomite (Upper Silurian) shows the lowest PV (average 25.6). For the limestone aggregates, the Mississinewa Member (Upper Silurian) shows the highest PV (28.1) and the Brassfield Limestone (Lower Silurian) shows the lowest PV (20.9).

To determine the factors affecting the frictional properties of dolomite and limestone aggregates, elemental magnesium content and acid insoluble residue values for various geological formations were determined. This is appropriate as INDOT currently uses an elemental Mg content as the acceptance criteria for dolomite sources, with a minimum level set at 10.3%. Also some impure dolomites and impure limestones are known to provide quality aggregates.

As presented in Table 2, regarding elemental Mg content of dolomite aggregates, Louisville Limestone (Middle Silurian) shows the highest elemental Mg content (13.1%) and the Kokomo Member with the highest PV shows the lowest elemental Mg content (10.9%). For limestone aggregates, Salamonie Dolomite (Lower Silurian) shows the highest elemental Mg content (8.29%) and Brassfield Limestone with the lowest PV shows the lowest elemental Mg content (0.6%).

Table 1. BPN and Physical Properties, Mg contents and Insoluble residues

A. Limestone Aggregates

Id. No.	IFV	PV	WI	Ab.	SpG	LA Ab.	F-T	Mg (%)	Acid(T)	P#200	M#200
L-001	38.60	28.14	10.46	2.170	2.59	28.50		5.40	12.72	2.40	10.32
L-002	40.20	24.72	15.48	1.780	2.59	27.00	22.25	0.70	5.76	3.64	2.12
L-003	44.38	22.69	21.69	2.000	2.60	21.37	7.98	9.50	4.09	2.04	2.05
L-004	43.14	23.49	19.65	1.030	2.63	32.09		3.60	3.37	0.24	3.13
L-005	39.48	21.67	17.81	0.930	2.65	28.60		2.20	4.96	1.53	3.43
L-006	43.00	20.87	22.13	1.350	2.63	37.23	10.92	0.60	1.97	0.15	1.82
L-007	41.13	25.50	15.63	1.260	2.62		16.62	0.90	4.54	2.36	2.18
L-008	41.76	24.97	16.79	1.150	2.65	26.49	27.46	2.50	6.00	0.70	5.30
L-009	42.26	27.40	14.86	2.065	2.64	25.94	15.43	9.05	6.38	0.30	6.08
L-010	45.30	25.39	19.91	2.330	2.62	23.03	19.47	10.00	6.17	1.80	4.37
L-011	43.68	25.39	18.29	0.780	2.70	32.24	7.46	4.60	5.14	0.86	4.28
L-012	46.69	27.51	19.18	1.005	2.67	25.73		2.30	6.04	0.60	5.44
L-013	48.87	29.24	19.63	1.050	2.64	27.72		1.90	11.21	4.83	6.38
L-014	44.10	25.00	19.10	1.530	2.63	28.49	8.06	1.60	4.13	1.43	2.70
L-015	43.21	27.12	16.09	0.880	2.64	22.40		1.10	4.41	1.86	2.55
L-016	41.02	24.72	16.30					1.10	4.59	1.16	3.43
L-017	47.85	25.73	22.12	2.470	2.52	38.17	3.01	0.50	1.41	0.14	1.27
L-018	44.69	22.55	22.14	1.970	2.59	34.76		0.70	2.28	0.12	2.16
L-019	42.37	23.95	18.42	1.320	2.63	24.79	11.30	0.70	4.01	0.64	3.37
L-020	40.10	22.05	18.05	1.150	2.69	24.39		1.20	3.61	0.47	3.14
L-021	42.50	22.01	20.49	1.690	2.64	26.71	5.38	1.00	3.60	0.16	3.44
Average	43.06	24.77	18.30	1.496	2.63	28.19	12.95	2.91	5.07	1.31	3.76

B. Dolomite Aggregates

Id. No.	IFV	PV	WI	Ab.	SpG	LA Ab.	F-T	Mg (%)	Acid(T)	P#200	M#200
D-001	40.00	24.90	15.10	0.79	2.73	25.53	0.36	12.40	1.70	1.23	0.48
D-002	43.30	25.00	18.30	2.39	2.60	28.59		12.90	0.71	0.02	0.69
D-003	41.80	27.30	14.50	2.64	2.61	27.05	9.57	11.30	3.13	2.07	1.06
D-004	46.90	35.30	11.60	6.25	2.39	29.36	9.86	11.30	2.11	0.11	2.00
D-005	44.00	27.70	16.30	2.02	2.62	29.19	7.30	11.60	9.82	3.55	6.28
D-006	42.70	26.80	15.90	1.56	2.63	26.62	2.24	12.40	5.29	1.79	3.50
D-007	44.00	28.80	15.20	3.05	2.58	32.43	4.79	12.90	1.34	0.21	1.13
D-008	44.60	28.70	15.90	2.38	2.59	31.73	0.76	13.10	0.22	0.03	0.20
D-009	44.80	31.00	13.80	1.21	2.67	22.47	5.59	10.70	7.42	4.40	3.02
D-010	45.90	31.30	14.60	3.76	2.49	36.98	3.29	12.60	4.16	1.81	2.35
D-011	46.20	30.00	16.20	2.02	2.63	24.75	5.99	12.20	6.31	2.71	3.59
D-012	47.60	31.30	16.30	2.74	2.59	29.50	2.24	11.90	7.65	3.39	4.26
D-013	43.50	32.70	10.80					10.10	5.56	0.78	4.78
D-014	46.70	32.00	14.70	4.00	2.48	30.28	13.18	11.20	5.27	0.71	4.55
D-015	40.50	24.10	16.40	1.03	2.67	29.76	2.56	12.80	0.50	0.27	0.22
D-016	40.10	23.60	16.50	1.86	2.61	32.62	0.55	12.90	0.08	0.02	0.06
D-017	41.30	24.90	16.40	1.00	2.72	25.69	0.31	12.90	0.35	0.03	0.32
D-018	43.60	28.40	15.20	2.48	2.62	27.25	3.62		2.15	0.94	1.21
D-019	41.60	28.00	13.60	1.59	2.65	30.75	6.45		6.66	4.45	2.21
D-020	41.90	28.10	13.80	2.95	2.59	28.54	5.33		5.90	3.14	2.76
Average	43.55	28.50	15.06	2.41	2.60	28.90	4.67	12.07	3.82	1.58	2.23

Table 2. BPN, elemental Mg content and insoluble residue of the carbonate aggregates

	Formation	Dolomite				Limestone				Remark
		IFV	PV	Mg	Acid Ins.	IFV	PV	Mg	Acid Ins.	
Wabash Fm	Kokomo Mbr.	43.5-46.9 (45.7)	32.0-35.3 (33.3)	10.1-11.3 (10.9)	2.11–5.56 (4.31)	-	-	-	-	D004 D013 D014
Wabash Fm	Mississinewa Mbr.	-	-	-	-	38.6	28.1	5.40	12.72	L001
Wabash Fm	Huntington Dol.	40.0-44.0 (41.5)	24.1-27.7 (25.4)	11.6-12.9 (12.4)	0.35-9.82 (3.09)	-	-	-	-	D001 D005 D015 D017
	Salamonie Dolomite	43.0-47.6 (45.7)	25.0-31.3 (29.4)	11.9-12.9 (12.4)	0.71-7.65 (4.71)	42.3-45.3 (43.9)	22.7-27.4 (25.2)	4.6-10.0 (8.29)	4.09-6.38 (5.45)	L003 L009 L010 L011 D002 D010 D011 D012
	Brassfield Limestone	-	-	-	-	43.0	20.9	0.6	1.97	L006
	Jeffersonville Limestone	-	-	-	-	41.1-48.9 (45.0)	25.5-29.2 (27.4)	0.9-1.9 (1.4)	4.54-11.21 (7.88)	L007 L013
	Louisville Limestone	44.6	28.7	13.1	0.22	41.8-46.7 (44.2)	25.0-27.5 (26.2)	2.3-2.5 (2.4)	6.0-6.04 (6.02)	L008 L012 D008
	Haney Limestone	-	-	-	-	44.1	25.0	1.6	4.13	L014
	Ste. Genevieve Limestone	-	-	-	-	41.0-43.2 (42.1)	24.7-27.1 (25.9)	1.1 (1.1)	4.41-4.59 (4.50)	L015 L016
	Beaver Bend Limestone	-	-	-	-	40.1-47.9 (43.5)	22.0-25.7 (23.3)	0.5-1.2 (0.82)	1.41-4.01 (2.98)	L017 L018 L019 L020 L021
	Pleasant Mills Formation	41.9-44.8 (43.6)	28.1-31.0 (29.3)	10.7-12.9 (11.8)	1.34-7.42 (4.89)	-	-	-	-	D007 D009 D020

It is important to consider the following: For dolomite aggregates, the lower the elemental Mg content, the more impure the dolomite. Therefore, the lower the elemental Mg content, the higher is the PV. For limestone aggregates, the higher the elemental Mg content, the more impure the limestone. Therefore, the higher the elemental Mg content, the higher is the PV.

Considering acid insoluble residue, for dolomite aggregates, the Pleasant Mills Formation (Middle Silurian) has the highest acid insoluble residue (4.89%) and Louisville Limestone (Middle Silurian) has the lowest insoluble residue (0.22%). For limestone aggregates, the Mississinewa Member (Upper Silurian) has the highest PV but also shows the highest insoluble residue (12.72%). Brassfield Limestone (Lower Silurian) has the lowest PV and also has the lowest insoluble residue (1.97%).

It is also important to consider: For dolomite and limestone aggregates, the higher the acid insoluble residue (fine quartz and clay), the more impure are the dolomites and limestones. Therefore, the higher the acid insoluble residue, the higher is the PV.

REGRESSION ANALYSIS

Dolomite Aggregates

From the laboratory variables for dolomite aggregates, absorption, elemental Mg content and acid insoluble residue less than #200 sieve were selected as independent variables. Using multiple linear-regression, the relationship between PV and these variables was developed:

$$PV\ (1) = 38.0232 + 1.5568 \times (\text{Absorption}) - 1.1750 \times (\text{Mg content}) + 0.3950 \times (\text{Insoluble residue} < \#200), \quad (\text{R-square} = 0.779,\ P < 21.19\%)$$

Next, elemental Mg content was correlated with percent of acid insoluble residue less than #200. The above equation after removal of acid insoluble residue is as follows:

$$PV\ (2) = 46.0331 + 1.5645 \times (\text{Absorption}) - 1.7651 \times (\text{Mg content}), \quad (\text{R-square} = 0.747,\ P < 1.75\%)$$

Limestone Aggregates

Based on laboratory testing of limestone aggregates, absorption, elemental Mg content and total insoluble residue become the significant independent variables. Using multiple linear-regression, the relationship between PV and these variables is:

$$PV\ (3) = 21.3374 + 0.2733 \times (\text{Absorption}) - 0.0376 \times (\text{Mg content}) + 0.6161 \times (\text{Total insoluble residue}), \quad (\text{R-square} = 0.499,\ P < 81.35\%)$$

Next, only total insoluble residue was considered as an independent variable because absorption and Mg content do not correlate as well with PV. The relationship between PV and total insoluble residue is:

$$PV\ (4) = 21.7006 + 0.6029 \times (\text{Total insoluble residue}), \quad (\text{R-square} = 0.495,\ P < 0.05\%)$$

Carbonate Aggregates (dolomite and limestone)

IFV and PV values and laboratory data for dolomite and limestone were analyzed collectively to predict the performance of carbonate aggregates in frictional resistance. Absorption, elemental Mg content and total insoluble residue become the significant independent variables based on stepwise regression. The relationship between PV and these variables is:

$$PV\ (5) = 19.47 + 1.721 \times (\text{Absorption}) - 0.2105 \times (\text{Mg content}) + 0.4786 \times (\text{Total insoluble residue}), \quad (\text{R-square} = 0.675,\ P < 0.61\%)$$

However, based on the distribution of elemental Mg content in carbonate aggregates, PV is more highly correlated with elemental Mg content using a 2nd degree polynomial equation than by the linear equation above. The polynomial equation is:

$$PV\,(6) = 19.5464 + 1.7129 \times (\text{Absorption}) - 0.0164 \times (\text{Mg content})^2 + 0.5189 \times (\text{Total insoluble residue}),\ (\text{R-square} = 0.683,\ P < 0.39\%)$$

CONCLUSIONS

For carbonate aggregates, the difference in mineral hardness within an aggregate piece has a significant effect on friction resistance. Uneven texture after polishing yields a high PV. Materials with a contrast in hardness are: quartz vs. calcite, calcite vs. insoluble residue materials, dolomite vs. calcite, impure dolomite and impure limestone.

Certain geological formations consist of impure carbonate materials. The Kokomo Member and the Mississinewa Member are impure carbonates with higher friction values. The Huntington Dolomite and Brassfield Limestone with their high purities showed lower friction values.

The factors greatly affecting PV for dolomite aggregate performance are IFV, absorption, specific gravity, sodium sulfate loss, elemental Mg content and percentage of insoluble residue, minus #200 sieve size. The most influential independent variables for dolomite are absorption and elemental Mg content.

The important factor affecting PV for limestone aggregates are total insoluble residue and percent insoluble residue, minus #200 sieve size. In all, the most influential independent variable is the total insoluble residue content.

Considering dolomite and limestone aggregates collectively, the most important variables are absorption, elemental Mg content and total insoluble residue.

Using multiple linear regression analysis, empirical equations were developed at the 5% significant level as described previously.

Higher elemental Mg values for dolomite aggregates indicate a high purity of dolomite. Such aggregates experience a lower PV. In the INDOT specifications, a minimum 10.3% elemental Mg content is required for carbonate aggregates used for surface courses with intermediate traffic requirements. Based on current results, dolomite with less than 10.3% elemental Mg should be considered as potential aggregate sources if other requirements such as absorption and soundness loss are met. Impure limestone, containing clay, quartz or dolomite should also be considered if they meet those same requirements.

As a starting point for further research using the British Polishing Wheel and Pendulum test the following is proposed. However, additional testing using new Polishing Wheel and Pendulum test equipment should be conducted on Indiana aggregates to verify these results.

Minimum Polish Value	Frictional Resistance of Bituminous Surface
24 or less	Poor
25 to 30	Marginal
31 or more	Good

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FULL-SCALE DEEP FOUNDATION LOAD TESTING IN SANTA FE FORMATION MATERIALS, I25/I40 SYSTEM INTERCHANGE (THE BIG I), ALBUQUERQUE, NEW MEXICO

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Abstract
The I25/I40 System Interchange Big I Project in Albuquerque (Bernalillo County), New Mexico, will extend along I25 north from Dr. Martin Luther King, Jr. Avenue to Comanche Road and along I40 from Sixth Street east to Carlisle Boulevard. All of the existing bridges in the central interchange are to be replaced by new structures. A total of 64 bridges and 86 retaining walls, along with new roadways, are to be designed and built at a cost of approximately $250,000,000. The purpose of the Big I Project is to reduce congestion on the two major interstate highways in New Mexico, north/south I25 and east/west I40. The reconstructed Big I will provide enough capacity to keep pace with traffic growth through the year 2020.

Full-scale load testing of two reinforced concrete drilled shaft foundations was performed at the Big I site during the design phase of the new Big-I Interchange. The test shafts were situated in the Tertiary age Santa Fe Formation soils, with the exception of a limited surficial thickness of man-made fill material above the Santa Fe Formation at one test location. The Santa Fe Formation in the Big I area consists of partially indurated, interbedded alluvial fan deposits and interbedded sands, gravels, cobbles and clays. The test shafts were sited in Santa Fe Formation soils having relative densities in the range of about 35 to 85 percent in order to characterize the relatively high shear strength of these deposits.

The State Geotechincal Section of the New Mexico State Highway & Transportation Department had developed certain objectives for the load test project through the design contract. The primary objectives of the drilled shaft load test project included:

- Develop site-specific geotechnical design parameters through back-calculation of estimated values of the ratio of horizontal to vertical stress (K) and the adhesion factor (α) from load test results.
- Utilize the resulting site-specific design parameters in the final design of deep foundations for the Big I project structures, resulting in lower factors of safety and economical foundations.
- Evaluate constructability of production drilled shafts within the Santa Fe Formation deposits

Load testing was performed using two hydraulic Osterberg load cells (O-cells) installed within each of the two test shafts, located at about two-thirds of the shaft depth and at the shaft base. Load test stages included expansion of the lower load cell to assess shaft end bearing (tip resistance), and expansion of the upper cell to assess side resistance, with no engagement of tip resistance. The test shafts were instrumented with vibrating wire strain gages, linear variable displacement transducers (LVDTs) and dial gages such that hydraulic load cell expansion and shaft displacement were measured and shaft elastic compression and load transfer in side resistance along the shaft length could be evaluated. The data derived from the load tests was utilized to back-calculate actual effective values of K and α for the geologic strata which comprise the Santa Fe Formation encountered at the test shaft sites. Design parameter values were then developed for the remaining geologic strata at the Big I project through comparisons of estimated soil parameters for the various strata.

Introduction
This paper presents a description and results of load testing of two full-scale drilled shafts conducted on behalf of the New Mexico State Highway and Transportation Department (NMSH&TD) at the site of the I25/I40 system interchange (the Big I) in Albuquerque, New Mexico during 1998 and 1999. Test Shaft 1-A was located at the intersection of I40 and I25, south of the eastbound lanes of I40, between the I25 northbound to I40 ramps and the I25 mainline freeway. Test Shaft L-1 was located west of the I25 mainline, north of Lomas Boulevard, and east of the I25 southbound off-ramp to Lomas Boulevard. Figure 1 depicts the test shaft locations.

Regional Geology

Albuquerque, New Mexico is located within the Basin and Range physiographic province, bordered to the east by the Great Plains, to the north by the southern Rocky Mountains, and to the west by the Colorado Plateau. Albuquerque is situated within the Rio Grande Rift valley or trough of the Basin and Range province. The Rio Grande Rift is a middle to late Oligocene structural feature, characterized by basins in echelon which are elongated north-south and are bordered by uplifts along the entire length of the rift zone, from southern Colorado to El Paso, Texas (Clary et al., 1984). The Albuquerque Basin is the largest of the structural basins in the Rio Grande trough. The geologic units associated with this geomorphology include Precambrian to Tertiary bedrock within the uplifted mountain blocks and thick accumulations of Tertiary to Quaternary alluvial basin fill sediments and localized volcanics within the down-dropped basins. Figure 2 depicts the geology of the Albuquerque Basin from Clary et al. (1984) (after Kelley, 1977).

The Sandia Mountains, located about 20 kilometers east of downtown Albuquerque, form the eastern bordering uplifted block. Several thousand meters of Precambrian metamorphic rocks (metavolcanics and meta-sediments) are exposed in the Sandia Mountains as a result of about 6,500 to 7,000 meters of differential vertical movement along one or more faults. The western margin of the Albuquerque Basin is marked by a complex system of faults, beginning with the Nine Mile fault due west of Albuquerque. Exposed Precambrian rocks along the western boundary of the basin consist of the Laudron Mountain uplift about 80 kilometers southwest of Albuquerque. Several volcanic cones, volcanic flow fields and intruded igneous masses located around the perimeter of the Albuquerque Basin are evidence of Basin and Range province Tertiary volcanism and intrusion. During Oligocene and Miocene time, intrusion of numerous monzonitic stocks, dikes sills and plugs was accompanied by complex faulting and folding (Disbrow and Stoll, 1957), primarily north and east of the Albuquerque Basin and east of the Sandia Mountains.

Chapin (1979) indicates that during early Miocene time (32 to 27 million years ago), reactivated north-trending zones of weakness caused crustal extension. Broad, shallow basins were broken and offset as the rifting proceeded, forming a series of west-northwest and northeast trending lineaments within the basement complex of the southern Rocky Mountains. The Albuquerque Basin lies within the middle of the 800- to 900-km long Rio Grande Rift valley and is filled with up to 4,000 meters of Tertiary-Quaternary alluvial basin fill deposits. Gentle topographic relief of the basin floor and relatively low dip of the basin fill sediments result in minimal exposures of the sediments. Uplift of the Sandia Mountains block is thought to have occurred concurrently with subsidence and filling of the basin (late Miocene time into the Quaternary) (Kelley and Northrop, 1975). A major portion of the uplift may have been after early Santa Fe time (10 million years ago); however, the presence of Holocene scarps suggests that the uplift continues into the present.

Geologic units of engineering significance within the basin consist of Quaternary relatively thin fluvial and alluvial fan deposits associated with current floor geometry and drainage patterns, including Holocene- and Pleistocene-age river-deposited alluvium and alluvial fan deposits up to about 40 meters thick at the surface throughout most of the Rio Grande River floodplain; Pleistocene basalt flows on the west mesa of Albuquerque (associated with numerous large and small eruptive centers) and locally known as the Volcano Cliffs ; Pliocene Santa Fe Formation sediments, which underlie the young surface deposits or outcrop in the form of thin, narrow bands along both sides of the Rio Grande River Valley; and Pennsylvanian Madera Limestone and Precambrian Sandia Granite, both of which outcrop east of Albuquerque.

Site Geology

The Big I is located along the Rio Grande River floodplain escarpment formed by eruption and down-cutting (erosion) into the Tertiary sequence. The core of the project site lies below the escarpment, where the Tertiary Santa Fe Formation outcrops. The Santa Fe Formation consists of clay, silt sand and gravel deposits, and is locally composed of overconsolidated, interbedded alluvial fan deposits and interbedded clays, mudstones, sands and gravels (variably indurated with calcareous cementation) and cobbles. There is little difference between the grain size of the Santa Fe Formation and the overlying Quarternary alluvium; soils which classify (USCS classification; ASTM D2487) as poorly graded sands (SP), silty sands (SM) and well graded sands (SW) are common to both geologic units. Standard penetration test (SPT: ASTM D1586) N-values vary widely within the Quaternary alluvium, from very low blow counts within fine-grained, moist layers to refusal blow counts (N>100 blows/0.3 meter) in coarse-grained gravel and cobble layers. The contact between the alluvium and the Santa Fe Formation is sometimes indicated by the presence of well graded sands or poorly graded gravels (GP) which possess high N-values as a consequence of the erosional surface. Unweathered, granular Santa Fe Formation materials are identified by consistent refusal SPT blow counts.

Subsurface Investigation

Subsurface investigation of the test shaft sites was performed as part of the Big I Project geotechnical investigation in support of the new interchange design. Hollow-stem auger borings, standard penetration testing and sampling and occasional Shelby tube sampling, cone penetration testing and laboratory testing were performed as part of the investigation.

Design of Test Shafts & Internal Instrumentation

Structural design of the test shafts was performed in accordance with American Concrete Institute (ACI) methods and with local construction practice within the Albuquerque area. A steel/concrete ratio of one percent was used, and paired #10 longitudinal bars and #4 hoops were utilized in the basic design. Internal reinforcing bars (spiral bars and X-bars) were included in order to minimize racking of the reinforcing cages when lifted. Cross-hole sonic logging tubes (six in Shaft 1-A, four in Shaft L-1) were located at respective 60- and 90-degree intervals around the cage perimeter and extended for the full cage length.

Instrumentation installed within each test shaft consisted of vibrating wire strain gages (VWSGs), mechanical and embedded telltales, and linear vibrating wire displacement transducers (LVWDTs). Figures 3 and 4 depict the strain gage and mechanical telltale locations and a summary boring log of the shaft. Twenty-four strain gages were installed in Test Shaft 1-A, and 16 strain gages were placed in Test Shaft L-1. Strain gages measured distribution of load (load transfer) at selected locations along the length of each test shaft, and were attached to the reinforcing steel cage at the locations shown in the figures. Preliminary strain gage layouts developed during the design phase were revised in the field based on the lithology encountered in the test shaft excavations. Each strain gage consisted of a Roctest (Plattsburgh, New York) Model SM-5A weldable gage with built-in thermistor, individually mounted (welded) to a 0.9 m (3 ft) long #5 deformed sister bar . Strain measurements were obtained directly in units of microstrain using an automated data logger linked to a notebook computer.

Telltales installed in each test shaft to permit independent measurement of the shaft movement at selected depths and shaft elastic compression between the O-cells included mechanical telltales which extended to just below the lower (base) plate of the respective O-cell and located above the mid O-cell, and embedded telltales (LVWDTs) located between the O-cells. Mechanical telltales consisted of steel rods inside a PVC sheath attached to the reinforcing steel cage, with a short length of deformed bar or a deadman plate embedded in the shaft concrete at the tip. Instrumen-tation for measurement of the O-cell expansion consisted of LVWDTs (three for each O-cell) located at 120-degree intervals.

Test Shaft Construction & Logging

Test Shaft 1-A was constructed to an overall depth of 24.8 meters (m) (81.5 feet) with a nominal diameter of 1,830 mm (72 in). Test Shaft L-1 was constructed to an overall depth of 16.0 m (52.5 ft) with a nominal diameter of 1,372 mm (54 in). Typically, a 915 mm (36 in) diameter pilot hole was excavated to full depth using a bullet auger, and the excavation was then reamed with a 1,830 mm (72 in) or 1,372 mm (54 in) diameter double-cutter dirt auger. A Watson 3100 track-mounted drill rig was utilized to advance the test shaft excavations. The upper 1.2 m (4 ft) of the Shaft 1-A was encased in 1,855 mm (73 in) I.D. permanent steel casing to prevent the auger from enlarging the diameter of the near-surface portion of the shaft excavation. Stabilization of the shaft excavation was limited to a caving section located between depths of 15.2 to 18.3 m (50 to 60 ft) below the ground surface, which was stabilized using a grout collar which was poured, permitted to set-up, and then re-drilled. Special stabilization techniques were not necessary during excavation of Shaft L-1, and steel surface casing was not utilized due to the potential for caving or sloughing of coarse-grained surficial soils during casing installation and drilling. Since surface casing was not utilized, significant enlargement of the near-surface portion of the shaft excavation to a depth of about 5 m (15 ft) occurred during pilot drilling. Localized sloughing of the near-surface coarse-grained soils within the excavation occurred during caliper logging and placing of the reinforcing cage.

The as-drilled diameter of the test shaft excavations (prior to placement of reinforcing steel and concrete) was measured utilizing a three-arm caliper and two or three logging runs for each excavation. Each caliper run began a short distance above the hole bottom, such that the tip of each excavation was not measured for diameter (assumed nominal bottom diameters were utilized for computation of end-bearing pressures). Measured diameters ranged from about 1,803 to 2,007 mm (71 to 79 inches) in Shaft 1-A and 1,397 to 1,600 mm (55 to 63 inches) in Shaft L-1. In addition, a video log of the Shaft 1-A excavation was made by lowering a specialized video camera to the bottom and then slowly raising the camera in approximate 1.5 m (5 ft) increments, stopping at each increment to rotate the camera through 360 degrees. The video image was viewed in real-time on a television monitor at the ground surface.

Steel reinforcing cages were fabricated in two segments; the lower segment included the two O-cells and associated plates. Strain gages with sister bars were pre-mounted on the cage segments prior to lifting and setting. Placing the cage in the excavation was achieved by lowering the lower segment and anchoring the cage at the ground surface, hanging the upper socket cage in position over the lower, welding the segments together, and then lowering the entire cage into position, with the cage bottom set at about 0.46 m (1.5 ft) above the hole bottom. O-cell hydraulic hoses, LVWDT and strain gage cables, telltale rods and cross-hole sonic logging tubes were progressively extended and attached to the cage as the cage was lowered.

Test shaft concrete consisted of a 19 mm (3/4 in) maximum aggregate mix, with design seven-day compressive strength of 31.7 MPa (4,600 pounds/square inch {psi}) and slump of 152 to 203 mm (six to eight inches) during placement. Concrete was tremied into the shaft using a pipe carefully inserted through holes in the O-cell plates to reach the shaft base and then slowly withdrawn as the level of concrete rose. During concrete placement, the tremie pipe tip was maintained below the top of the fresh concrete, in order to avoid damage to the internal instrumentation by free-falling concrete. Estimated concrete takes were about four and 12 percent greater than the neat-line volume, respectively, for Shafts 1-A and L-1. Results of concrete compressive strength tests at five to seven days (corresponding to the date of load testing) ranged from about 16.3 to 22.8 MPa (2,400 to 3,300 psi).

After initial concrete set and prior to load testing, cross-hole sonic logging of the test shafts was conducted. Fifteen and six logs, representing all paired combinations of all tubes, were performed for Shafts 1-A and L-1, respectively. Average, measured compression wave velocities were in the range of 3,660 to 3,750 m/second (12,000 to 12,300 ft/second) for the two test shafts, indicative of good quality (sound) concrete, with the exception of anomalous (soft) zones encountered within the perimeter of the bottom 100 to 300 mm (0.3 to 1.0 ft) of Test Shaft 1-A and the full section of the bottom 60 to 180 mm (0.2 to 0.6 ft) of Test Shaft L-1 with material wave velocities about 11 to 40 percent (Test Shaft 1-A) and about 28 to 33 percent (Test Shaft L-1) lower than the sound concrete values. The reduced-velocity zones were thought to be due to sloughed soils at the edges (Shaft 1-A) or across the entirety (Shaft L-1) of the shaft base.

Load Testing Equipment & External Instrumentation

The load tests were performed using 660 mm (26-inch) diameter Loadtest, Inc. O-cells with a rated load capacity of 16,015 kN (1,800 tons) in each direction, installed at the locations shown in Figures 3 and 4. A Loadtest hydraulic pump was used to pressurize the O-cells, and pressure measurement was by high-pressure Bourdon gages and an electronic pressure transducer. Water was utilized as the hydraulic fluid.

Digital dial indicators and LVDTs were used to monitor vertical movement of the mechanical telltales and the top of the shaft (using the exposed end of selected longitudinal reinforcing bars). All top-mounted instruments were secured to a steel independent reference beam supported on earthen berms and timbers at a minimum clear distance of 5.5 m (18 ft) from the shaft. Load test equipment and external instrumentation were read and recorded manually and automatically by notebook computer. Thermal movement of the reference beam was monitored utilizing a digital level, and top-of-shaft movement data was corrected using the reference beam deflection data.

Load Testing Procedure

The O-cell load tests were performed in general accordance with ASTM D1143, Quick Load Test Procedure. Load was applied through the hydraulic pump system, generally in increments of 4,137 kPa (600 psi) for four minutes each; instruments were read at load application and at one-, two- and four-minute intervals. Strain gages were read automatically at 30- to 45-second intervals. The pump was then activated (or the valve opened in the case of an unload cycle) to apply the next load increment.

Each load test consisted of two stages: Stage 1 Pressurize lower O-cell (at shaft base) while the mid O-cell is locked off to assess end-bearing using the entire shaft side shear as a reaction; Stage 2 Pressurize mid O-cell (at two-thirds shaft depth) with the lower O-cell open (free to drain no engagement of end bearing) to assess side shear of the shaft segment between the O-cells (lower socket), using side shear above the mid O-cell as reaction. Initially, a third stage was planned, involving re-pressurizing the mid O-cell to assess side shear in the upper socket with lower socket side shear and end-bearing as reaction. However, only Stages 1 and 2 were completed for each test shaft. Excessive tilting of the Shaft 1-A lower O-cell at the maximum O-cell base movement precluded closing the lower O-cell to engage end-bearing and execution of Stage 3. However, it was thought that the magnitude of upward movement of the upper shaft socket in Stage 2 (29 mm or 1.1 in) may have resulted in full development of side shear for the upper socket, which is the objective of the Stage 3 test. For Test Shaft L-1, the lower O-cell seal apparently was lost at the end of the Stage 1 test, at which point several gallons of water were pumped to the lower O-cell without any back-pressure, precluding execution of Stage 3. It is likely that the magnitude of upward movement of the upper socket in Stage 2 (31 mm or 1.2 in) resulted in full development of side shear for the upper socket, which is the intent of the Stage 3 test.

Load Test Results

Load test data was reduced and graphed in several ways, including interpreted load versus depth (load transfer); applied load versus movement; side shear versus depth (for selected load increments of each stage); and side shear mobilization curves (side shear versus shaft movement) for Stage 2 tests. Microstrain values were adjusted for elastic compression of the shaft and then converted to load by calculation utilizing the shaft diameter (caliper log results) and rigidity (adjusted for steel area). In general, the two or three interpreted load values at each gage level were averaged to account for any eccentricity in the shaft loading, yielding a single interpreted load value for each level. Exceptions to this approach occurred in situations where, in our judgment, a particular strain gage or gages yielded questionable data, such as possibly due to eccentric loading effects or erratic operation. The general shape of the interpreted load versus depth curves was used in applying judgment to the strain gage data. The total elastic compression of a given socket at each load increment was divided by the number of strain gage levels within the socket and then converted to microstrain, permitting adjustment of the strain gage readings for elastic compression. Side shear values for each load increment were estimated using the tributary area method (shaft side surface area for each strain gage level bounded by mid-points between the gages or by a mid-point and an O-cell plate) and the interpreted load values.

Estimated values of the ratio of horizontal to vertical effective overburden stress (K) were developed for all load increments of each stage utilizing the side shear data and estimated values of soil unit weight and friction angle based on Big I Project-specific criteria and SPT N-values. Estimated values of the adhesion factor (α) for fine grained soil strata were back-calculated (for Stage 1) in a similar fashion, using the side shear data and an estimated value of the undrained shear strength based on SPT N-values and published correlations between N and s_u or unconfined compressive strength for similar soils (however, use of undrained shear strength to characterize the fine grained, partially-saturated, overconsolidated and very stiff to hard Tertiary-age soils may be inappropriate, as these soils likely possess both frictional and cohesive components). Adjusted side shear values were computed by subtracting the weight of the portion of the shaft socket above the gage level from the interpreted load at the gage level. The adjusted side shears were then plotted against the movement of the socket, adjusted for elastic compression of the socket at the gage level.

Test data and results for Shaft L-1 are discussed below and presented in Figures 5 and 7, respectively presenting interpreted load versus depth and load versus movement for the Stage 1 test. The Stage 1 test was performed to a maximum gross applied load of 14,412 kN (1,620 tons) and maximum net load of 14,082 kN (1,583 tons) at a measured downward O-cell base movement of about 71 mm (2.8 in) (about 5 percent of the shaft diameter). The maximum load was maintained for a period of four minutes, with associated downward O-cell base creep of about 3 mm (0.12 in). The shape of the Stage 1 load versus movement curve suggested that compression of slough at the hole bottom may account for about 13 mm (0.5 in) of the initial downward movement of the O-cell base, and further that end-bearing was not fully engaged until after this initial movement. Therefore, the end-bearing likely was engaged over a net downward movement of about 58 mm (2.3 in), which is about 4 percent of the shaft (nominal) diameter. It is believed that the ultimate end-bearing of the shaft was approached at this magnitude of movement; utilizing the shaft diameter of 1,372 mm (54 in), without adjustment for the side shear capacity of the short segment of shaft below the lower O-cell, a maximum end-bearing value of about 9,738 kPa (101.7 tsf) based on gross load was computed for the shaft base.

The total upward movement of Shaft L-1 at the maximum load increment was about 7.5 mm (0.3 in), including adjustment for elastic compression; upward creep was less than 1 mm (0.04 in). After the maximum load increment, the hydraulic pump/O-cell system lost all pressure, such that further load could not be applied. However, the magnitude of lower O-cell expansion prior to the pressure loss proved sufficient to permit Stage 2 testing. The estimated, average elastic compression of the shaft was 0.4 mm (0.02 in) for the upper socket and 1.0 mm (0.04 in) for the lower socket at the maximum applied load.

The estimated shaft side shear at the maximum applied load varied from zero at Levels 7 and 8, to 81 kPa (0.8 tsf) at Level 6, to 449 kPa (4.7 tsf) at Level 3, with significantly higher values of 865 kPa and 1,379 kPa (9.0 and 14.4 tsf) at Levels 2 and 1, respectively. Estimated values of K varied from zero at Levels 7 and 8, to 1.1 (Level 6) to 2.8 (Level 5), with elevated values of 5.9 and 7.6 for Levels 2 and 1. Quite high estimated values of side shear and k were computed for the Level 1 and 2 strain gages throughout the Stage 1 test. These strain gage data may have been affected by proximity to the lower O-cell, or the side shear/K-value computations may be affected by geometric characteristics of the lower portion of the shaft. The estimated side shear and K-values for Levels 1 and 2 were not utilized as reliable estimates of side shear capacity for this segment of the shaft. The small magnitude of upward shaft movement (7.5 mm or 0.3 in) when compared to an ultimate side shear failure criterion of one percent of the diameter (14 mm or 0.6 in), indicates that the ultimate side shear capacity of the entire shaft above the lower O-cell was not achieved in Stage 1.

The Stage 2 test of Shaft L-1 was performed to a maximum gross applied load of 11,049 kN (1,242 tons) and maximum net loads of 10,892 kN (1,224 tons) in lower socket side shear and 10,349 kN (1,163 tons) in upper socket side shear. The total downward movement of the lower shaft socket and total upward movement of the upper socket were about 31 mm (1.2 in) and 47 mm (1.9 in), respectively at the maximum applied load, including adjustment for elastic compression. The load versus movement data for the lower O-cell indicated that some load was transferred to the shaft base, even though this O-cell was free to drain. This was attributed to soil (possibly sidewall slough) trapped in the annular space around the lower O-cell and compressed between the plates as this O-cell drained and closed. Data for the telltale located at the shaft base, below the lower O-cell, indicated upward O-cell base movement of about 15 mm (0.6 in) during Stage 2, possibly suggesting drag or some similar effect. The maximum upward movement of the upper socket was measured to be about 43 mm (1.7 in) including elastic compression. Figures 6 and 8 respectively present interpreted load versus depth and load versus movement for the Stage 2 test.

The magnitude of upper socket movement (31 mm or 1.2 in) prior to the hydraulic leak is greater than the failure criterion of one percent of the shaft diameter (14 mm or 0.6 in), indicating that side shear failure was achieved for the upper socket in this stage. The downward movement of the lower socket was 47 mm (1.9 in), which also exceeds the failure criterion. The magnitude of movement suggests that ultimate side shear failure of both the upper and lower shaft sockets was achieved in Stage 2. Estimated side shear values at the maximum applied load varied from zero at Levels 7 and 8 (near top of shaft) to 272 kPa (2.8 tsf) at Level 4 for the upper shaft socket, and from 282 to 706 kPa (2.9 to 7.4 tsf) for the lower socket. Estimated values of K at the maximum load varied from zero (Levels 7 and 8) to 2.5 (Level 5) for the upper socket and from 1.8 to 4.8 for the lower socket.

Discussion & Conclusions

For Test Shaft L-1, the measured ultimate capacity in uplift for the upper socket (Stage 2 test) was almost 200 percent of the estimated capacity, with a socket upward movement (adjusted for elastic compression of about 31 mm (1.2 in). The measured downward capacity of the lower socket (without engagement of end-bearing, though the Stage 2 test data suggest that some fraction of the applied load may have transferred through the lower O-cell to the shaft base) was about 250 percent of the estimated capacity, at the downward shaft movement of 47 mm (1.9 in), including elastic compression. The measured end-bearing capacity was 14,412 kN (1,620 tons), corresponding to an equivalent bearing pressure of 9,738 kPa (101.7) tsf at the estimated net downward movement of the lower O-cell base of about 58 mm (2.3 in) after compression of slough. The estimated bearing capacity and equivalent pressure were 2,669 kN (300 tons) and 1,816 kPa (19.0 tsf), or about one-fifth of the measured values, considering that after compression of the slough, ultimate end-bearing at the shaft base likely was achieved during the test.

The method of Mayne and Harris (1993), which is applicable to dense to very dense, highly overconsolidated geomaterials, was utilized to estimate side and tip resistance. Although this method has been reported to provide higher skin friction values for overconsolidated materials than the standard alpha or beta methods, for Santa Fe soils the method underestimated the side resistance afforded by the site soils. For the case of end-bearing, use of the method appeared to imply a factor of safety of about three or greater against ultimate capacity. It was recognized that in order to limit settlement of production shafts to a tolerable level, shaft settlement on the order of 12 mm (0.5 in) would result in development of only a portion (estimated to be about one-third to one-half) of the ultimate end-bearing capacity. In the case of shaft side resistance, use of the Mayne and Harris (1993) method for the site soils appears to imply a factor of safety of about 1.2 to 2 or perhaps greater. When examining the test data, it should be noted that the ultimate side shear capacity of the lower socket for Shaft 1-A was not reached in Stage 2, and further that some fraction of the Stage 2 applied load may have transferred through the lower O-cell to the shaft base as end-bearing for Shaft L-1.

Recommended values of the ratio of horizontal to vertical stress and the adhesion factor for use in design of drilled shafts were developed for each soil layer encountered within the entirety of the Big I project site, along with end-bearing values for drilled shafts bearing in the Santa Fe Formation. Deep foundations for the project structures are anticipated to be in Santa Fe Formation materials. The recommended values of K and α for soil layers encountered at the test shaft sites were based on review and analysis of the load test data, in particular the back-calculated values of these parameters. Recommended k- and α-values for soil layers not encountered in the test shafts were estimated by comparing the estimated values of friction angle or undrained strength for these layers to those of the layers encountered in the load tests. Recommended parameter ranges for the various soil layers are as follows:

Fill -	recommended K-values of 0.5 to 0.6.
Valley Alluvium -	recommended K-value of 0.5 within the upper portion, increasing to 1.0 to 1.5 in lower layers.
Terrace Alluvium -	recommended K-values of 0.8 to 1.2 within the granular layers; -value of 0.5 within the clayey layer.
Santa Fe Fm. -	recommended K-values for the Santa Fe ranged from 0.7 to 1.0 within the weathered Santa Fe layers, to 2 to 3 for the gravelly layers, to 1.0 to 3.5 for the unweathered layers; recommended α-values of 0.5 to 1.0 for clayey layers respectively.

Recommended k-values for the gravelly and unweathered Santa Fe Formation layers were based on the results of the load tests and the density (reflecting the degree of induration or overconsolidation) and shear strength of these layers (as measured by the estimated values of friction angle, derived from SPT N-values). The indurated, unweathered layers of the Santa Fe Formation likely exhibit shear strength which includes both frictional and cohesive components; therefore, the recommended values of K likely reflect some allowance for contribution to overall shear strength from the cohesion.

An ultimate end-bearing pressure of 5,750 kPa (60 tsf) was recommended for use in design of shafts bearing in the unweathered, indurated layers of the Santa Fe Formation. This recommended value is equivalent to the bearing pressure measured for the 1,830 mm (6-ft) diameter Test Shaft 1-A, though it was thought that maximum end-bearing capacity likely was not mobilized for this shaft. The recommended value is about 60 percent of the maximum bearing pressure measured for Test Shaft L-1.

Acknowledgements

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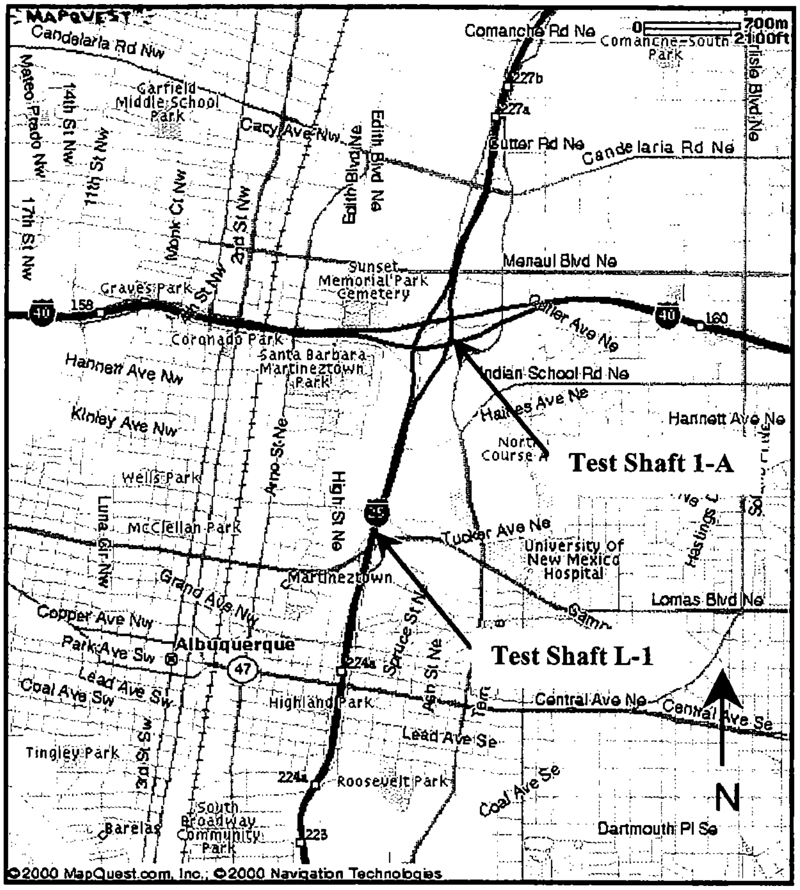

Figure 1 – Vicinity map.

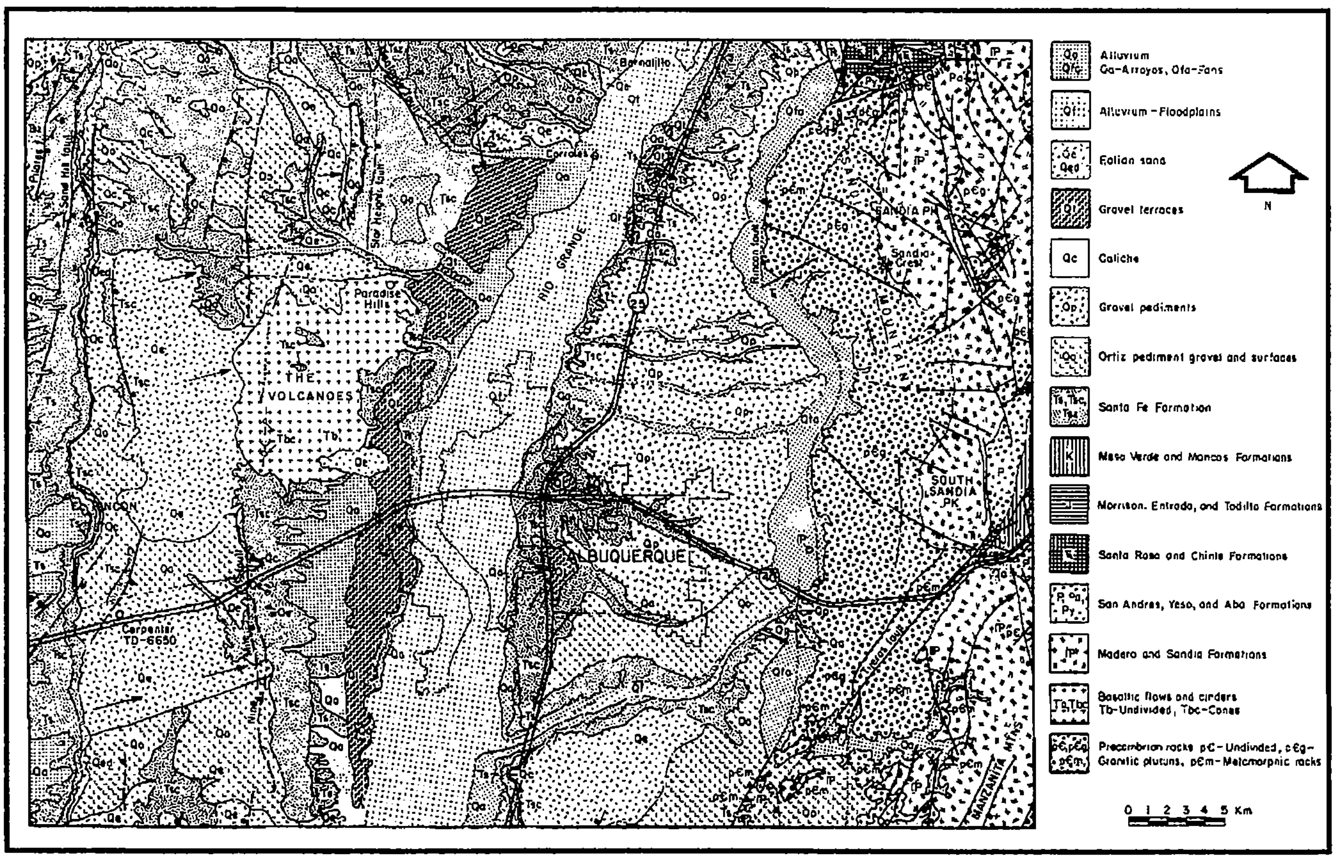

Figure 2 - Geologic map of the Albuquerque area (from Kelley, 1977)

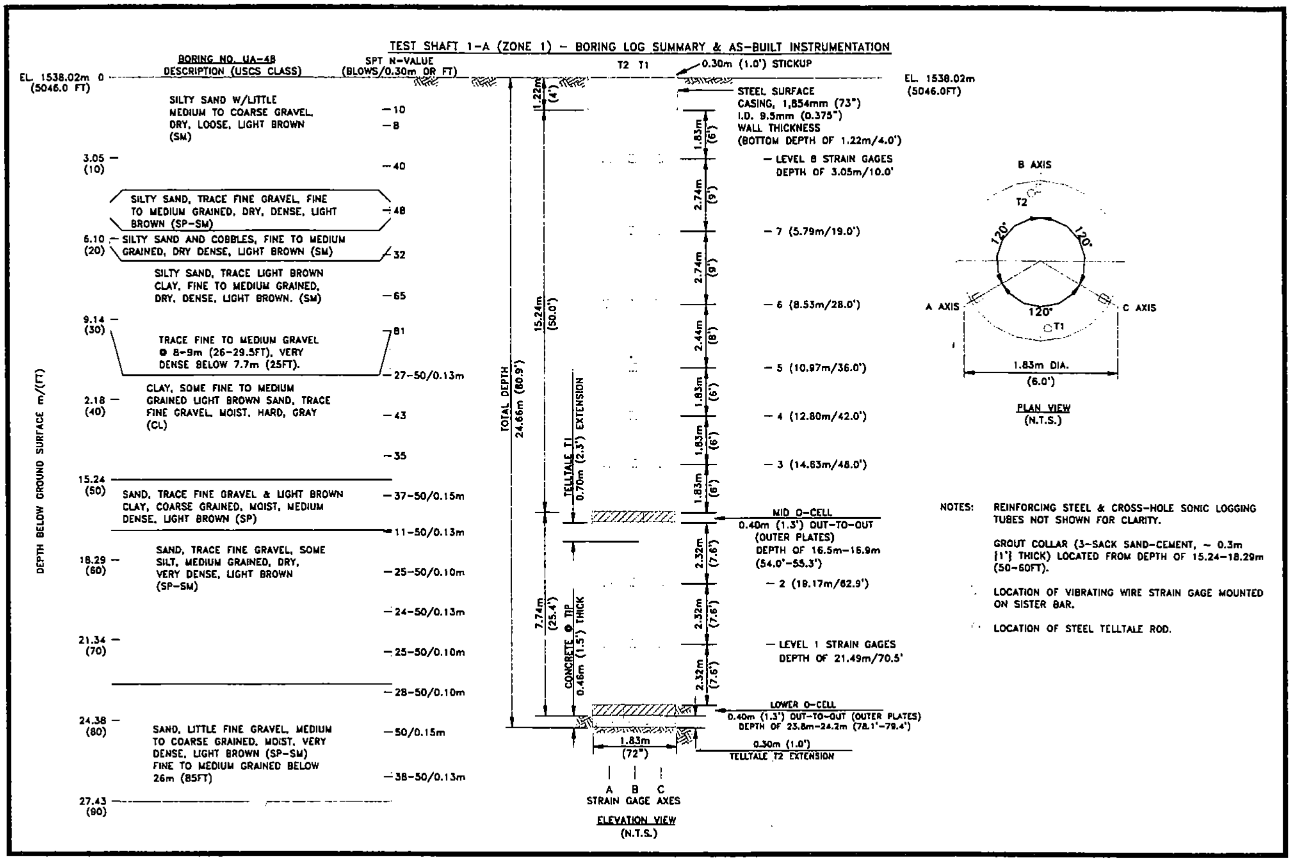

Figure 3 – Schematic of Test Shaft 1-A.

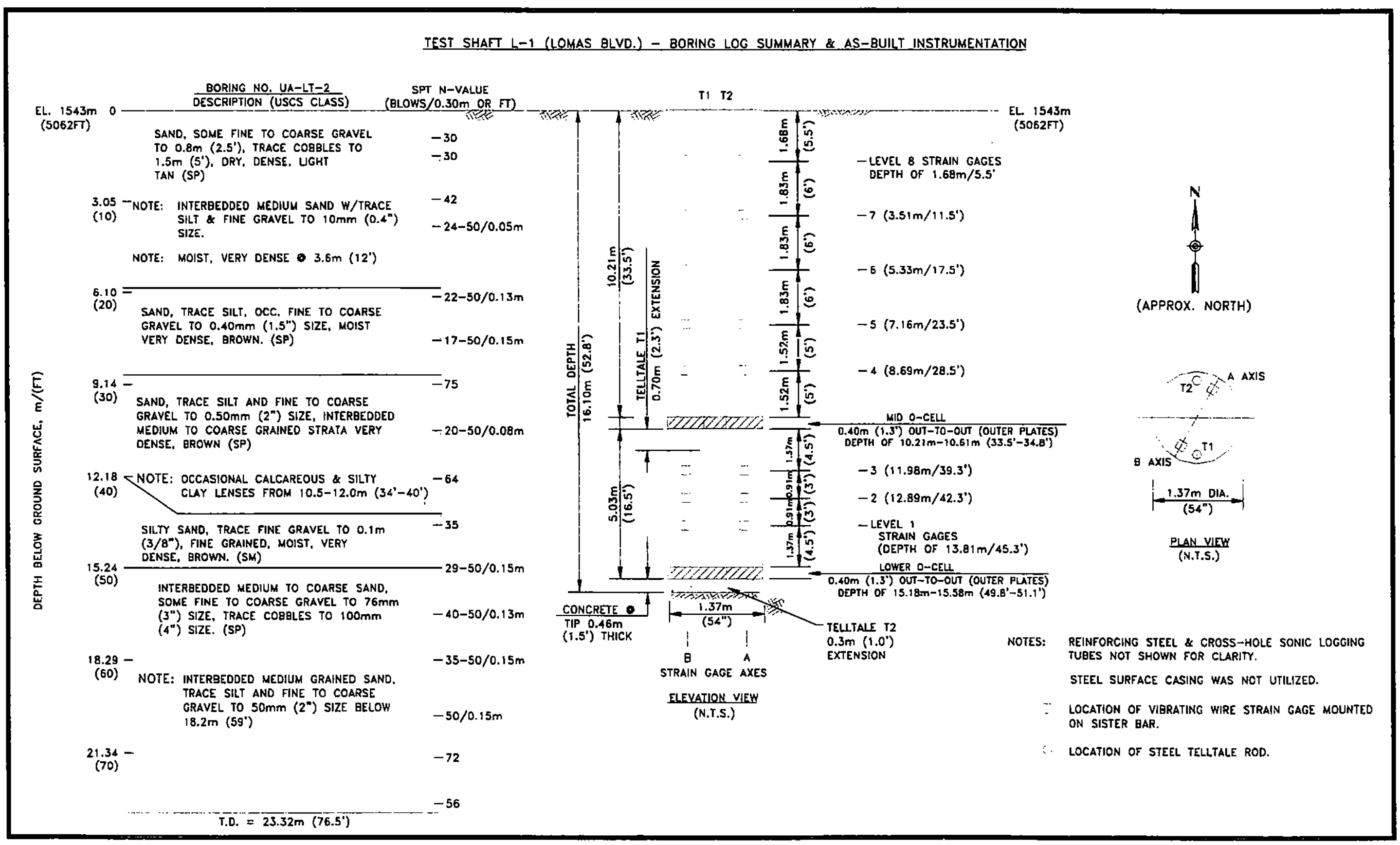

Figure 4 – Schematic of Test Shaft L-1.

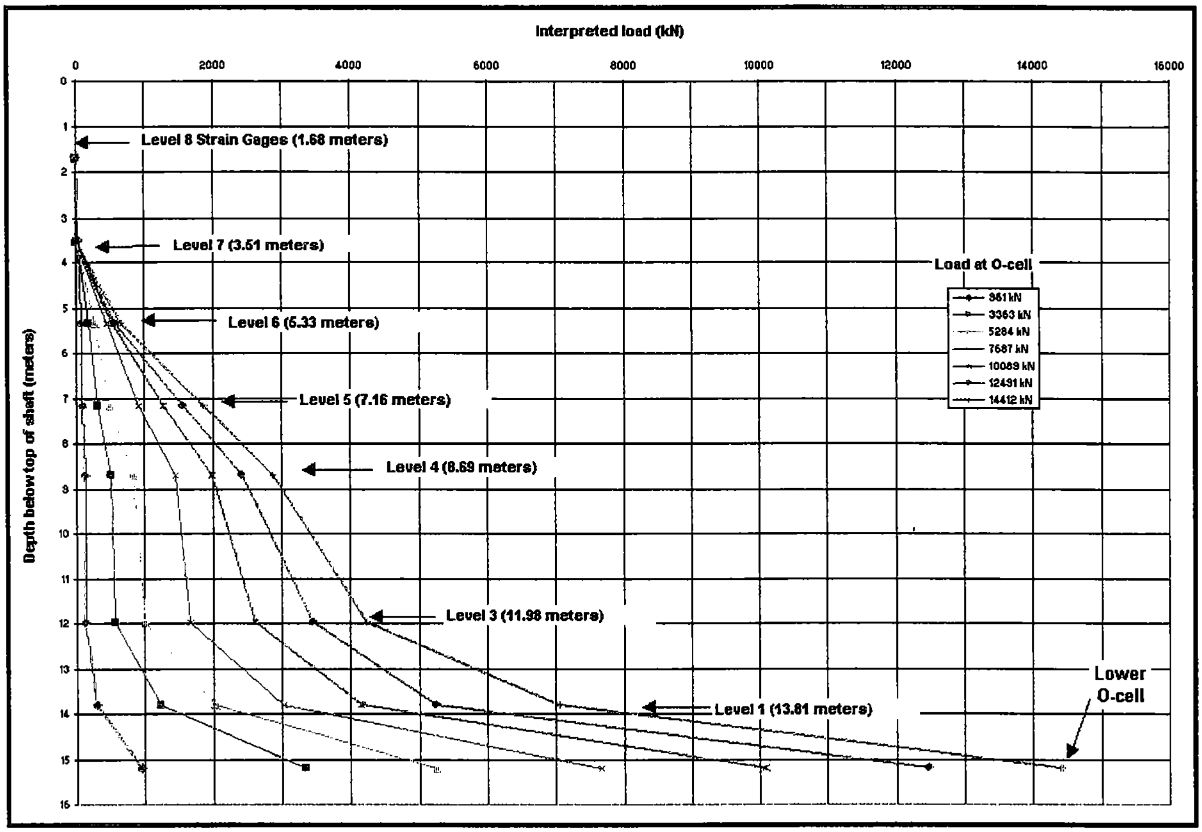

Figure 5 – Shaft load transfer, Stage 1 Test, Shaft L-1

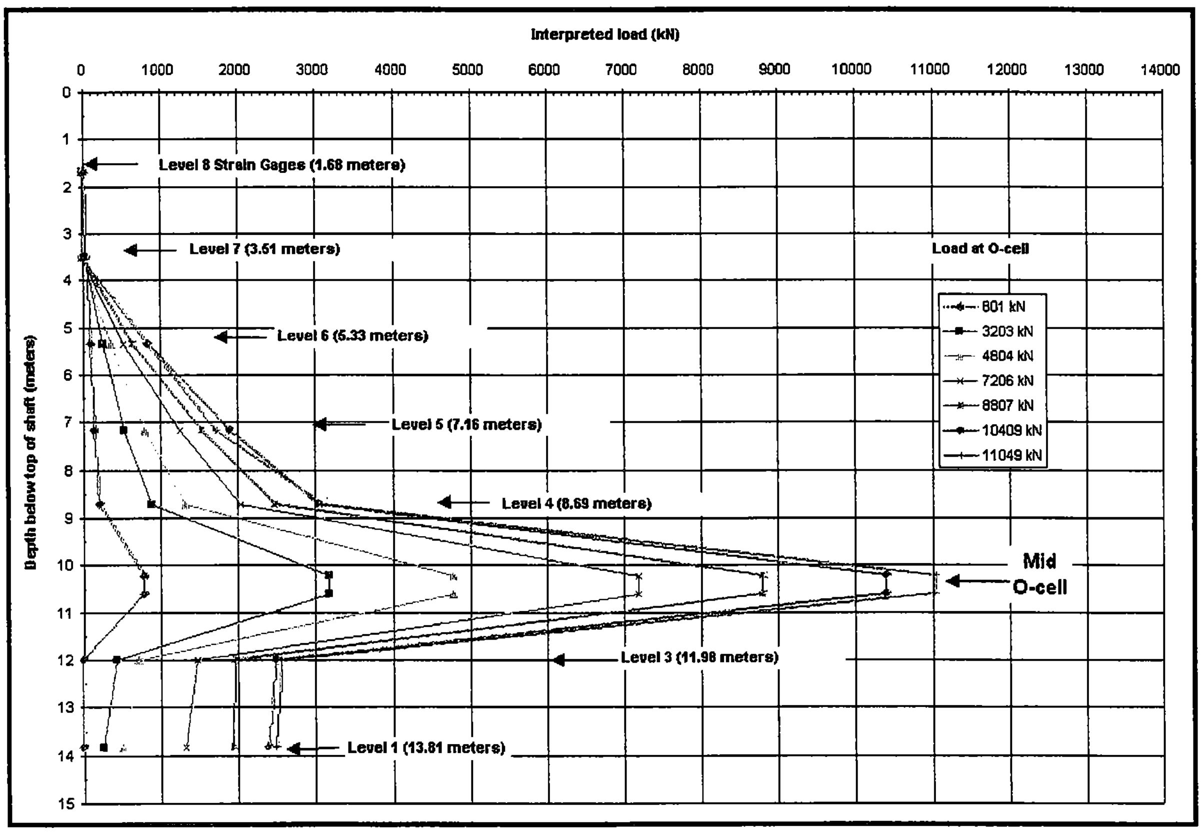

Figure 6 – Shaft load transfer, Stage 2 Test, Shaft L-1.

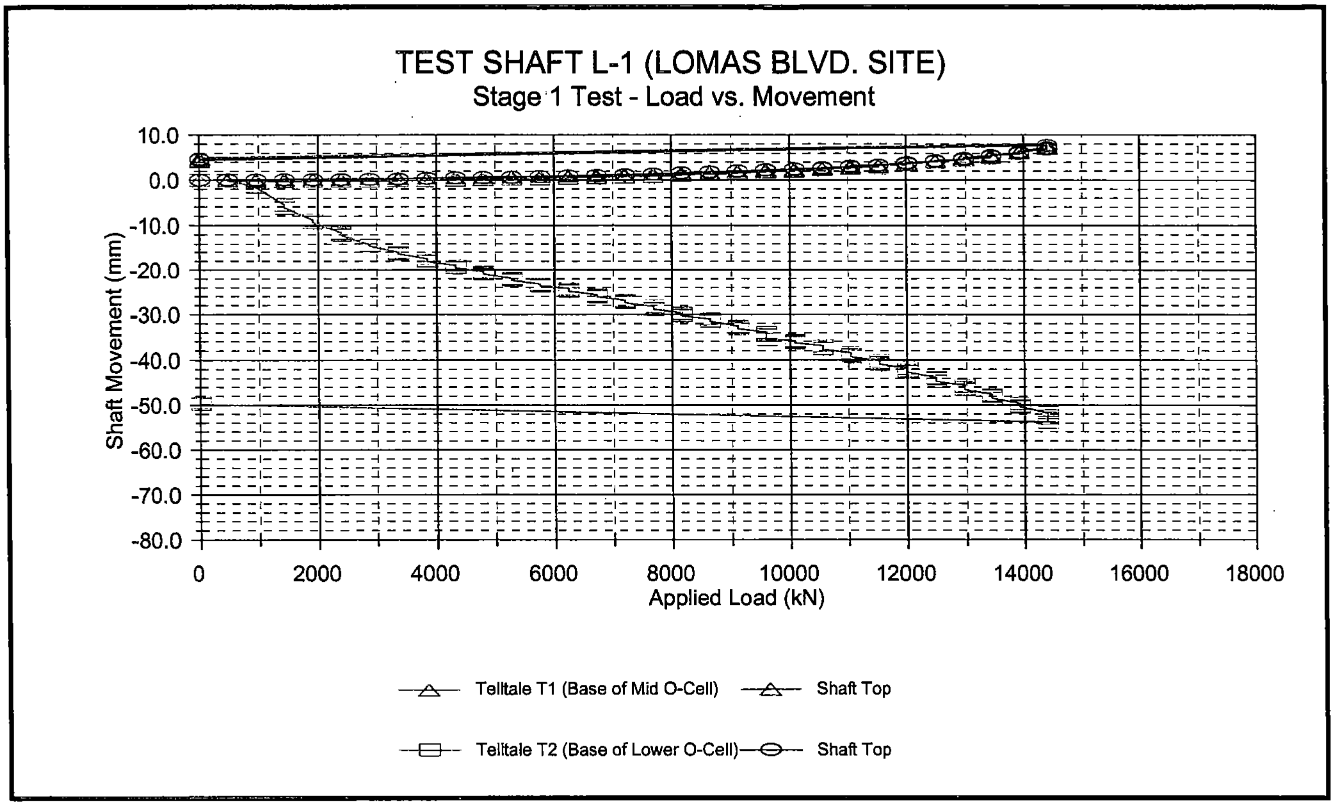

Figure 7 – Load vs movement, Stage 1 Test, Shaft L-1.

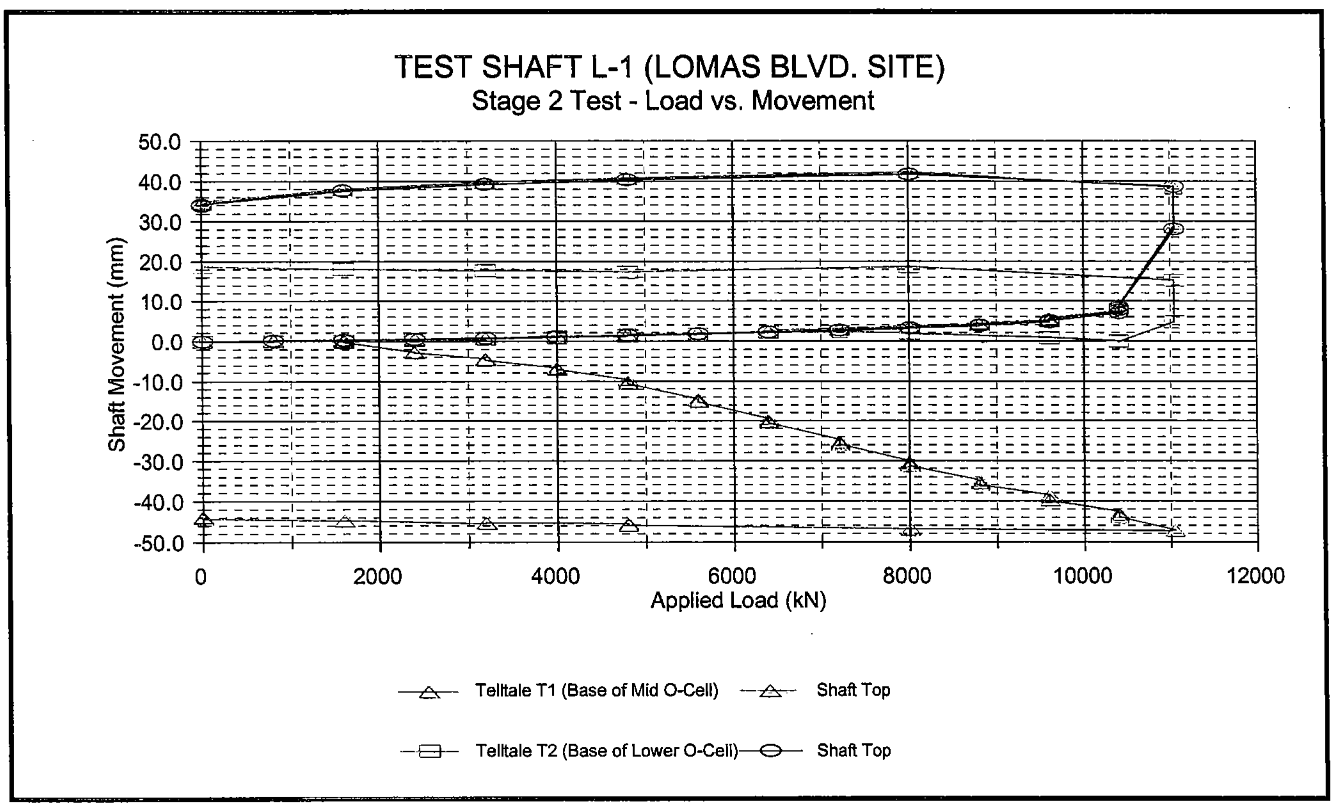

Figure 8 – Load vs movement, Stage 2 Test, Shaft L-1.

UNDERGROUND COAL MINING ALONG THE MON-FAYETTE EXPRESSWAY: ISSUES, ASSESSMENT AND TREATMENT

by: *Don Gaffney, Michael Baker Jr., Inc. (dgaffney@mbakercorp.com); Mark Mayle, TriLine Associates, Inc.; and Ken Heirendt, Pennsylvania Turnpike Commission*

ABSTRACT

In 1994, the Pennsylvania Turnpike Commission (PTC) received environmental clearance to proceed with the final design and construction of the first sections of a 65-mile, limited access roadway from Pittsburgh, PA, to Morgantown, WV. This roadway is in the heart of Pennsylvania's bituminous coal field. Consequently, a wide range of underground coal-mining issues were addressed.

These issues included subsidence from abandoned workings; impacts on active mine workings and reserves; modification of permitted surface facilities; and secondary effects including acid mine drainage and mine fires. Within the constraints of Pennsylvania's coal-mining law and based on other recent successful projects, technical guidelines were developed to be applied by the final design consultants. Threshold criteria were established for various treatment measures, and sample special provisions and details were included.

These guidelines were modified during the design process as site specific conditions were determined, and then were implemented during construction. The varied success of the special provisions and details during early construction sections led to their further modification.

Several specific examples illustrate how assessment and treatment evolved during final design and construction. The examples include structure foundation support over abandoned and active workings, where guidelines were modified in response to economic, risk-benefit considerations; roadway support over a coal slurry impoundment, where a contractor option was approved in lieu of the original contract plan; and exposure of coal mines during roadway excavations, where the potential of encountering mine pools and mine fires were addressed.

PROJECT SETTING

The Mon Fayette Expressway has received national prominence, with articles appearing in both Engineering News Record (ENR) (November 8, 1999) and Civil Engineering (October, 1999). The PTC is making an investment of more than $3 billion to design and construct a north-south, limited-access highway through southwestern Pennsylvania and into northern West Virginia. The highway is through the heart of the Pennsylvania bituminous coal field which once fueled Pittsburgh's famous steel industry. Since the major mill closings of the 1980's the area has been economically depressed, and it is anticipated that this project will contribute to the area's revitalization.

The first 17 miles to be designed connect Interstate 70 at the south to PA Route 51 at the north, resulting in 21 construction contracts. Thirteen prime consultants are designing the Expressway, with assistance from numerous subconsultants. Michael Baker Jr., Inc., is design manager for the project, and Trumbull Corp. is construction manager. TriLine Associates, Inc, is a subconsultant to Trumbull, providing geotechnical and other services.

The Expressway cuts across the Appalachian Plateau as it generally follows the path of the Monongahela River. This plateau is typified by broad, rolling hills underlain by gently dipping bedrock and no significant geologic structure. Gentle anticlines and synclines locally are evident with a trend dipping to the Monongahela River.

The stratigraphic units encountered by the Mon-Fayette Expressway range from the Upper Pennsylvanian Conemaugh Group to the Permian Dunkard Group. Typical of all rocks in western Pennsylvania, lithologies consist of cyclic sequences of shale, claystone, limestone, sandstone, and coal. As seen by the following columnar section, coals frequently serve as marker beds dividing formations:

Group	Formation	Basal Bed
Dunkard	Washington	Washington Coal
	Waynesburg	Waynesburg Coal
Monongahela	Uniontown	Uniontown Coal
	Sewickley	Sewickley Coal
	Redstone	Redstone Coal
	Pittsburgh	Pittsburgh Coal
Conemaugh	Casselman	Ames Limestone
	Glenshaw	Upper Freeport Coal

The Pittsburgh Coal was the most widely mined coal seam that affected the Expressway, although both deep and strip mines were encountered in the Redstone Coal, Waynesburg Coal, and Washington Coal. For the most part, overburden rock of the Pittsburgh Coal consists of laminated shales and sandstones which are subject to raveling and collapsing upon loss of support due to coal removal.

The same year the Expressway received environmental clearance, the Pennsylvania legislature passed Act 54, which changed the rules by which underground coal mine operators could mine and impact surface features. No longer were they required to protect structures and other features from damage. Instead, they could compensate for or repair damages after mining. This allowed operators to convert from old, room-and-pillar techniques to long-wall mining. It has been reported that sections of Interstate 70, near the southern terminus of the Expressway's first construction sections, have been subsided as much as 5 feet by mining since the passage of this act. Allowance for subsidence in the Act and the desire not to have subsidence under the Expressway led to a disagreement with an active coal mine operator as to financial and operational damages from construction of the roadway. Legal issues are not the subject of this paper. However, the Expressway also crossed through the surface facilities of the mine, creating technical problems discussed below.

The PTC recently completed two other major expansion projects, the James E. Ross (Beaver Valley) Expressway (Route 60) and the Amos K. Hutchinson (Greensburg) Expressway (Route 66). Both of these projects also traversed Pennsylvania's bituminous coal field, and required treatments addressing underground coal mine impacts. The PTC had also been performing major renovations along its mainline (Interstate 76) in this area, dealing with underground mines. During its original construction through this area over 50 years ago, the PTC took precautions against impacts from coal mines. All of this practical experience came to bear on the approach to designs related to underground coal mines on this project.

DESIGN APPROACH

The construction of the Mon-Fayette Expressway in Washington and Allegheny Counties, PA, required addressing the issues related to underground coal mining. A basic design philosophy was adopted which differentiated first between active and abandoned mines underground, and second between structures and roadways at the surface. Active coal mines required negotiations not only for purchase of support coal and changes to MSHA-permitted surface activities, but also coordination of construction activities above ground with mining activities underground. Abandoned mines had the potential for subsidence, exposure, acid mine drainage, and mine fire problems.

Initially, design guidelines for treatment of abandoned mines were developed and issued to the prime consultants. These design guidelines were developed considering the PTC's experience with underground mines, understanding of the technical aspects of the issues, and benefit-risk cost comparisons. The guidelines evolved into a series of generic special provisions for underground mine-related roadway construction. Underground mine-related special provisions for structures were developed individually, considering the unique conditions presented in each case.

Original Guidelines for Structures - Recommendations to treat mines were defined in terms of depth to the coal seam and area of treatment. Wherever coal was found from 0 to 100 feet below the bottom of foundations, consultants were directed to recommend treatment to achieve 100 percent support. Depths greater than 100 feet were to be evaluated on a case-by-case basis. A second depth of 400 feet was provided, beyond which treatment would only be considered if the consultant "strongly believes" treatment is warranted.

The area of treatment should be based on the entire structure, not just individual substructures. Projecting angles for treatment outside the structure footprint were established based on depth to the seam:

Depth to Coal	Angle
0 - 100 feet	35 degrees
100 - 300 feet	25 degrees
300 - 400 feet	15 degrees

Other widths would be considered where the consultant "strongly believes" treatment is required.

While these guidelines did not directly reflect coal seam subsidence characteristics, they did represent an improvement from Pennsylvania's historical use of treatment to 50-percent support within a projected angle of 15 degrees starting 15 feet from the structure footprint.

Final Guidelines for Structures - The depth to the underlying mine still determined the method of stabilization for the bridge foundations. However, the areal extent of treatment was modified from the entire structure to individual substructure units. Also, the types of treatment became more standardized.

If the depth to the underlying mine was less than 50 feet below the bottom of footing elevation for the substructure, a deep-mine undercut was performed where the overburden between the substructure and the deep mine was removed. Then, depending on the specific conditions, either spread footings were placed below the mine level and the backfill placed, or the excavation was backfilled with specially selected material and then end-bearing piles were then driven into the stratum below the mine. In locations that the depth was greater than 50 feet below the bottom of footing elevation for the substructure, a detailed subsurface stabilization (grouting) plan was developed to fill the mine openings, overburden voids and subsidence fractures. Footings or piles were either placed to bear on the grouted rock, or piles were placed in holes drilled through the grouted rock to bear on the stratum below the mine.

Original Guidelines for Roadways - Design considerations for the construction of roadways considered mine conditions both above and below grade. Above-grade abandoned mines would be exposed by excavations for the roadway. Along with the subsidence condition, local dip of the mine and the presence of water which could produce acid-mine drainage were critical to evaluation. Coal exposures had to be drained if water was present, and large openings had to be closed to prevent unauthorized and unsafe entry into the workings. Above-grade workings had been successfully addressed in several previous PTC projects, so special provisions for construction and treatment details were provided.

Coal mines below grade were again classified by depth. Those from 0 to 50 feet below grade were to be treated to ensure 100 percent support. Below 50 feet, treatment was to be provided on a case-by-case basis. At depths below 100 feet, only cases where the consultant "strongly believes" treatment to be necessary would be considered.

The area to receive treatment was more difficult to define, given consideration for both roadways on embankment and roadways in cut. For embankment areas, the treatment was to be within an area defined by the greater of the original right-of-way width or 100 feet horizontally beyond the outside shoulders. In cut areas, the treatment area was defined by projection of a 15 degree angle from the bottom of cut to the coal seam. Other widths would be considered where the consultant "strongly believes" treatment is necessary. In all cases, treatment was to be limited to over-excavation and backfill.

Final Guidelines for Roadways - As with the structure guidelines, the roadway guidelines were modified during the design process. For roadways, the type of treatment became more flexible.

If the depth to the mine was less than 50 feet below road grade, a deep-mine undercut was performed, similar to that for structures. This involved the excavation of overburden materials and inspection of the exposed mine. The removal of the coal mine was not required if all voids were removed or fully collapsed during excavation. Intact coal typically was only removed if it was within five feet of the final subgrade elevation. A blanket of rock was placed at the bottom for drainage, and the excavation was then backfilled with normal embankment material.

In areas where the depth to the mine was greater than 50 feet below the road grade, nothing was done during the earthwork stage of construction to stabilize the mine . However, in some areas where the depth to mine was greater than 50 feet below grade, flexible pavements have been used on the Mon-Fayette Expressway.

Original Guidelines for Active Workings - Originally, guidelines for active workings only addressed support issues. The guidelines just applied the same criteria presented for roadways and structures for defining the minimum extent where coal removal should be limited.

Final Guidelines for Active Workings - Much more design coordination with mine operators was required than was originally envisioned. In addition to limiting coal extraction locally, surface activities were affected. These activities, performed under State and Federal permits, included coal haulage and disposal. Impacts to these operations required not just developing new designs, but also filing modifications to the permits.

One particular activity addressed during was the construction of the roadway through an existing and active fine-coal refuse slurry pond. The slurry pond consisted of clay-sized particles of coal, shale and sandstone that were the byproduct of a coal preparation plant. The original roadway design required the construction of a temporary dike through the pond and the removal of the fine coal refuse to construct the proposed roadway embankment and a new slurry pond dam. In addition to the analyses to satisfy the PTC's stability concerns, analyses were needed to satisfy the concerns of the mining operator, Pennsylvania Department of Environmental Protection, and MSHA. The design had to address the potential loss of waste storage capacity, by increasing the total slurry pond height to offset the areal loss.

Original Guidelines for Secondary Impacts - Secondary impacts of underground mining of concern during design were mine fires and acid mine drainage (AMD). The primary principle was to avoid or minimize contact with either of these. No mine fires were anticipated to be encountered. Where AMD was to be encountered, the guideline was "don't make it any worse than existing conditions." If necessary, the designer was directed to provide special drainage pipes for acid environments. Provisions were made to monitor and document water flow and quality before, during and after construction to verify the guideline was met. Short-term mine water discharges during construction were handled through a special provision which called for treatment (if necessary) and controlled release of this water.

Final Guidelines for Secondary Impacts - Handling of AMD was not modified significantly during design. Coal seams in roadway excavations above grade were to be treated by excavating benches and backfilling to the original slope line with durable rock. The benches were graded to drain any water into ditches or roadway drainage. Mine water anticipated to be encountered at or below grade was assumed to be handled by drainage systems installed for groundwater control.

MODIFICATIONS THROUGH CONSTRUCTION

Application of the Guidelines for Structures - The use of deep mine undercuts provided the most sound and conservative means of stabilization for bridge foundation construction. When the coal and overburden materials were removed, conditions could be visually inspected to ensure proper removal of all unstable foundation material prior to backfill. The major concern with the method was the potential for settlement of backfill materials, especially where additional embankment was to be placed such as in the abutment areas of the bridges. This was handled through proper construction control.

The use of subsurface stabilization grouting plans was considerably more difficult and costly. One reason is that the mine conditions could never be visually inspected during construction. In preparation for the grouting plan, the inferred location of the deep mine from local mine maps typically was plotted on the foundation plans and then a grouting plan was designed to stabilize the foundation footprint for the substructure units. Although the mine maps were fairly accurate, they were not always plotted accurately on the foundation plans. This was typically due to the lack of reliable surface references on the mine maps. The designs that used these maps to produce grids of grout holes for the location of mine voids and subsurface subsidence fractures upon actual drilling sometimes found predominantly intact coal instead of open-mine conditions. This ultimately made it more costly to locate subsurface voids and fractures for stabilization, by the addition of grout holes. It was determined in the field that use of primary grid patterns in the influence area of the foundation footprint should be completed without reference to the provided mine maps and then conditions analyzed and treated by performing secondary or offset holes in areas of intact coal.

Additional problems associated with the subsurface stabilization plans were the subsidence conditions encountered. Generally, the mines encountered during construction were either collapsed (fully subsided) or apparently had been backfilled during mining operations. Most were completely flooded with water. Only the recently abandoned portions of active mines and main haulage ways of the older mines were found to be open voids.

Many of the designs required the use of low strength concrete grout mixes to be injected in areas of voids. The review of drillers' grout hole logs often indicated larger voids (6 to 10 feet) than those encountered during performance drilling (1 to 3 feet). Use of concrete grout in these smaller voids resulted in poor grout takes and thus poor subsurface stabilization. The use of flowable fill grout without concrete with an 11-inch slump proved to be the best method of ensuring grout take in the smaller voids and fractures, and only use of a stiffer grout with a 5 to7 inch slump or concrete grout to construct cutoff walls or curtains to prevent flow of less stiff grout from the influence areas.

Application of the Guidelines for Roadways - Most of the time, the guidelines and special provisions for the handling of underground mines worked well in roadway areas. In some locations, such as under small ancillary structures, it was required to remove all the coal in the roadway template. In other locations, it was decided to leave the coal in place and collapse the voids. This was done where an abundant amount of water was discovered at the mine level.

Application of the Guidelines for Active Mines - Contractors had to coordinate with mine operators for sequencing and scheduling of blasting and other activities with impacts on mine operations. Designs related to specific surface impacts had to be adjusted to account for changes in mine operations from the time of design.

For example, during the time from design to construction, the elevation of the fine-coal refuse slurry pond discussed above increased by over 8 feet. This presented a marked change in the schedule of construction as well as the safety of the construction of the temporary dike. The contractor proposed to stabilize the refuse in-place utilizing soil mixing. This involved the mixing of a high-early strength grout with the refuse to harden it to a consistency appropriate for foundation of roadway-embankment and slurry-dam construction. This work was performed using specially adapted tools affixed to an excavator and a crane-supported mixing apparatus. Depths of stabilized slurry ranged from 0 to greater than 50 feet. Design criteria required an unconfined compressive strength of 100 psi. In addition to the revised design, another permit modification was required to be prepared and submitted on behalf of the mine operator.

Application of the Guidelines for Secondary Impacts - During excavation of the largest deep mine undercut on the Mon-Fayette Expressway, a mine fire was encountered. The mine fire was discovered during the completion of blasting boreholes. The PTC and design engineers were notified of the mine fire, and a mine fire expert was consulted. Once the fire was delineated by a series of test holes, the contractor was directed to construct two parallel cut-off trenches along the edges of the undercut to prevent the advancement of the fire under the areas adjacent to the roadway and to excavate the heated and burning coal. The trenches had to be advanced beyond the excavation of the mine fire 'hot' materials. This change in the undercut excavation disrupted the contractor's planned method for constructing the undercut. The mine fire 'hot' materials had to be excavated and hauled to cooling pits outside the roadway limits, where they were air cooled prior to burial. Blasting agents also had to be modified for higher temperature use.

During construction, field personnel had flexibility to determine the optimum locations for out-letting any mine water encountered during excavations. Where significant amounts of mine water were encountered below grade, subsurface drainage wicks were constructed to either outlet water into existing drainage structures or through rock-fill galleries. In addition to the independent water monitoring programs, water was tested for pH and iron content at the location where it would enter the surface waters at the time of initial discharge to verify no special treatment was required.

CONCLUSIONS

At first during construction, design specifications were enforced as an attempt to provide the best product possible. However, it was learned early in construction that some of the proposed treatments were not practical to construct. As time progressed and the subsurface conditions were better understood, it also was determined that certain required specifications could be modified to enhance their utility. Specifications were modified through consultation with the PTC, designer engineers, and design and construction managers to allow more discretion during the construction activities. The construction manager was given the flexibility to change construction treatments with the variations in conditions encountered. The modified specifications for grouting provided the ability to adjust mix designs, grouting patterns, and scheduling of production. Modified specifications for over-excavation and backfill also allowed the construction manager to adjust limits to reflect mine conditions encountered.

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ROADWAY REMEDIATION – CASE HISTORIES IN KARST

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ABSTRACT

The remediation of roadways overlying karst terrane is not a new subject, but certainly a rapidly growing field as traffic, road construction and awareness increase. Repair of real or potential sinkholes in roadway areas is generally composed of two parts art and one part science. A number of solutions have been employed; excavation and backfilling with earth materials (with or without geotextiles), "dynamic destruction" (i.e., dynamic compaction or ground modification techniques), "sinkhole mixes" of grout placed in washed sinkhole "throats" and excavated areas, and most often, grouting.

Excavation to a clean rock opening (throat) is often the most economical and is many times the procedure of choice. Problems have occurred as a result of deeper (hence wider) excavations than anticipated as well as the use of improper fill materials and their placement. Ground modification procedures have been used and are often successful. Usually, excavation and dental concrete as well as significant regrading are necessary adjuncts to "dynamic destruction".

Generally, some form of grouting, compaction or slurry, is the most prevalent and successful procedure in the authors' experience. However, grouting should not be a prescriptive placement of cement, sand and water in an unwavering, predetermined pattern of holes. Several case histories using various remedial methods within folded carbonate rocks will be discussed, as well as the reasons for their effectiveness (or failure).

INTRODUCTION

Whether it is a growing awareness or increased construction (and/or traffic), the amount of remediation within roadways atop karstic areas is certainly on the increase. Unfortunately, sometimes remediation consists of pouring transit mix concrete into a sinkhole and hoping that it works. Often this remedial measure fails and the sinkhole reopens at the same or a nearby location. We can easily blame this upon the vagaries of karst, but hopefully it is possible to reduce the number of failures by attempting to combine our knowledge of the local subsurface with an understanding of the available and suitable remedial measures for the presumed subsurface conditions.

Often, the engineering geologist/geological engineer is asked to make an assessment of the problem and immediately provide an accurate cost estimate and/or start repairs. Obviously this is not fair, but who wants the roadway closed for a week or watch the next day's rain further destabilize the already questionable bridge foundation.

Direct information on the subsurface, such as well-defined boring or test pit data, would obviously be useful. As a minimum, an understanding of the type of carbonate underlying the area of concern should be attempted. Is it above the weak, geologically recent sediments of, for example, Florida; or the hard Mesozoic and Paleozoic limestones and dolomites of the interior and more northerly United States; or are wc atop metamorphosed carbonates (e.g., solutioned marbles)? Is the bedding flat-lying or folded (hence tilted and likely faulted as well)? And of course, economics generally plays a major role.

FAILURE MODES

What type of sinkhole is it? Was it even a sinkhole or is it a backfilled debris pit? The debris pit possibility can normally be evaluated relatively easily and usually correctly, except of course in obtaining reimbursement from the responsible contractor.

Failure of the soft recent sediments of, for example Florida, was characterized by Chen and Beck, 1989 as:

A. Solution sinkholes;
B. Subsidence sinkholes; or
C. Cave collapse sinkholes.

Near vertical solution features within the limerock allow the generally granular overburden to erode/fall into underlying solutioned limerock channels, forming surface depressions of varying sizes. A more catastrophic failure condition can occur if an overlying cave roof fails into a large solution cavity as shown on Figure 1. Such failures often occur in areas of flat-lying harder carbonates such as those found near Mammoth Cave, KY.

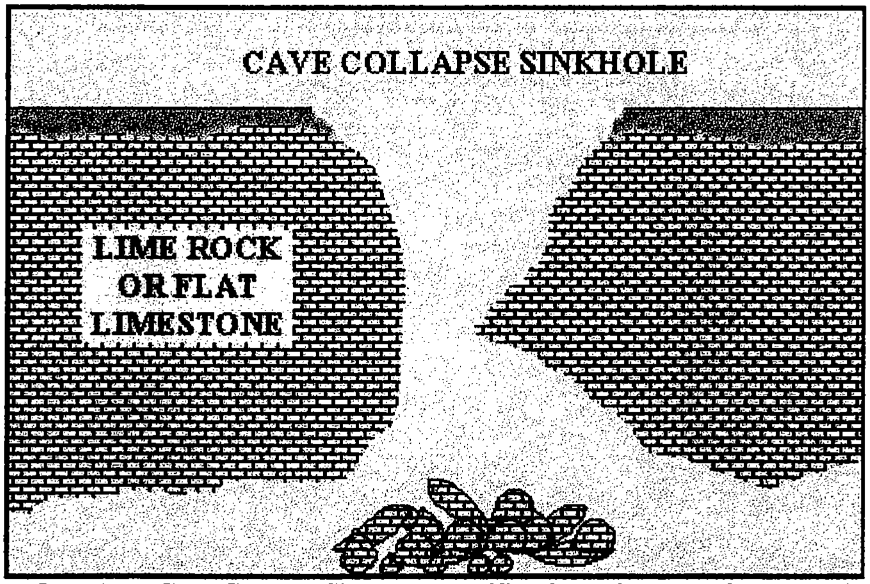

FIGURE 1 – Cave Collapse Sinkhole (modified from Chen and Beck, 1989).

The major dissimilarities to be considered are the strength between the different types of carbonates and variations in the potential effects from contaminant movement into a sinkhole. In evaluating cave collapse features, the strength of the rock, the size of the cavity and the potential for present and future contamination movement into water supplies are of prime significance. Figure 1 types of failures can occur in weak or strong rocks, are common in flat-lying solutioned carbonate rock, but are far less common in folded, faulted and tilted rock terrane.

The more common failures in such "bent" rock essentially start with a soil arch formed by the erosion of the overburden soils into underlying solutioned carbonates or marbles. This creates more of a soil mechanics failure than a rock mechanics (cave collapse) phenomenon. An example of a typical Ridge and Valley sinkhole is shown on Figure 2.

This kind of sinkhole occurrence can be more complex than the cave collapse type. One must be aware of the vagaries of contamination movement along bedding planes; through fractures and shear zones; within or above the water table; as well as the variations in soil and rock properties across strike (exposing numerous different beds, each with their own penchant for weathering/solutioning).

It would be useful to have information from a well-designed geotechnical investigation, but often this is not the case. Usually, it is just select a solution, arrange for the repairs, and of course, do it economically and within budget.

REMEDIATION

After ensuring that it is truly a sinkhole, one must evaluate the nature of the underlying materials, observe the size of the sinkhole and consider the expected rock depth. The next questions to be answered are more from an engineering standpoint. What are the loads imposed upon the overburden soils or underlying rock? What is the importance of, and/or what are the safety concerns for, the facility/structure in peril? From a practical standpoint, can the owner or agency afford the remediation?

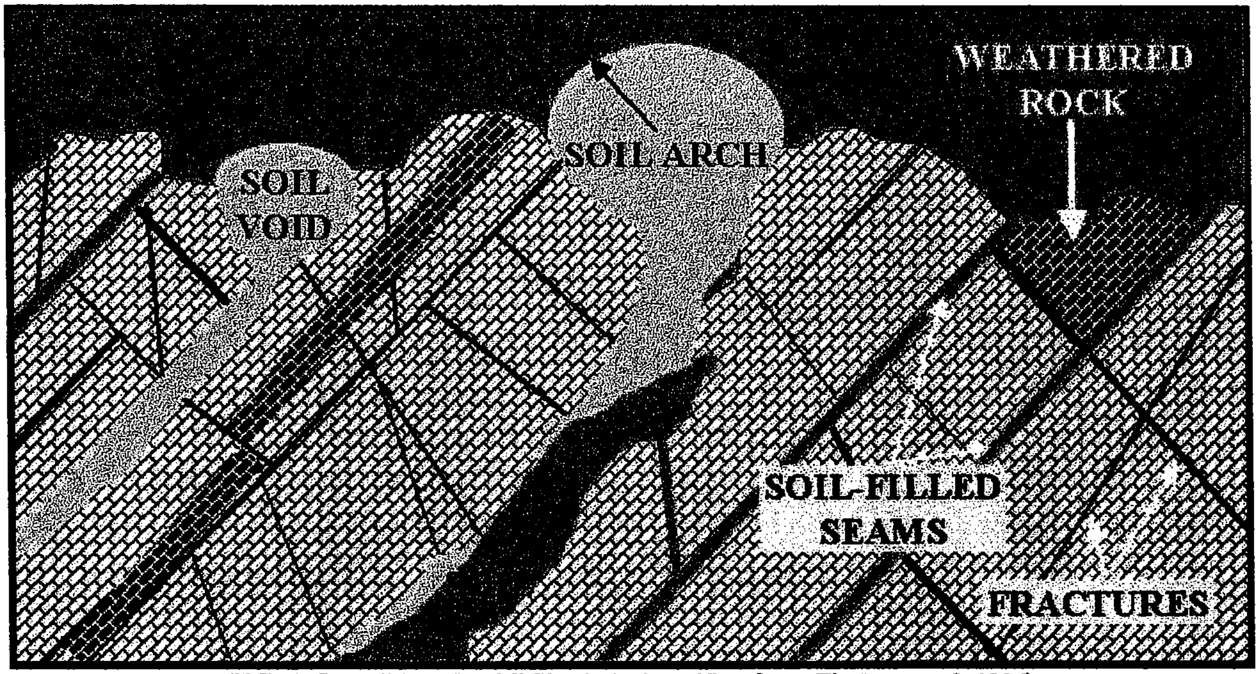

FIGURE 2 – "Bent Rock" Sinkhole (modified from Fischer, et al, 1996).

Before-the-fact remediation can be accomplished by a variety of schemes:

1. Move the structure/facility/roadway or do not build.
2. Piles or caissons to sound rock.
3. Excavation to sound materials and filling the rock cavity and excavation with cement, rock, fly ash or a combination thereof.
4. Grouting (slurry and compaction grouting are the most common).
5. Dynamic compaction (or dynamic destruction).

Post construction remediation can usually be accomplished in several ways:

6. Excavation and backfill.
7. Underpinning.
8. Grouting (probably the most commonly used procedure).
9. Pouring transit-mix concrete down the sinkhole.

The following is a brief description of the remedial efforts described above using the same numbering system:

1. Relocation of the route to an area of better subsurface conditions can be the best response to karst concerns.

2. Piles or caissons installed to (or generally into) sound rock is one of the more common foundation solutions for large and/or heavily loaded structures such as bridges. Often, probe holes are planned for each column location. Pipe piles or caissons have the best chance of being founded on sound rock as a result of the ability to see "down the hole" before concrete is placed. Unfortunately, H-piles have been used as a result of their apparent cost advantages and flexibility in installation lengths. However, one has only to visualize the various lengths and configurations a flexible H-pile can achieve upon meeting with pinnacled limestone to understand the limitations and concerns that arise with the use of H-piles in karst.

3 & 6. Excavation to sound material can often be an economical procedure. However, without a knowledge of the depth to sound rock (which is usually highly variable) before starting excavation, the extent and ensuing costs can vary significantly. For example, a sinkhole area on a state highway in Maryland was first excavated to 6 or 7 meters and

repaired by excavation and backfilling (Martin, 1995). After additional maintenance and investigation, the excavation continued to 22 meters before a final repair was effected. Commonly, this procedure is chosen for its probable speed in achieving a repair and for its apparent economics. Often, excavation to sound materials with some form of controlled backfill works well and effectively. However, time pressures for effecting a repair, for example on an existing roadway, can result in significant costs. In the Maryland highway case previously noted, repairs cost some $700,000.

Backfill can be impermeable (e.g., cohesive soils and concrete) or a graded filter that allows the passage of water into the subsurface. A combination of large rock fill and an impermeable cover has also been used. Again, the nature of the overlying construction, the degree of conservatism required and the local environmental concerns all must be considered.

4 & 8. Grouting is often the procedure of choice because of the ability to work within the roadway right-of-way (even after a failure) or close to structures, and its relatively non-intrusive nature. Controlling grout flow by using thick mixes or quick-setting agents can maintain a limited area of repair. Compaction grouting can densify weak granular soils, fill voids or compress soft cohesive materials where appropriate. Larger voids can be filled with lower cost aggregate or transit mix grout. On-site batch plants can be used to place large quantities of a variable-mix grout where desirable, when grouting can be planned. Air- or hydro-track drilling of grout holes can provide useful, reasonably economical subsurface data if drilled by knowledgeable technicians and monitored by an engineering geologist/geological engineer with experience at karst sites.

5. Dynamic compaction, the dropping of large weights from great heights, can be most effective in remediating large areas such as stretches of roadway. The term "dynamic compaction" is likely still appropriate for southern limestone areas where the overburden and limerock is granular in nature. The term "dynamic destruction" seems to better fit areas underlain by cohesive residual soils that generally overlie older, harder (Proterozoic, Cambrian and Paleozoic) rocks. Dynamic destruction can collapse near-surface cavities or soil voids; seal underlying rock cavity openings with low permeability overburden soils, as well as densify both granular and cohesive soils. In addition, shallow rock voids may be exposed by these procedures and remediated by backfilling, usually with an appropriate cement slurry or lean grout. Areas of deeper soft soils may be identified and remediated by an exploratory grouting program. Of course, the area remediated looks like a World War I battlefield when completed. Backfilling and the use of borrow fill materials will likely be required to bring the area of concern back to grade.

7. The underpinning of structures can be used if the subsurface conditions are known prior to selecting the appropriate means (e.g., pin piles, caissons, pipe piles, etc.). The performance of any underpinning operation will require a relatively detailed knowledge of the subsurface, experienced designers, significant construction time, and likely will be quite expensive.

9. Placing transit mix grout or concrete in a sinkhole is often the first thought when an owner or constructor finds a sinkhole after a rainy night. Sometimes this works, but often the sinkhole is merely relocated a few feet away and rediscovered after the next heavy rain. Commonly, the wrong grout mix is used. The mix is too thick or coarse to flow through the existing subsurface channels; there is too little cement to bond the aggregate and reduce permeability and erosion; or it does not have a shrinkage-limiting agent added. While a quick fix, most "sinkhole mixes" (or alternately, flowable fills) do not work well. It is possible to improve the effectiveness of the quick fix by using the appropriate proportions of cement, water, fine aggregate (such as mason sand), combined with an appropriate anti-shrinkage agent. However, unless a definitive "throat" leading to the rock cavity can be sufficiently opened (usually by washing), even an appropriate flowable fill will merely fill the soil void, not the causative cavity. Even with the best of transit mix grouts, the chance of a sinkhole reopening in the future is fair to good in the authors' experience.

Examples for several of these remedial procedures are subsequently discussed.

CASE HISTORIES

WYNDHAM FARM BOULEVARD

Wyndham Farms is a 242-acre, 240-lot residential subdivision located in one of northwestern New Jersey's carbonate valleys. Wyndham Farm Boulevard is the major (two-lane wide) north-south trending roadway connecting all of the sections of the development with Greenwich Street, County Route 636, which bisects the site and provides the primary access to the development. In early January, 1999 the subgrade, curbing and utility installation along Wyndham Farm Boulevard had been completed to provide primary access to the last two northerly sections of the development to be constructed. While waiting for good weather to install the 4-inch asphalt base course, a January "thaw" occurred that caused rapid snowmelt followed by heavy rain. Without the asphalt base course, runoff water could not enter the catch basins of the storm water drainage system and large amounts of water backed-up and "ponded" in and around the roadway adjacent to the catch basins. In addition, poorly compacted utility trench backfill settled, ponding water alongside the roadway. As a result of the ponded water conditions, sinkholes developed in Wyndham Farm Boulevard at the location of a primary storm water line crossing. In addition, a catch basin within a drainage swale just to the east of the roadway also blocked-up and another "pond" developed which caused additional sinkholes to form adjacent to the roadway and resulted in "piping" soil from around the storm water drainage line. The sinkholes forced the closure of Wyndham Farm Boulevard for a period of approximately two months and resulted in extensive delays to construction in the last two sections of the development.

Physiographically, the site lies in one of the intermontane valleys of the New Jersey Highlands, just to the south of the Kittatinny Valley (in the Valley and Ridge Physiographic Province). The rocks that underlie the site are part of the Kittatinny Supergroup and the overlying Jacksonburg Limestone. The formations existing below the site include the dolomites of the Allentown Formation and Beekmantown Group (or Epler and Rickenbach Formations) and the overlying limestone of the Jacksonburg formation. The oldest formation is the Allentown Formation at the northern end of the site with the youngest, the Jacksonburg Limestone, at the south end of the site. Beekmantown Group sediments underlie the central portion of the site. Most of the site lies upon an up-thrust block bounded by the Brass Castle Fault that outcrops within the Rickenbach Formation in the northerly portion of the site. The carbonate rocks found underlying Wyndham Farms have been solutioned. A preliminary investigation consisting of seven test borings and 29 test pits was conducted prior to site planning (Fischer and Fischer, 1997). A pinnacled rock surface was found underlying the overburden soils over most of the site. The overburden was mostly glacial tills overlying residual soils with minor amounts of topsoil and fill. Wyndham Farm Boulevard traverses the full geologic section of the site, lying approximately perpendicular to strike.

The location of the catch basins and storm water drainage line affected by the sinkholes discussed herein are just to the east of a major low gradient drainage swale and directly atop the trace of the Brass Castle Fault. Within the drainage swale and just to the west of Wyndham Farm Boulevard, a deep (20 foot) sanitary sewer line was installed in a blasted rock trench. We believe that the extensive vibrations and gas pressures induced within the fractured dolomite by the uncontrolled blasting operations, coupled with the ponded runoff water, was the cause of numerous sinkhole occurrences at this location. As a result of the eventual loss of ground beneath the roadway, the two catch basins, much of the storm water system piping and curbing had to be removed and reconstructed after remediation.

The initial sinkholes at the roadway catch basins were quite small with no defined "throats". A program of drilled probes with site-mixed slurry grouting was initiated to attempt to remediate these sinkholes. As the rotary drilled probe and slurry grouting program got underway, uncontrolled surface runoff water continued to flow into the subgrade. A large linear sinkhole, 20 feet wide and across most of the width of the roadway, then opened up just to the south (up-gradient) of the catch basin/storm water line which undermined the catch basins and pipeline and forced their removal. The resulting void was excavated and filled with water to try to locate any sinkhole "throats". No significant "throats" were found and the void was filled with 10 cubic yards (CY) of transit-mixed grout and then up to subgrade with compacted dense graded aggregate. Another sinkhole formed within the roadway approximately 100 feet to the south of the catch basins and storm water line. Again clean out and filling with water failed to locate a throat. The sinkhole surface void was filled with site-mixed grout and a small west/northwesterly oriented "throat" was found during these operations. This small sinkhole eventually took more than 10 CY of site-mixed grout to fill. Pneumatic percussion (air-track) probes were drilled surrounding the sinkholes and the probe holes slurry-grouted with site-mixed grout. The remediation of this area of Wyndham Farm Boulevard and surrounding areas eventually included drilling 135 probes and jetting 6 additional probes into small sinkholes. A total of 350 CY of grout (325 CY of site-mixed grout and 25 cubic yards of transit mixed grout) was utilized to fill voids in this area.

Site-mixed slurry grout was placed in the drilled or jetted probes. The grout was pumped through "tremie" pipes placed in the probes after drilling. As the holes filled, the "tremie" pipes were pulled in increments to prevent pressure build up in the grout. This procedure is effective in sinkholes with small "throats" or openings into and within the rock allowing the grout to flow into and seal unseen small cracks or open joints to prevent future sinkhole development. Site-mixed grout (varying mixes of cement, mason sand, water and bentonite) was chosen for the ability to easily vary the grout mix to suit the nature of the subsurface in the probe hole being grouted. The uncontrolled rock blasting operations utilized in trench excavation for utilities and foundation excavations combined with a lack of surface grading and drainage during construction at Wyndham Farm Boulevard and many other areas of the site resulted in sinkhole remediation operations at the development continuing through July, 2000, more than 1½ years of continuous operation. Nearly 3,000 CY of grout was placed throughout the site area (Ottoson, et al, 2001).

The sinkhole remediation of this most critical section of Wyndham Farm Boulevard has now been completed for approximately one and a half years. A severe hurricane (Floyd) with extensive rainfall and local flooding occurred in the fall of 1999. Sinkholes have not redeveloped in the remeditated areas as of early 2001.

MEDFORD ROAD

This wide, two-lane county roadway crosses a variety of subsurface materials in the Wakefield Valley of Carroll County, Maryland. The area of concern crossed the Precambrian Wakefield Marble Member of the Sams Creek Formation. The Wakefield Marble in the site locale is a light colored rock with numerous multi-colored bands containing layers of calcite and dolomite. It is often in fault contact with the metabasalts or phyllites of the Sams Creek Formation in a metamorphosed zone of brecciation. Solutioning was apparently concentrated along fractures. Solution features range in size from tiny vugs to long, linear features, to actual caves. A semi-linear "row" of preexisting and potential sinkholes could be observed in aerial photographs of the locale. The roadway crossed this suspect trend.

Test borings, pneumatic percussion probes and geophysical data were available for the roadway right-of-way. However, little information on the overburden materials was available prior to construction as these initial investigations concentrated on coring the rock. The geophysical (electromagnetic) data proved to be of little use in the overall interpretation of the available subsurface data. Moreover, the limit of the Wakefield Marble was not well-defined although the available subsurface information unequivocally indicated the solutioned nature of the formation below the area.

The remedial procedure of choice was "deep dynamic compaction" even though the overburden soils were cohesive in nature. It was felt that this brute force method would both proof test the area of concern by exposing areas of soft soil or collapsing voids or rock cavities and force the clayey overburden soils into any smaller rock voids. This would hopefully seal them to prevent the future erosion of soils into solutioned cavities within the marble. The dynamic compaction (or perhaps more descriptively, dynamic destruction) approach was selected not as a guaranteed *cure-all*, but as an economical and practical approach to ameliorate the potential hazard with the proviso that the roadway should be periodically monitored to check the long-term efficacy of the procedure. Proof rolling with heavy construction equipment was planned for areas of deeper rock where no definitive geotechnical information was available in an attempt to define whether sinkhole formation was likely in the near future. Upon the basis of the available geotechnical information, the use of a geotextile was recommended in several areas as a belt and suspenders conservatism (Fischer and Fischer, 1995).

The work was performed in 1990 in accordance with the planned treatment. The dynamic destruction operations appeared to be quite effective, locating numerous suspect areas and compacting the surface soils. A large weight (18.5 tons) was dropped some 60 to 70 feet yielding a total energy of some two million foot pounds. A nominal 15-foot square pattern with three drops per location was established as the planned procedure. Where softer areas were revealed by the planned operations intermediate drops were made and often, an increased number of drops were accomplished. Consequent repairs in the softer areas consisted of excavation to rock with the placement of lean concrete into exposed voids and the use of compacted stone fill in the drop zones. As the finished section looked like a World War I battlefield, a large quantity of fill was required to bring the roadway area to grade. The area requiring the most repairs was that suggested by the aerial photographic review.

The remediated roadway area has been monitored since 1990. A suspect area (as possibly defined by minor subsidence) has been observed just beyond the end of the geotextile-supported base area, but within what was originally the most suspect location. Several test borings were drilled within this possible settlement area. Unfortunately, rock was not cored so the cavity potential is not fully known. One of the borings encountered a softer soil zone between 25 and 35 feet below grade where the auger met refusal. Rock was encountered within the other three borings at depths varying from about 25 to 36 feet below grade. One of the original (1988) borings encountered rock at 48 feet below what was pre-roadway grade, and a six inch void was encountered just below the surface.

At the time of the completion of this paper, a contract for a remedial exploration/grouting program was being let. However, some 10 years of satisfactory roadway performance has been achieved at a relatively low cost.

PENNSYLVANIA TURNPIKE SERVICE AREA

The site area lies in a carbonate floored valley that extends from the vicinity of Lancaster, PA to the eastern edge of Montgomery County, just to the west of Philadelphia, PA. The valley itself is an inlier of folded and faulted Paleozoic-age sedimentary rocks in the Piedmont Province of Pennsylvania. The valley is underlain by Cambrian and Ordovician limestone and dolomite, and by Cambrian quartzite and quartz schist of the Lebanon Valley sequence (Berg, et al, 1908). The susceptibility to chemical erosion of the carbonate rocks is responsible for the subdued topography of the valley. The hills to the north of the site locale are cored by Precambrian crystalline rocks (at depth) and by quartzite and schist. The service plaza is on a fault-bounded block of Ledger Formation dolomite.

This formation is described by the Pennsylvania Geological Survey as: "moderately well-bedded; massive, light gray, locally mottled, coarsely crystalline dolomite, cherty, with siliceous beds. Joints have a blocky pattern and are moderately well developed; moderately abundant; irregularly spaced, having a wide distance between fractures; open and steeply dipping."

A geotechnical study was performed after a sinkhole (or sinkholes) occurred at a fuel storage area adjacent to the service area. The study included a review of aerial photography, data from previous investigations within the service area, and three rotary wash test borings using techniques designed for investigation in carbonate rock areas. (Fischer and Canace, 1989).

This study revealed that in this area the bedrock had been subject to fracturing and thus increased solutioning in the past. Numerous open channels and soil-filled cavities were noted during the drilling and coring operations. Rock depths varied from 35 to 65 feet below grade in a ±50 foot square area. On the basis of the recovered core, a small fault was believed to trend through the westerly portion of the proposed tank site.

Prior to the initiation of remedial work at the site, sheet piles were installed in a 41- by 36-foot rectangular area just outside the proposed tank locations. The existing tanks were removed and the area backfilled with 3/8-inch pea gravel. Both the placement of the stone backfill and site work being performed on nearby storm sewers complicated drilling and grouting operations at the tank site. The pea gravel backfill resulted in changes in the drilling and grouting procedures and increased costs as a result. It was originally planned to use one rotary-wash drilling rig and one air percussion (air-track) rig for the drilling operations. However, as result of the placement of the gravel backfill prior to and during the beginning of the drilling and grouting operations, it was necessary to install drill casing to limit grout flow into the stone, a function that is difficult and slow for an air-track. As a result, two conventional truck-mounted drilling rigs were used. In addition, owing to the weight of the truck-mounted drilling equipment and the sloped nature of the incomplete stone fill, it was necessary to occasionally employ heavy site equipment to facilitate the movements of the drilling rigs.

A primary grout hole pattern grid of 10 feet was established for the slurry grouting operations. Intermediate secondary grout holes exhibited little grout take indicating the effectiveness of the lean cement, mason sand and water grout mix used. A total of 57 cubic yards of grout were placed within the 41- by 36-foot fuel storage area.

As predicted, the completion of a grout wall with internal cavity filling moved the formation of sinkholes to the parking area outside the remediated tank area. Eventually, repair of these areas through slurry grouting procedures and the repair of leaking water line finished the remedial work.

CONCLUSIONS

Remediation of highway facilities should almost be a given in some karst terranes. Remediation before construction is always more economical than resolving post-construction problems. Appropriate remediation techniques at karst sites are a challenge. Many procedures are available, but all depend upon having or developing an understanding of the subsurface, the nature of the structure or facility of concern, the degree of conservatism desired, schedule requirements, and economics. One must be aware of the potential consequences of remediating one area and forcing the sinkhole to move laterally unless the entire causative subsurface cavity (or cavities) is filled or completely sealed.

Even with a detailed subsurface investigation (science), the variations existing below most karst sites required a great deal of extrapolation (art) and guesstimation (sometimes referred to as witchcraft). Then the reality of economics often necessitates changes and/or limitations to the best of investigatory and remedial programs. Without some knowledge of the actual subsurface conditions, remediation procedures can be an expensive shot in the dark.

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TWO APPROACHES TO OVERBURDEN STABILIZATION WITH LIMITED RIGHT-OF-WAY, I-87/287, NEW YORK STATE THRUWAY, NYACK, NEW YORK

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ABSTRACT

As part of an on-going rock slope remediation effort intended to improve public safety, the New York State Thruway Authority (Thruway) designed a rock excavation project to widen the roadway shoulder and catchment ditch in the southbound lanes at Milepost 18.3 on the New York State Thruway (Interstate 87/287), Nyack, New York. The Thruway owns a narrow easement at the top of the 30 m high rock slope, with private residences on adjacent properties. The narrow easement, combined with a narrow catchment ditch and shoulder along the heavily traveled highway, resulted in severe access restrictions in design investigation and construction. Excavation was to be completed top down, with a coordinated effort to limit lane closures to 20 minutes during blasting. During excavation, two areas of greater than anticipated overburden thickness were encountered above bedrock, varying from 1 m to 4 m and from 3 m to 7 m, respectively. It was not possible to lay the soil slopes back to the design angle due to the limited right-of-way and high property costs. Soil stabilization measures were required.

In those areas where overburden excavation at near vertical slopes was required, a robust system of soil nails and fiber reinforced shotcrete facing was employed, with a total design and construction period of five weeks. Construction was performed from the top of the slope "on rappel", while site blasting and excavation continued below. Soil nailing increased global stability and protected the steepened overburden from future erosion. Only subsurface easements were needed for nails extending beyond the Thruway's right-of-way.

In those areas where the Thruway easement was wide enough to allow some flattening of the overburden slopes, a proprietary system (Pentifix® by Geobrugg®) of soil nails, tensioned in-place wire rope nets, and a vegetated slope face was employed. The Pentifix® system was chosen based on cost comparison with other stabilization designs. This proprietary design has been used extensively in Europe; however this is the first installation of a Pentifix® system in the United States. The system provides increased global and surficial slope stability, erosion control, and the appearance of natural grassed slopes.

INTRODUCTION

Two slope stabilization techniques were employed to allow construction of steep soil slopes within the restricted right-of-way at a rock remediation project on the New York State Thruway in Nyack (Milepost 18.3 on I-87/287):

- Soil nails were used to retain 1 m to 4 m thick overburden soils at the top of a 30 m high, near vertical rock cut; and,
- Geobrugg® netting and soil nails were used to retain a 1H:1V soil slope, ranging from 3 m to 7 m in height.

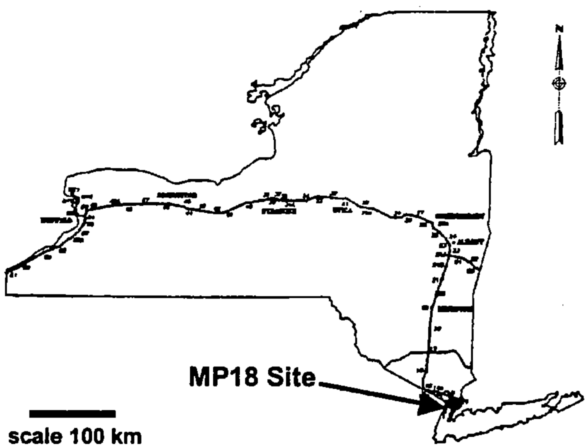

Figure 1. Site location map.

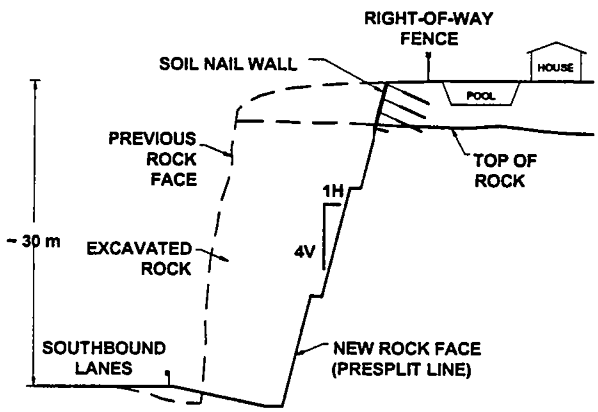

Figure 2. Typical slope section at soil nail wall.

Because of site access limitations, heavy equipment such as drill rigs and excavators could only access the work areas by travelling steep ramps beside the roadway.

The site is located between Mileposts 18.0 and 18.5 Southbound on the New York State Thruway (I-87/287), just west of Nyack in Southeastern New York State (Figure 1). Southbound I-87/287, via the Tappan Zee Bridge over the Hudson River, provides access into the northern New York City metropolitan area. The planned construction, part of the Thruway's ongoing rock remediation and roadway widening program, includes widening of the roadway, creation of a rock catchment ditch and re-cutting existing rock slopes to 1H:4V using presplit blasting techniques. The slope has a maximum height of 30 m and was originally excavated in the 1950's. A typical section, through the highest point of the cut slope, is shown in Figure 2.

The rock remediation design called for excavation of 2H:1V soil slopes approximately 0.5 m high at the crest. No other treatment was anticipated for the assumed overburden thickness. However, at the start of construction, two overburden areas were exposed, ranging from 1 m to 4 m and from 3 m to 7 m thick. The designed 2H:1V slope could not be constructed without significantly encroaching upon adjacent property (Figure 2). In order to maintain soil slope stability and limit encroachment, stabilization techniques were required.

SITE GEOLOGIC SETTING

Bedrock at the site is dark greenish gray, Triassic-aged, coarse-grained diabase (dolerite), intruded as a massive sill (Palisades Sill) between red bed sedimentary rocks. Subvertical and subhorizontal planar cooling joints are common, and mesoscale helical curvilinear cooling joints with slickenslides occur, typical of diabase sills in the area (Faust, 1978). The overburden soil consists of a reddish brown bouldery till composed of clayey, sandy, and gravelly silt, 1 m to 7 m thick overlying the undulating bedrock surface. During construction, most of the overburden and shallow bedrock was dry, with some moist to wet areas noted.

SOIL STABILIZATION TECHNIQUES EMPLOYED

Two methods of soil slope stabilization were employed: soil nailing, and the proprietary Pentifix® system by Geobrugg® (Rüegger and Steiner, 1998). Both systems utilize soil nails to improve the factor of safety against deep seated and surficial soil slope failure, as well as facing elements to stabilize the soil slope and prevent erosion. In general, the primary differences in the two systems are summarized below:

Soil Stabilization Technique Characteristic	Soil Stabilization Technique	
	Soil Nails with Shotcrete Facing	Soil Nails with Wire Rope Facing and Vegetated Slope*
Maximum Slope Angle	Vertical	45° to 60° in soil, up to 80° in rock
Soil Nail Elements	Grouted steel dowels	Grouted steel dowels
Typical Soil Nail Element Spacing	1.5 m x 1.5 m	3.4 m x 3.4 m
Facing Element	Reinforced shotcrete	Tensioned wire rope nets
Final Appearance	Shotcrete facing or architectural finish, such as pre-cast panels	Grassed slope

* - Pentifix® System by Geobrugg®

Soil Nail Wall with Shotcrete Facing

Stabilization of surficial soils was required for an approximately 100 m long portion of the alignment at the crest of the excavation. Several alternatives were considered by the Thruway's engineering team, including gabions, soldier piles, concrete retaining walls, and a soil nail wall. Due to access limitations, heavy equipment could not be brought in across neighboring property, thus construction of a soldier pile wall was deemed not feasible. Concrete or gabion retaining walls would have required significant temporary excavation on neighboring properties, resulting in estimated costs of at least twice that of the soil nail wall option. The Thruway chose the soil nail wall, as it could be installed with light equipment, minimal neighbor access, and was the least expensive. In addition, both soil nail wall construction and rock excavation could proceed simultaneously. Because some of the nails extended beyond the Thruway right-of-way, subsurface easements were required. Golder Associates initiated design on June 7, 2000, and Janod Construction started work on June 21, completing the soil nail wall by July 18, 2000.

Analysis was completed employing the Service Load Design (SLD) approach. In SLD, strength factors are applied to the soil strength, nail tendon strength, nail head strength, and nail pullout resistance, and a factor of safety against failure is computed for various failure modes and load combinations. The Federal Highway Administration (FHWA) approved design procedure and the computer program GoldNail (version 3.11; Golder Associates Inc., 1996) were used for design. The minimum computed factor of safety against rotational failure through the reinforced soil mass for the static and seismic cases exceeded 1.35. The soil nail wall was designed and constructed according to accepted methods (Byrne and others, 1996).

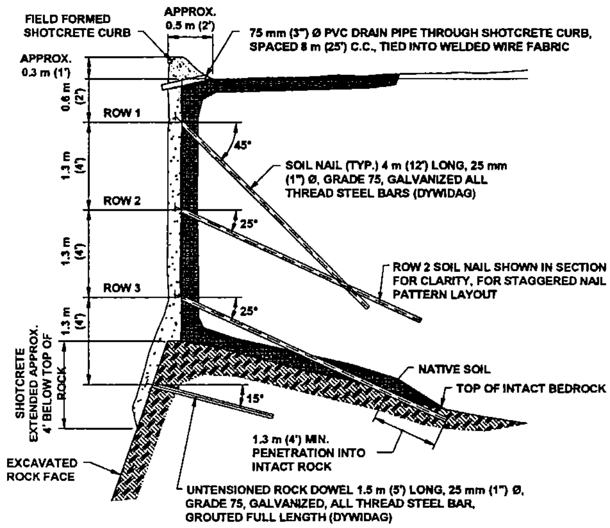

Figure 3. Typical soil nail wall design.

Figure 4. Rock dowel (foreground, on rappel) and soil nail installation (background, drill rig).

A typical section of the soil nail wall is shown in Figure 3. As shown, the primary components of the soil nail wall include 25 mm diameter, Grade 75 Dywidag reinforcing steel threadbar soil nails and 75 mm thick fiber reinforced shotcrete facing. For corrosion protection, the Dywidag bars, face plates and nuts were hot dip galvanized. Soil nail lengths range from 2 m to 4 m. Drill holes were advanced using 76 mm diameter compressed air hand drills and an IR-390 hydraulic track drill (Figure 4). Soil nails and rock dowels were installed at angles (rake) ranging from 15° to 45° below horizontal. The steep rake of the upper rows minimized right-of-way impacts. Upon completion of drilling, nail bars were inserted into the holes, and were then tremie grouted with a sand-cement grout. Two PVC centralizers were used on soil nails 2.7 m or longer, placed 1 m from each end of embedded (grouted) length. The rock dowels were installed in a similar manner.

The support had to be adaptable to varying soil thickness, ranging from 1 m to 4 m. A specialized labor crew, working from a bench left in place at the base of the overburden excavation and suspended by ropes and safety harnesses, installed the support after rock excavation commenced. Hand scaling was conducted to remove loose boulders in the soil as well as loose weathered bedrock.

The soil nail wall facing was generally constructed in the following sequence:

1) Geocomposite drainage strips were placed vertically in direct contact with the exposed soil on 1.5 m centers for the full height of the excavation and extending at least 1.3 m below the soil-rock interface.
2) An initial 75 mm layer of steel fiber reinforced shotcrete was applied to the exposed soil face, using the dry mix process. The initial fiber reinforced shotcrete thickness was

checked by verifying that 75 mm carpentry nails placed into the soil as telltales were completely encased.

3) Welded wire mesh reinforcement was placed over the initial 75 mm layer of shotcrete.

4) Galvanized face hardware was then attached by screwing a galvanized nut onto the threadbar, stopping approximately 75 mm short of the initial shotcrete face. Continuous horizontal waler bars and vertical bearing bars of #4 steel reinforcing bar (rebar) were then placed above, below, and along the side of each row of nails. The soil nail face plate and a second galvanized nut were then secured to each soil nail.

5) Screed wires were run along each row of nails at a minimum distance of 140 mm from the soil surface to facilitate verification of the minimum shotcrete thickness.

6) A final layer of steel fiber reinforced shotcrete was applied until the minimum 140 mm thickness was achieved and the steel reinforcement was completely covered.

7) A continuous drainage curb and PVC drainage pipes at roughly 7.5 m centers were installed at the top of the soil nail wall in order to minimize the likelihood of surface water infiltrating behind the shotcrete and resulting in possible buildup of water pressure and erosion of soil behinds the wall.

Soil nail wall construction inspection and soil nail testing were monitored according to FHWA accepted methods (Porterfield and others, 1994).

Figure 5. Application of initial shotcrete layer over geo-composite drainage strips. Note tarp use to contain rebound.

The geocomposite drainage strips allow water between the soil and shotcrete wall to drain and prevent water pressure buildup behind the shotcrete (Figure 5). The extension allowed for visual verification that strips were not trapped behind shotcrete and allowed water to drain at the wall base. In addition, a horizontal drainage strip in hydraulic connection with the vertical strips was placed in an area with the thickest soil. This horizontal strip is intended to function as secondary relief in the event that one of the vertical strips becomes clogged.

Prior to final shotcrete application, soil nail verification pullout testing was conducted. Typically, verification tests are conducted on sacrificial nails prior to installation of production nails in order to verify that the installation methods and materials employed result in soil nails or rock dowels capable of achieving the specified design capacity. However, because the soil exposure area on the Milepost 18.3 project was limited to the production face, verification testing was run on production nails. In all, two verification tests were performed on the soil nails and two verification tests were performed on rock dowels. Verification tests were conducted to verify that installation methods would provide a soil nail or rock dowel capable of achieving the specified design adhesion capacity with a specified factor of safety. All of the verification test nails were loaded to 200% of the design load. Creep tests were conducted at the highest load of the verification test by holding the load for up to 60 minutes. Because the verification testing on production nails satisfied the specified requirement of proof testing on at least 5% of the production nails, proof testing was not required.

Pentifix® System

At a location west of the soil nail wall, an 80-m long section of the excavation with a thicker overburden zone, ranging in thickness from 3 m to 7 m, was encountered (Figure 6). In this area, the originally designed 2H:1V slope would have, if excavated, extended beyond the Thruway right-of-way. The Thruway choose to install a stabilization system consisting of soil nails and Geobrugg® steel wire rope nets (Pentifix® system). As with the soil nail wall, the Pentifix® system was less costly than a gabion basket wall. Design was completed by Golder Associates in September 2000, and Janod Construction completed the installation between November 13 and December 21, 2000.

Figure 6. Pentifix® slope area after soil/rock removal.

The designed slope for the Pentafix® supported excavation is 1H:1V. Slope stability analysis was performed using SLOPE/W software (Geo-Slope International, Ltd., 1999). A hand calculation using the simple method of slices was used to check the program results. The minimum computed factors of safety against rotational failure of the reinforced soil mass are 1.5 for long term conditions and 1.1 for seismic conditions. Soil nail installation inspection and testing were closely monitored according to FHWA accepted methods (Porterfield and others, 1994).

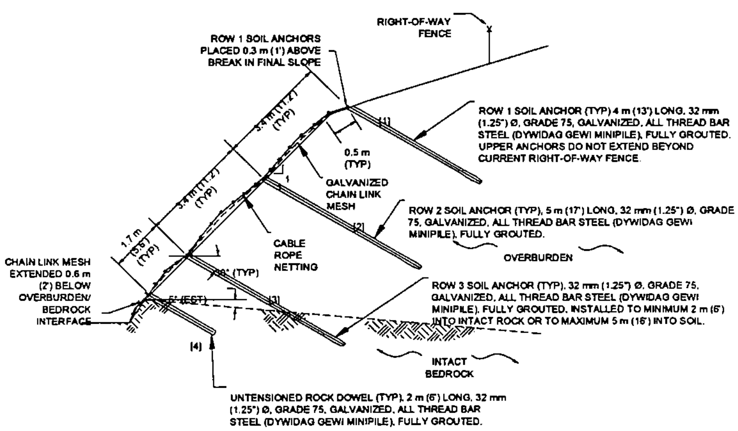

Figure 7. Typical Pentifix® construction design.

Steel wire rope nets, jute netting and galvanized chain link mesh were placed over the excavated soil slope to provide stability of surficial soils and soil erosion protection. Typical construction is shown in Figure 7. Two to four rows of soil nails and rock dowels were installed, depending on the height of the slope, to accommodate one to three rows of wire rope net panels.

The overburden soil consisted of till similar to the overburden in the area of the soil nail wall but with a slightly higher sand content. The till, in general, was moist to wet, requiring the construction of drainage systems. Prior to soil nail installation, and after final 1H:1V slope grading, four counterforte drains were installed at the lowest portions of the soil-rock interface and in areas of observed soil seeps. Drains having a minimum width of

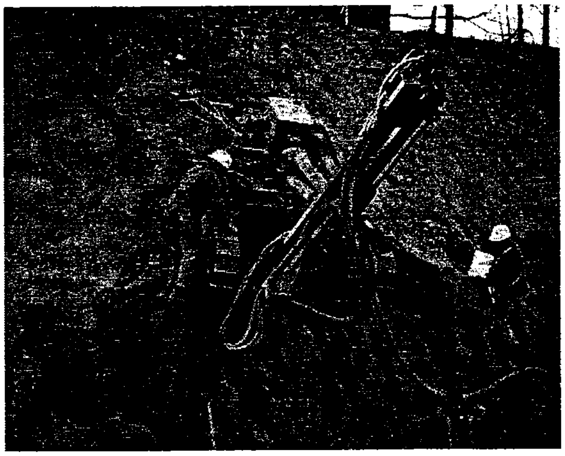

Figure 8. Soil nail and rock dowel installation with light air track drill.

Figure 9. Pentifix® wire rope net installation over chain link mesh and jute netting.

0.75 m were excavated into the final slope, lined with non-woven geotextile, and then backfilled with open graded stone. A coarser stone was used to armor the upper portion of the drain. Following drain installation, soil nails and rock dowels were installed using a light airtrack drill rig suspended from steel cables attached to large trees above the soil slope (Figure 8).

The Pentifix® system soil nails and rock dowels are galvanized 32 mm diameter GEWI Dywidag Grade 75 reinforcing steel threadbar grouted in place. Two to three rows of soil nails were installed at approximately 30° below horizontal. The soil nails ranged in length from 3.6 m to 4.9 m in order to extend beyond the critical circular failure plane. The upper soil nail row was placed 0.3 m above the slope break to provide proper anchorage, and in general, upper soil nail lengths were limited to 4 m to avoid right-of-way encroachment. Due to the wire rope net panel dimensions, soil nails and rock dowels were spaced horizontally and vertically every 3.4 m in a non-staggered pattern. Due to limited allowable flexure of the wire rope net panels, soil nail placement positions were critical, and needed to be placed within 30 cm of the target position.

Figure 10. Pentifix® wire rope lacing. Note use of come-along tool to provide sufficient tension.

After verification and proof testing of representative soil nails, the remainder of the Pentifix® system was installed. Prior to wire rope net installation, the hydroseeded slope was covered with jute netting and 50 mm square galvanized chain link fencing to provide additional erosion control. The wire rope net panels consisted of diagonal galvanized steel wire rope net having a mesh size of 300 mm, and a steel cable rope diameter of 8 mm (Figure 9). To accommodate the undulations in the soil-rock interface, two panel shapes were used: full panels having a dimension of 3.4 m x 3.4 m, and half panels having a dimension of 3.4 m x 1.7 m. Intermediate soil nails and rock dowels were needed in areas requiring half panels. The corners of the wire rope nets were attached to the soil nails via head assemblies with threaded and tensioned steel cable. Following attachment, the wire rope net panels were seamed together with steel wire rope, and tensioned with a light come-along (Figure 10).

While the Pentifix® system has been used for several years in Europe (Rüegger and Steiner, 1998), the system installed at Milepost18 is the first in the United States. Pentifix® system advantages include long-term slope stability, erosion control, and minimal regular maintenance, at costs significantly less than other retaining systems. In addition, vegetative growth will provide an aesthetic appearance to the slope for the motoring public.

CONCLUSIONS

Both the soil nail wall and Pentifix® designs were installed at the Milepost18.3 project in order to provide soil slope stability in areas of very limited right-of-way. These soil stability construction elements allowed for successful completion of the entire rock remediation project within the original construction timeframe at significant cost savings over other retaining systems, and will provide relatively maintenance-free long-term stability. In addition, both systems will be aesthetically pleasing, as the medium gray tone of the shotcrete matches the weathered gray of the diabase, and the wire rope nets should be covered by vegetation in one to two years.

ACKNOWLEDGMENTS

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AN APPROACH TO EARTHQUAKE ANALYSES OF ROCK SLOPES

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Abstract

Traditionally, pseudo-static analyses have been used to evaluate the seismic stability of highway slopes and embankments. These analyses use a constant horizontal force, equal to a percentage of gravitational force, acting on the potential sliding mass. Limit equilibrium analyses are used to calculate that the factor of safety. Minimum factors of safety of between 1.0 and 1.3 are typically required, depending on the sensitivity of the slope. However, pseudo-static analyses are often considered to be obsolete and conservative.

In recent years, there have been significant developments in dynamic stress and deformation analyses of many types of structures, including embankments. Using this approach, the amount of horizontal and vertical deformation is calculated from the design earthquake, using a synthetically-generated time history acceleration at the site from the maximum credible earthquake event on active faults in the region. Usually, a calculated deflection of a few inches from the design earthquake is often considered acceptable, while deflections of several feet are not.

Dynamic stress and deformation analyses can be used for rock slopes where sliding occurs along a complex, curved surface of interconnected joints, where substantial displacement is required before the shear strength drops from the peak value to the residual, or basic, value. However, where the stability of a rock slope is governed by potential failure of a wedge or block of rock which rests on a single joint plane or a combination of joint surfaces, very little displacement is required before the peak strength of the sliding plane drops to the residual or basic strength. In these cases, it is necessary to use the more conservative pseudo-static approach.

This paper briefly describes the principles of pseudo-static and dynamic and deformation deformation analyses and suggests guidelines for the use of each type approach for the analysis of the seismic stability of highway rock slopes.

Introduction

In considering the stability of highway rock slopes, the minimum required factors of safety are normally 1.5 for the static condition and 1.0 to 1.3 for the earthquake condition. In areas of the country where the likelihood of a strong earthquake is low, or even moderate, the static analysis case usually governs and there is little need for close examination of the earthquake case. However, in highly seismic areas, the earthquake case often governs the stability

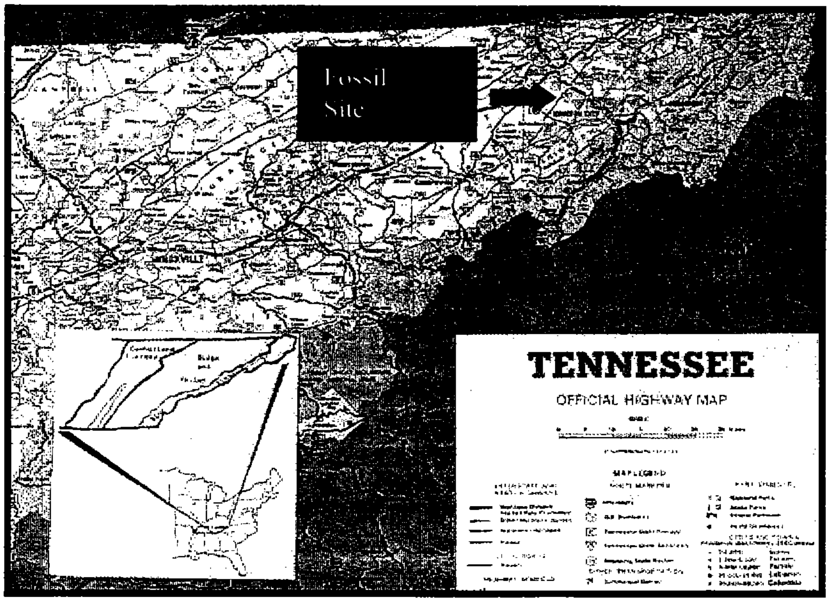

Figure 1. Location of Gray Fossil Site and State Route 75 road project in Washington County Tennessee.

About 500 feet of the new road construction crossed an old deposit of lacustrine sediments, first identified by Larry Bolt, staff geologist with TDOT in Knoxville. Geotechnical concern surrounded the soft clay deposit and how to mitigate the unstable subgrade material. Undercutting the material and back-filling with graded solid rock was being considered along with the use of a geogrid synthetic reinforcement of the subgrade when bone fragments and teeth began showing up in the excavated material (figure 2 and figure 3). Geologists with TDOT, the Tennessee Division of Geology, and the University of Tennessee began looking more closely at the site and finding more fragments of bones and teeth.

The Find

When animal skulls, backbones, ribs and leg bones of mammals began being uncovered TDOT stopped work in the affected area to assess the site in terms of a possible major fossil discovery. A team was formed in July to investigate the fossil deposit to determine if what was initially uncovered was significant. The team consists of persons representing the Tennessee Department of Transportation and The Tennessee Department of Environment and Conservation (Divisions of Geology and Archeology), The University of Tennessee (Departments of Anthropology and Geology), and East Tennessee State University. As a result of the team efforts, numerous animals were found to be represented in the deposit. These included tapir, elephants, turtles, frogs, and a crocodile skull as well as numerous plant leaves, lignite, and seeds.

Figure 2. This image shows the left side of a tapir skull unearthed during the road grading operations along SR75 in Gray, Tennessee.

Figure 3. Road construction equipment encountered lignitic clay deposits along a section of SR 75 in Washington County, Tennessee.

It was first thought that the deposits were of Pleistocene age but fossils later identified by university specialists as being rhinoceros changed the relative date to Miocene (at least 4.5 to 5 million years old and possibly older) (figure 4). It is interpreted that the deposit may

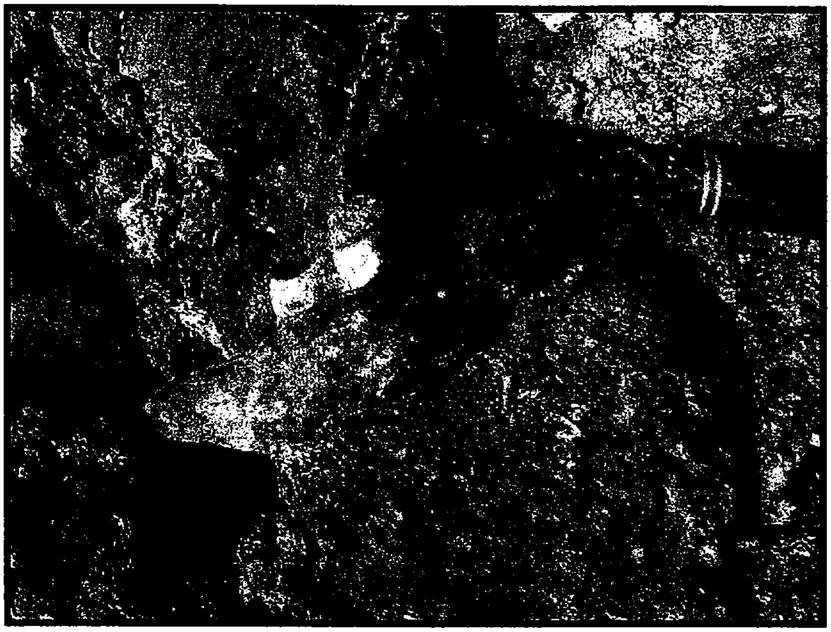

Figure 4. The tooth and partial jaw of a Miocene rhinoceros, *Teleoceras* sp. from the Gray Fossil Site is shown in this photo. Tooth is approximately 2.25 inches long and 1.15 inches wide, and crown to tip of root is almost 3 inches

represent the remains of sediments deposited in an old sinkhole-lake or pond. The sinkhole quite possibly contained a spring that issued water and the animals lived in and around the pond environment, including tapirs and crocodilian species (figures 5,6,7, 8 and 9). Other animals may have come to the pond for water or food. Another scenario is that the deposit may represent an old stream meander cut off from the main stream and developed into a small lake. A watering hole environment seems to be very a plausible description of the fossil site history. The area is estimated to include about 5 acres and thought to include a large deposit of fossils.

The significance of the deposit is that the animals were living in the environment they ended up being buried in, and as such we have a complete context of the geologic history for that span of time. The climate, the vegetation, the animals and the sedimentary record all preserved in the clay deposit.

This site represents the first terrestrial Miocene deposit in Tennessee along with Miocene vertebrate, invertebrate, and plant fossils. It is anticipated that this new Miocene site will rival in importance the famous Cretaceous age Coon Creek fossil site in West Tennessee, known for the presence of over 300 different species of mostly marine invertebrate animals.

A Solution to the Fossil Problem

A concentrated effort was made by TDOT engineers and geologists as well as the Fossil Site Research Team to evaluate three alternative proposals relative to completing the road project. These alternatives included: a) building the road in the same place but allowing researchers to remove what fossils they could in a short time period; b) building a bridge over the fossil site in the same location as the road would be; c) relocate the road alignment to the north, outside of the fossil site, therefore preserving the fossils and site for research.

Alternative "A" would have required the use of undercutting the subgrade material and placing a rock pad in lieu of the soft clay. This alternative also considered the use of geogrid reinforcement. Field testing of the fossil site clay material revealed N values of 3 blows per foot which were correlated to a CBR value of 0.6 to 0.8, and an undrained shear strength of 300 to 400 psf (14.4 to 19 kPa).

The TDOT Geotechnical Engineering office in Knoxville, Tennessee undertook the design of a geogrid system. The design involved the use of Tensar BX1200 geogrid in conjunction with 12 inches of No.57 stone for the subbase (Barker, 2000).

Alternative 'A" was estimated to cost approximately $300,000 to $400,000 dollars but this action would seal up the fossil material from now on, preventing the research from continuing.

Alternative "B" involved the construction of a low bridge over the fossil site. This alternative involved the use of end bearing piling or cast inplace drilled shafts. The construction of the bridge would have placed footings in the fossil deposit rendering those areas off limits to research and damaging fossil material. The cost of this alternative was estimated to be approximately $1.5 million dollars.

Alternative "C" involved the complete relocation of the road alignment in the area of the fossil deposit. The realignment would be located just 400 - 500 feet or so north of the existing road. Drilling in the proposed relocated location revealed no fossil deposit material. The relocation alternative was estimated to cost approximately $700,000-$800,000 dollars. This would include the cost of relocating two houses and a barn. Alternative "C" would also allow for the complete preservation of the fossil site for research.

The road has been realigned to avoid the fossil deposit at the direction of Tennessee Governor Don Sundquist. The state archeologist, Nick Fielder, who will continue the efforts of the research team, is directing activities related to the fossil deposit. The fossil deposit is now known as the Gray Fossil Site.

The fossil site is presently fenced, gated and guarded to protect against possible unauthorized removal of the fossil bone material. Community efforts have greatly helped in the effort to protect this special fossil site.

This discovery required the cooperation and teamwork of a multidisciplinary group of professionals including engineers, geologists, paleontologists, archeologists, biologists, university professors as well as the support from political aspects of our government. The success of this discovery and it's future is most dependent on the excellent communication and cooperation with the professionals, local citizenry, interest groups and politicians that has brought the fossil site to its present protected state.

Efforts are now underway by Nick Fielder, Tennessee State Archeologist, to bring together specialists in the field of paleontology and other scientific fields to asses the direction of future research. This may materialize into a two or three-day seminar to review the material recovered at present and discuss future excavation procedures as well as preservation techniques.

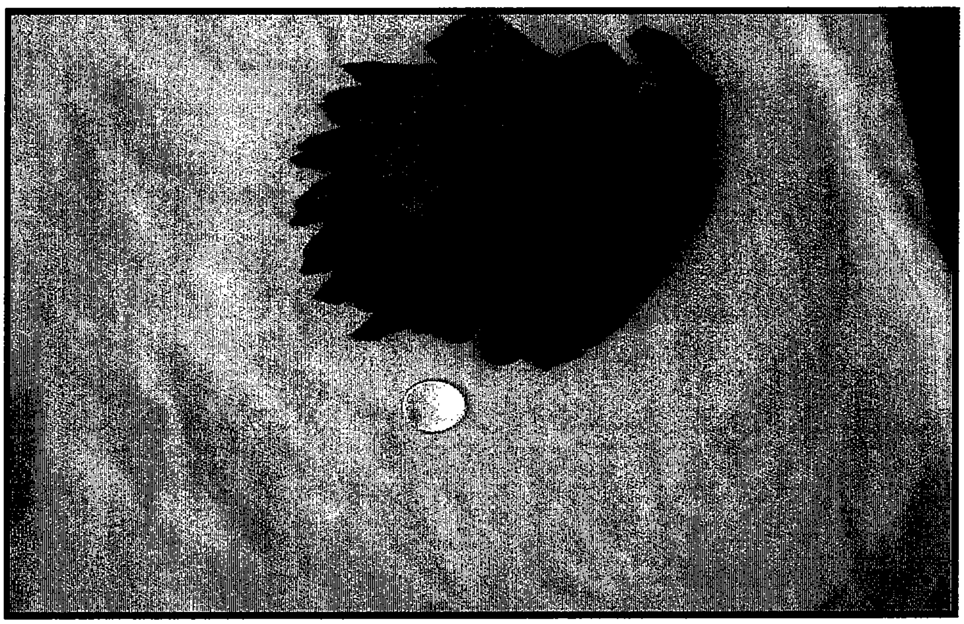

Figure 5. This image shows a tortoise shell found in the clay deposits at the Gray Fossil Site. The shell was actually found under the skull of a tapir excavated during road construction.

Possible developments at the Gray Fossil Site may include long-term research studies, public viewing of research excavation of the fossils and dependent on funding sources, a display facility at the site. It is extremely important to the future of the Gray Fossil Site for the community to remain active in the protection and support of the scientific research that will take place here in the future.

Harry Moore is a geologist with the Tennessee Department of Transportation and manages the Geotechnical Engineering office in Knoxville, Tennessee.

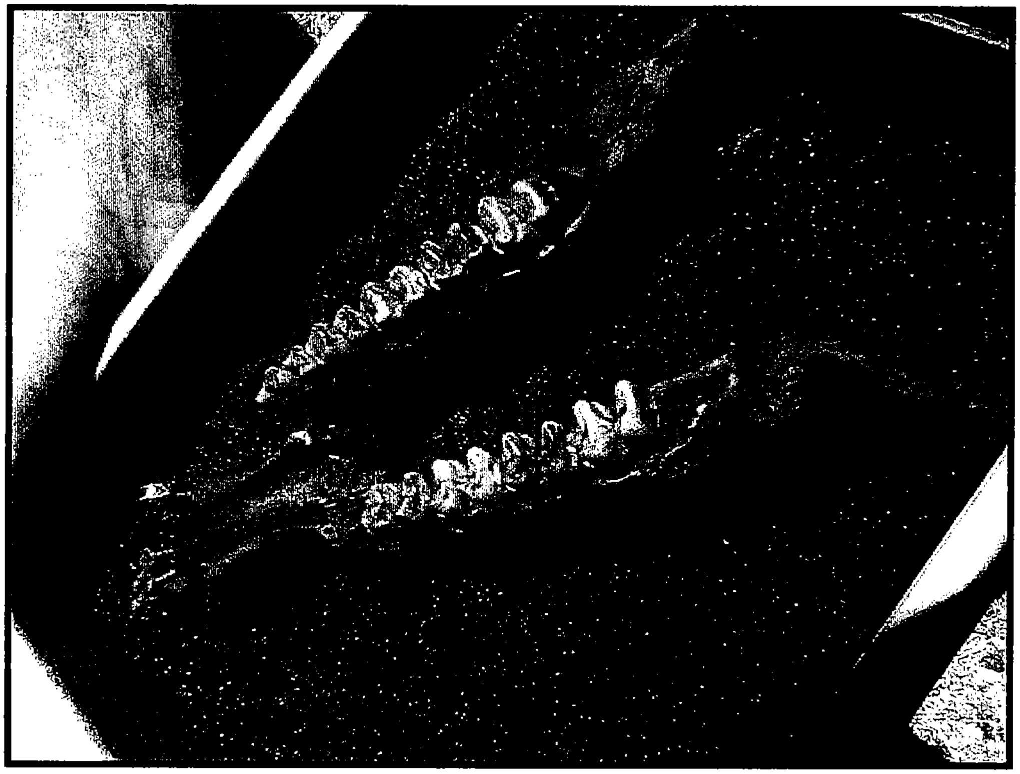

Figure 6. Tapir bones were commonly found in the lignitic clay material as evidenced by this lower mandible of a juvenile tapir (note rear molars not yet protrude through the jawbone).

Figure 7. Crocodilian species were also found in the clay material at the Gray Fossil Site.On the left is a tail vertebrate; on the right is a crocodilian skull held by finder Rick Noseworthy, TDOT biologist, accompanied by TDOT geologist Harry Moore.

Figure 8. Larry Bolt, TDOT geologist who first identified the Miocene fossil deposit, examines some bone fragments taken from the blackish gray lignitic clay.

Figure 9. A backbone of a tapir and pieces of its ribs are exposed in the gray clay of the Gray Fossil Site. Pieces of at least eleven separate tapirs have been found at the site, including several skulls and mandibles.

Acknowledgements

The author wishes to acknowledge several individuals who have contributed to this investigation. Larry Bolt, TDOT geologist, who had the insight and background to recognize the special characteristics of the clay deposit and to bring it to my attention deserves special acknowledgement. Also, David Barker is recognized for his contribution in designing the geogrid concept considered in the initial phases of the discovery. TDOT drilling personnel are also acknowledged for their work on the site characterization drilling needed to define the deposit limits. These include Con Lynn, Carl Elmore, Ledford Russell, Teddy Plemmons, and Jimmy McGill,.

TDOT Project Engineer, Lanny Eggers, and his field inspector Larry Hathaway are acknowledged for their diligent purpose of securing the fossil deposit and providing assistance to the fossil recovery effort. Special thanks is also given to Mr. Rab Summers of Summers Taylor, Inc. of Elizebethton, Tennessee, the grading contractor who built the SR 75 project and aided the effort to save the fossil remains.

University of Tennessee personnel contributed significantly to the project and include Dr. Paul Parmalle, Dr. Walter Klippel, Marta Adams, and Dr. Mike Clark, Dr. Bob Hatcher, and Chris Whisner (UT Knoxville) and Dr Michael Gibson (UT Martin). Geologists from the Tennessee Division of Geology who also helped in the recovery of the fossil bones include Martin Kohl, Bob Price, and Peter Leminski.

A special thanks goes to Nick Fielder, Tennessee State Archeologist, for his organization and operation of the Gray Fossil Site Research Team. Special acknowledgement is also credited to Dr. M. Voorhies with the University of Nebraska and Dr.Alan Holman with the Michigan State University for their help in identifying rhinoceros and crocodilian species found at the Gray Fossil Site

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MITIGATION OF LANDFILL GAS MIGRATING INTO AN AREA OF PROPOSED ROADWAY CUT SLOPE FOR THE SOUTHERN BELTWAY TRANSPORTATION PROJECT

HALE[1], P.A., DOWNS[1], T.L., HEIRENDT[2], K.

ABSTRACT

The Southern Beltway Transportation Project extends from Pennsylvania Route 60 near the Pittsburgh International Airport to the Mon-Fayette Expressway near Finleyville, Pennsylvania. The focus of this discussion will be the proposed interchange with S.R. 30 near Clinton, Pennsylvania. This interchange will intersect several areas of deep mining, and pass adjacent to a former strip mine that was subsequently used as a municipal landfill. The landfill underwent regulatory closure in 1986. A methane extraction and collection system currently serves to mitigate offsite migration of gas towards S.R. 30. Previous assessments of the proposed alignment indicated that no wastes would be encountered by the proposed construction at this location. Subsurface drilling and sampling was performed to support geotechnical investigations of the proposed alignment. Groundwater sampling was performed to determine whether leachate from the landfill would impact the proposed alignment. During the drilling program, methane was sporadically encountered in a fractured sandstone unit in the subsurface. A detailed review of aerial photographs, mining records, and regulatory agency files indicated that mesoscopic geologic structure at the site was providing migrational control for both the methane and landfill leachate. The methane encountered is expected to impact the construction activities for the proposed interchange. Cost effective landfill gas mitigation strategies were developed. Mitigation strategies consisted of over-excavation of strata adjacent to the proposed final roadway grade and construction of a gas impermeable layer adjacent to the roadway. The site characterization approach, survey and sampling results, interpretations, and mitigation solutions are presented.

INTRODUCTION

The purpose of this paper is to present the findings and conclusions associated with the Mazzaro Landfill Site adjacent to the Southern Beltway Transportation project. The primary environmental concerns for this site consist of encountering landfill solid wastes within the alignment, groundwater contamination due to the landfill leachate and encountering methane gas emissions from the landfill. The locations of the project alignment and Mazzaro Landfill Site are presented in Figures 1 and 2, respectively. The Tan alignment of the Southern Beltway Transportation Project extends from U.S. Route 22 in Robinson Township, Washington County to the Southern Expressway (PA 60) in Findlay Township, Allegheny County (see Figure 2). This portion of the proposed roadway is also known as the Findlay Connector. The proposed diamond interchange at this location will provide access to US 30. Interchange construction will also require the realignment of a portion of Point Park Road (see Figure 3).

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SITE DESCRIPTION AND FEATURES

The potential impacts of the Mazzaro Landfill to the project were identified during the corridor study associated with the Southern Beltway Project (See Figure 1). As a part of the Mazzaro Landfill, unlined and lined landfill areas were identified. The unlined portion of the landfill was in operation first with a subsequent construction and operation of a lined landfill location. Currently, the Mazzaro facility no longer accepts waste material and is under a Consent Order with PADEP and the Allegheny County Department of Health to correct problems associated with the landfill. These problems include off-site leachate migration, acid mine drainage, and methane gas emissions.

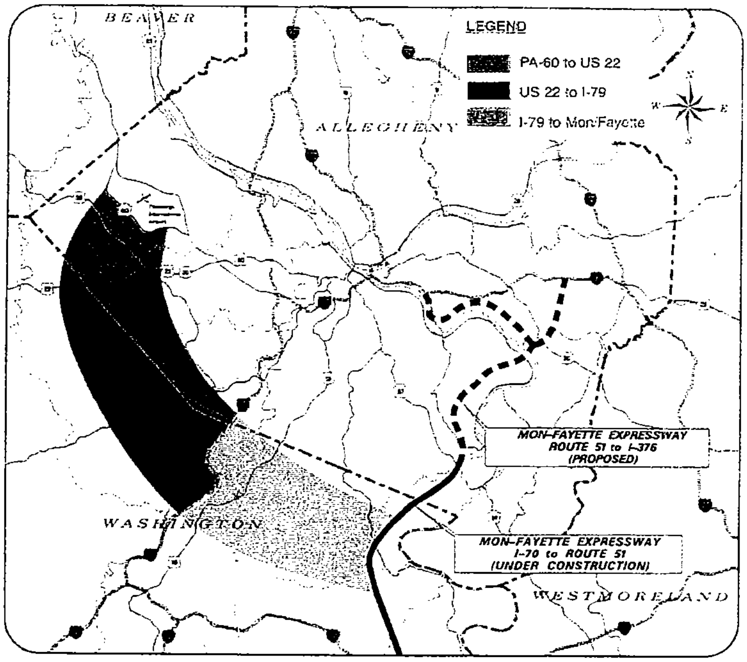


Figure 1. Location of the Southern Beltway Transportation Project. The Findlay Connector is shown as PA 60 to US 22.

The Tan alignment of the Southern Beltway intersects the southern end of the Mazzaro property, approximately 457 meters (1,500 feet) south of the actual landfills (Figure 2). Within this area of the site, the Mazzaro property has been strip mined and backfilled with mine overburden materials. Based upon interviews, site reconnaissance, and aerial photograph review, the historic landfill activity was primarily limited to the northwestern section of the property within the lined and unlined landfills.

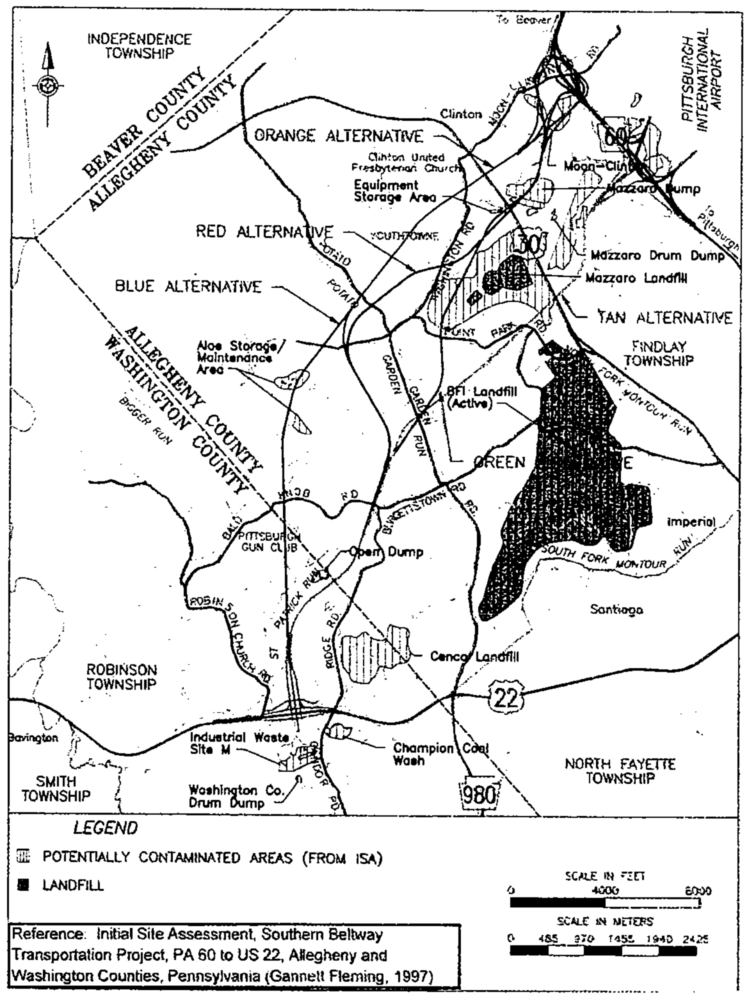


Figure 2. Various alignments studied for the Findlay Connector.

Physical Setting

The site lies within the Pittsburgh Low Plateau section of the Appalachian Plateau physiographic province. This province is characterized by generally horizontal strata subjected to gentle folding. Stratigraphically the site lies above

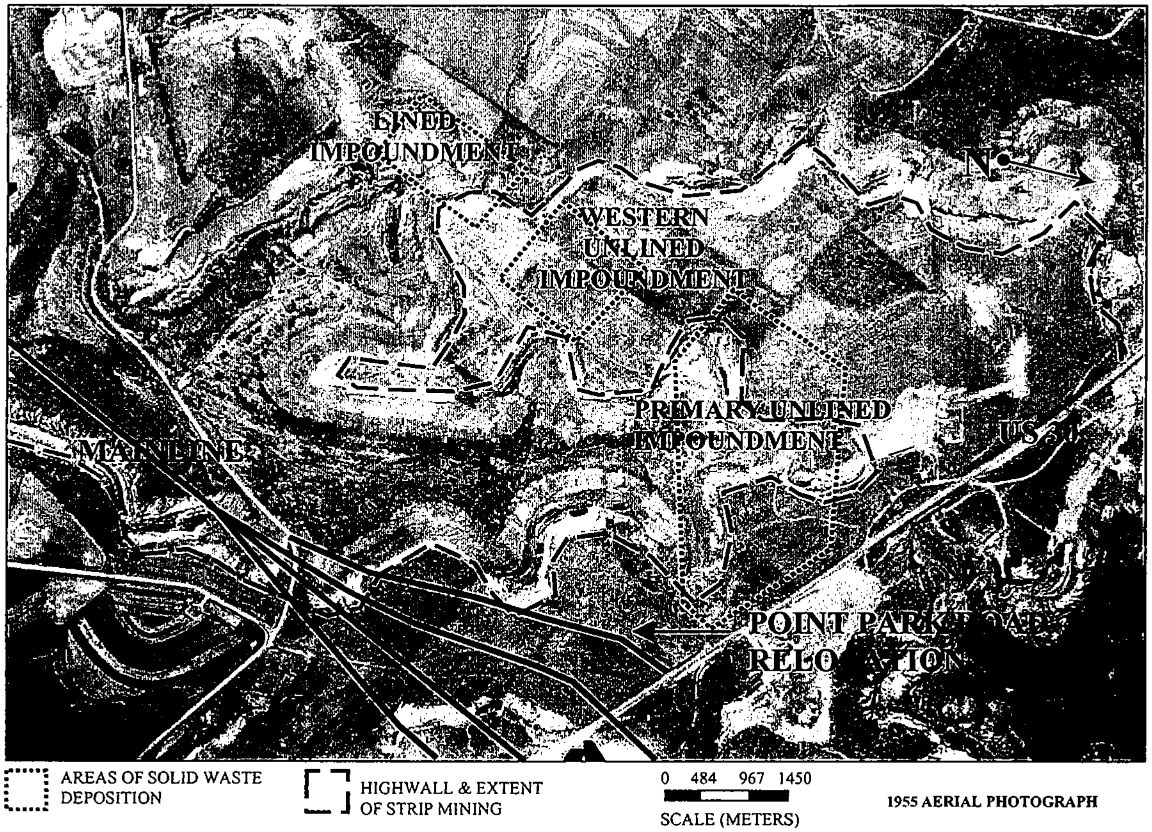


Figure 3. 1955 aerial photograph showing the proposed alignment, highwalls, and documented limits of solid waste deposition.

the horizon of the Pennsylvanian age Pittsburgh Coal. This member serves to separate the Casselman Formation below from the Pittsburgh Formation above. Soils are dominantly related to the weathering and erosion of the parent bedrock, typically producing residual, colluvial, and alluvial soils.

Site History and Land Use

The site has had three primary land uses during its history. These uses include agriculture, mining, and municipal landfill activities.

Agricultural activities are shown in 1936 aerial photographs. These photographs are the earliest available for the area and show the project site as farmlands. The surrounding landscape consists entirely of farmlands and forested hilltops. Therefore, it is assumed that the earliest land use for the site was agricultural, if any.

Mining activities are evident in the aerial photographs dating from 1955 to 1963, which indicate that strip mine activities were extensive within this area. Strip mining techniques were used to remove the Pittsburgh coal seam primarily along the crop line. Figure 3 shows the evidence of the strip mining activities and their associated highwalls.

In addition to the strip mine activities, the published literature and boring data indicate that deep mine activities were also prevalent in the area. The Clinton Block Coal Company deep mined this area extensively.

The Mazzaro property northwest of the proposed alignment was used for the disposal of municipal and sanitary wastes. The landfill operated from 1972 to 1986. Permitted wastes were placed in three locations (Figure 3). The initial permit allowed waste placement in an unlined valley left from earlier strip mining activities. The unlined impoundment consisted of two discrete areas, broken up by a power line right-of-way passing through the property. Wastes were prohibited from further disposal in the unlined impoundment in 1986, at which time disposal began in a lined facility. This western most disposal location operated until 1988, when the Mazzaro Landfill was closed. The landfill currently operates a methane collection system. This system draws methane off of the main unlined impoundment via an extraction well network.

In addition to the above-mentioned conclusions, an aerial photo review indicated that unpermitted waste disposal occurred on the Mazzaro property between 1956 and 1971. The 1971 and 1975 aerial photographs (Figure 4) that show several haulage roads and shallow excavations clustered around the unlined portion of the landfill evidence this. The 1975 aerial photograph also shows an active disposal cell along the southern edge of the site, near the highwall and proposed relocated Point Park Road alignment.

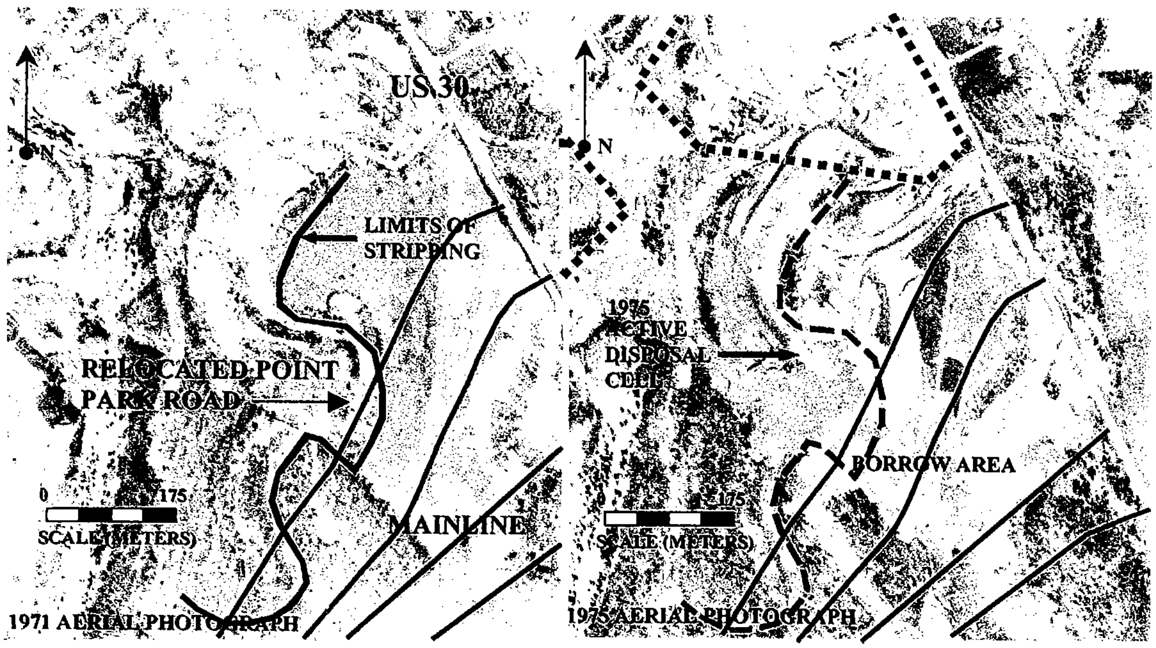


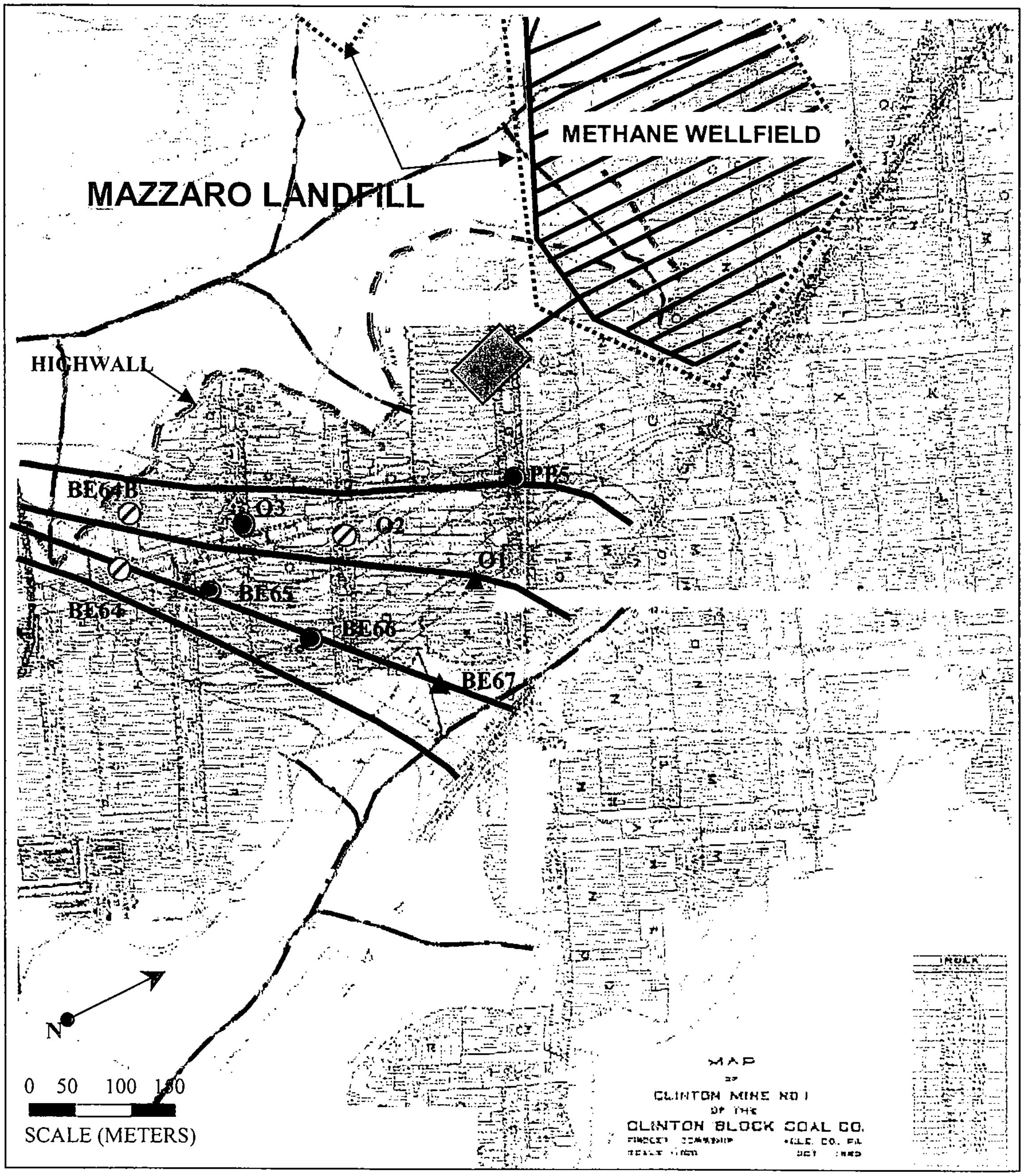
Figure 4. Comparison of 1971 and 1975 aerial photographs showing disturbance and waste disposal outside of permitted areas.

FIELD INVESTIGATION

This investigation consisted of a document review, field reconnaissance, geotechnical borings, installation of groundwater sampling points, and other environmental media sampling. A summary of each of these investigative elements and their results is presented below.

Field reconnaissance activities were carried out concurrently with other elements of the field investigation. Reconnaissance activities included field mapping of mining features (highwalls, spoil piles, barrow areas, etc.), determination of field sampling locations, interviews with residents and methane collection system staff, and field verification of disturbed areas observed on aerial photographs.

There were a total of 9 borings drilled within the suspected contamination zone on the Mazzaro property (Figure 5). These borings were advanced within this area to collect subsurface data required for the geotechnical design and construction of the roadway as well as to collect the necessary samples for analytical testing. One monitoring well was installed for the purpose of collecting groundwater samples for analytical testing. Split-spoon samples were collected and Standard Penetration Tests (SPT) were conducted continuously through the overburden soils. Split inner barrel rock coring (NQ2) was performed to the completion depths of the borings.



LEGEND

STRUCTURE CONTOUR BASED ON MINE MAP BASE OF COAL ELEVATION DATA
DRAINAGE
METHANE
BORING
MONITORING WELL

Figure 5. Figure summarizing site conditions and elements of the subsurface investigation.

Borings were monitored with a Perkin-Elmer photoionization detector (PID) and a combustible gas indicator (CGI) for health and safety purposes. The PID monitoring indicated little to no presence of the organic compounds to which the PID is sensitive. Methane, however, was found in high concentrations (greater than 100% of the lower explosive limit (LEL) in borings 0-2, BE-64A, and BE-64B (Figure 5).

In order to evaluate the existing groundwater conditions at the site, monitoring wells were installed in selected geotechnical borings. A monitoring well and piezometer were installed in borings BE-67 and O-1, respectively.

One groundwater sample was collected from Boring BE-67 and tested to characterize the groundwater within the proposed alignment. The sample was analyzed for the EPA Priority Pollutant suite of analytes which include volatile organic compounds (VOC), semivolatile organic compounds (SVOC), Pesticides / Polychlorinated Biphenyls (PCB), and Metals.

SOILS, GEOLOGY, AND HYDROGEOLOGY

The borings advanced at the site encountered both natural insitu soils and weathered strip mine overburden spoil. The natural insitu material consists mainly of clayey soils with varying degrees of sand and silt sized particles that were derived in-place as a part of the continual weathering of the underlying bedrock. The mine spoil consists primarily of a matrix of the natural insitu soils and the Pittsburgh Sandstone bedrock mechanically broken down by the strip mining activities. In general, the strip mine overburden spoil exhibits similar consistency characteristics as the natural insitu soils, with an increased frequency of rock fragments and trace amounts of carbonaceous materials.

The rock represented at the site consists of the Pittsburgh Sandstone, Pittsburgh Coal and the Pittsburgh Limestone units (Figure 7). The Pittsburgh Sandstone consists of discrete as well as interbedded layers of limestone, sandstone, siltstone, and shale of the Pittsburgh Formation. The fractures within this formation commonly exhibited iron staining and clay infilling. The Pittsburgh Coal seam within the site limits was deep mined by room and pillar methods and strip mined. The borings drilled within the deep mined portions exhibited both intact coal (presumably pillars) and void. Below the Pittsburgh Coal, underclay, interbedded sandstone, limestone, and shale of the Casselman Formation were encountered consistently across the site. Bedrock forms a local structural dome in the vicinity of the site (based on published sources and top of coal elevation data)

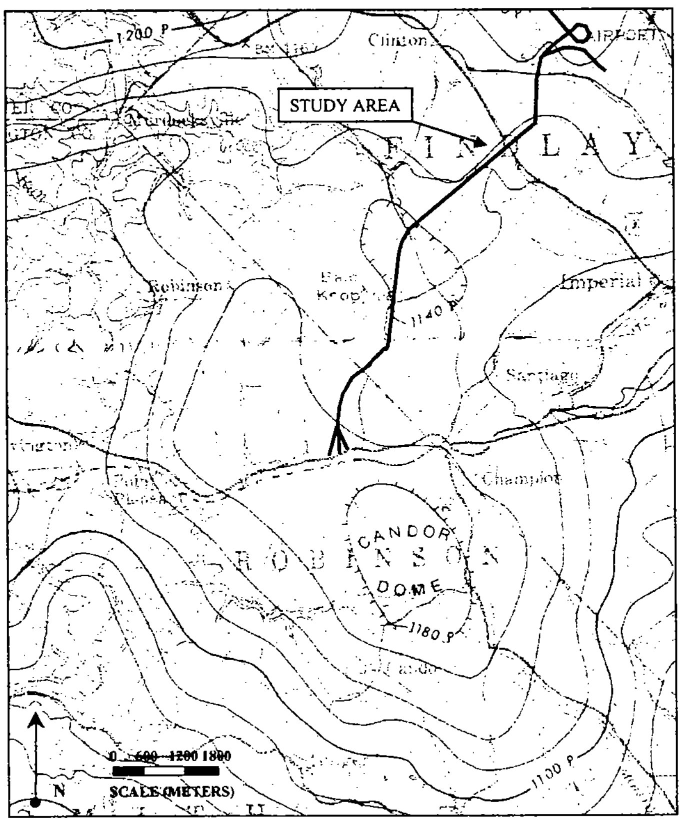


Figure 6. Structure contour map of base of Pittsburgh Coal.

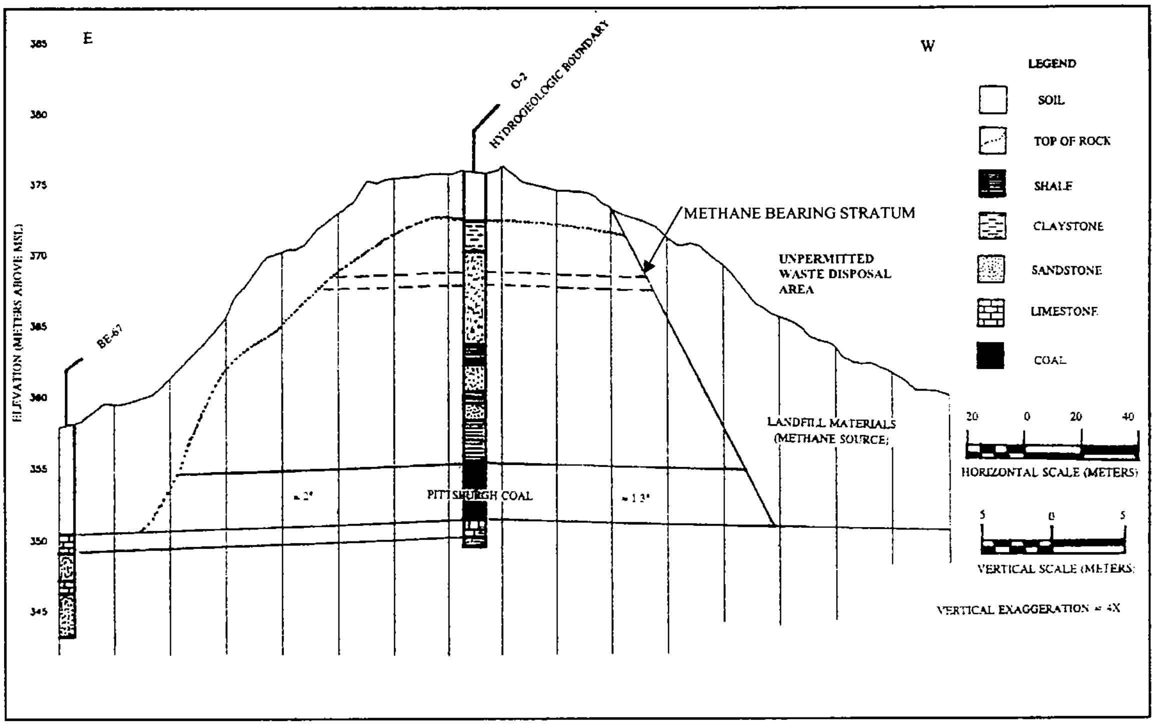


Figure 7. Conceptual model of site geology based on subsurface investigation.

(Figure 6). The overall regional dip is to the southwest, in the direction of a structural basin having its nadir near the Bald Knob area (Figure 6).

Limited hydrogeologic data is available for this site. Synoptic water levels were collected in the monitoring wells installed in borings BE-67 and O-1. Water levels in these wells correspond to the mine elevation of the former Clinton Block Coal Company deep mine. The data indicates a decrease in groundwater elevation from monitoring well O-1 to BE-67. This indicates groundwater migration towards the southeast from the mine. This is supported by the results of the groundwater testing. This testing indicated high levels of manganese and iron. Elevated levels of these metals in groundwater are indicative of the presence of acid mine drainage.

An additional data point for the interpretation of ground water migration patterns is the wetlands due west of relocated Point Park Road. Static water level in this area lies approximately three meters below the mine pool in monitoring well O-1. Therefore there is westerly flow of groundwater from the mine also. Groundwater flowing away from boring O-1 to both the southeast and west is indicative of the presence of a hydrogeologic boundary in the vicinity of O-1. This hydrogeologic boundary is shown on Figure 5 by the structural contours derived from base of coal elevation data taken from the mine map that serves as the base for Figure 5. The hydrogeologic boundary roughly parallels the topographic ridge that trends north south across the site. This boundary should serve as a leachate migration barrier between the landfill and the proposed alignment (see Figure 7).

INTERPRETATION OF DATA

Landfill Solid Wastes

The solid wastes contained within the Mazzaro property are located approximately 150 meters northwest of the proposed alignment. None of the western most borings encountered any evidence of solid waste deposition. This is consistent with all available research literature collected for this project. Therefore, no solid landfill waste is anticipated to be encountered during the construction of the proposed roadway.

Groundwater Quality

All available groundwater data was collected and evaluated with respect to the proposed design and construction of the proposed roadway alignment. The groundwater sample collected from the boring BE-67 monitoring well was analyzed for VOC, SVOC, Pesticides / PCB, and Metal contaminants. The analytical results for this sample indicated no contamination of the groundwater due to the Mazzaro Landfill activities within this portion of the site. Based on Gannett Fleming's interpretation of the geologic structure and the likely hydrogeologic boundary between the landfill and the monitoring well, no groundwater remediation is necessary with regards to landfill leachate.

Historical analytical data is consistent with this interpretation. The historic data collected by previous researchers, in the vicinity of BE-67, indicated high levels of manganese and iron in the groundwater at this location. Refer to Figure 5. This location is at the head of Montour Run and also located down gradient of the strip and deep mined area north east of the proposed alignment. High levels of manganese and iron are indicative of acid mine drainage. Therefore acid mine drainage is migrating from the former mine areas towards the southeast.

Methane Occurrence

Elevated levels of methane were encountered in borings O-2, BE-64A, and BE-64B. The concentration of methane at the boring casing exceeded the lower explosive limit for an extended period of time. The readings obtained ranged from 100 to 150 percent of the lower explosive limit (LEL). The emission of the methane gas vapors from the top of the boring was visually evident resembling heat convection currents. The quantity of methane was not measured however; the concentration of methane did not dissipate after three days of venting. In all instances, the methane was detected when the boring was advanced into a fractured sandstone horizon occurring at approximately elevation 368 meters MSL.

Two likely sources of methane were considered. The first and most probable source was the adjacent landfill. The presence of methane is documented in this location and evidenced by the landfill gas recovery system currently in operation. The second methane source considered was coal bed methane of the previously mined Pittsburgh Coal seam. The likelihood of the methane being derived from the coal seam was considered to be low for the following reasons:

- Methane was encountered when the drilling entered a sandstone layer approximately 13 meters above the Pittsburgh coal.
- Other borings advanced into the deep mined Pittsburgh coal seam within this area did not indicate the presence of methane.

near US 30. This is beyond the practical excavation depth of standard equipment, especially in bedrock. Therefore a specialty contractor would be required, constituting higher project costs.

Alternative 2, implementing a methane collection system, was also deemed not a viable option for methane mitigation. This was due to the uncertainty regarding the quantity and extent of methane migration in the subsurface. Resolving this uncertainty, as well as the operation and maintenance costs of such a system, also translates into higher project costs.

Alternative 3,consisting of a methane migration barrier, proved to be the most advantageous mitigation measure. This option is a passive treatment (with minimal operation and maintenance costs), built with standard construction equipment, and designed to be built concurrently with the adjacent roadway. This mitigation design consists of over-excavation of the cut slope in those areas affected by methane, and placement of fine grained cohesive soils against the face of the over-excavated slope. Soils of this type are prevalent in the project area, therefore select borrow will not be required. Construction of this barrier will be incidental to the excavation costs associated with the entire project.

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52nd Annual
Highway Geology Symposium

FIELD TRIP GUIDE

May 17, 2001
Rocky Gap State Park
Cumberland, Maryland

Sponsored By:
Maryland State Highway Administration
Maryland Geological Survey

Thursday, May 17, 2001

Contents

Summary of Stops

OVERVIEW OF FIELD TRIP GEOLOGY

Our field trip will be within the Valley and Ridge Division of the Appalachian Physiographic Province of the state. This area extends from just west of Conococheague Creek in Washington County to Dans Mountain in Allegany County.

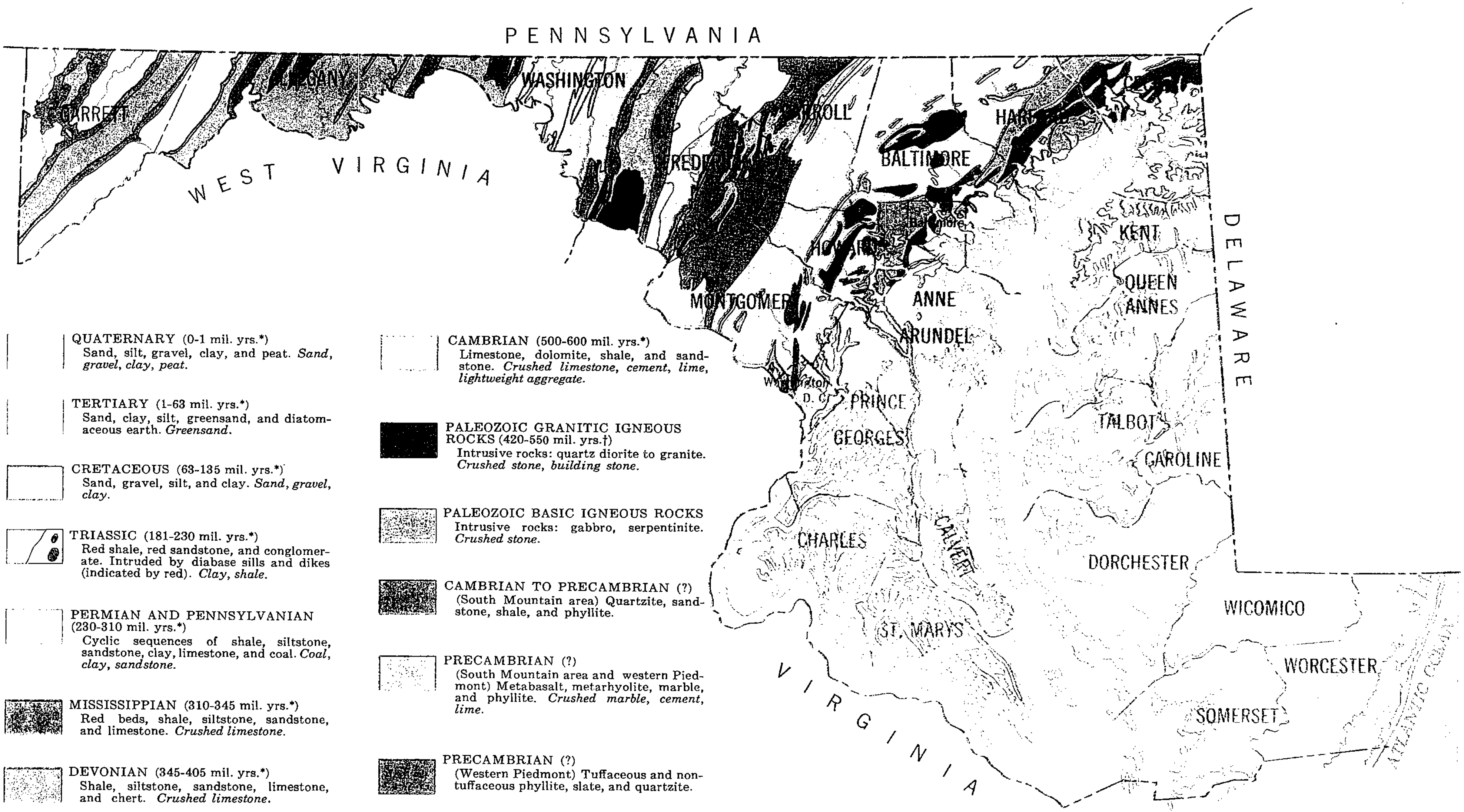
Rock ranges from Ordovician to Mississippian in age. The orientation of the sandstone ridges is north 10° - 15°. The valleys drain southward to the Potomac River and are underlain by shales and siltstones.

The first named road that crossed the area was the Baltimore Pike, which connected Cumberland to Baltimore. West of Cumberland, this road was known as the National Road, and has the distinction of being the first federally supported road. The National Road/Baltimore Pike became US 40. As time went by, sections of US 40 were upgraded, and in some areas replaced. The abandoned segments show on the state road maps as MD 144.

With the construction of the Interstate Highway System, the part of I-70 that was built in Maryland replaced US 40 as the primary road to the west from Baltimore. At the town of Hancock, I-70 turns north and enters Pennsylvania where it joins the Pennsylvania Turnpike on its way west to the Ohio River Valley. This left the westernmost part of the state without an interstate highway to connect it to the port of Baltimore.

In the late 1960's, the Federal Appalachian Development Program began to address the depressed economic situation in the Appalachians by funding a roadway named the National Freeway to connect Morgantown, West Virginia to I-70 at Hancock, Maryland. Also known as US 48, the sections of this highway from Cumberland to Morgantown were completed by the mid 1970's. The segments east of Cumberland were completed in the late 1980's under the interstate program, and when the ribbon was cut in 1991, the entire road was incorporated into the interstate system as I-68.

We will travel east on I-68 from the conference center to Sideling Hill for the first stop. From Sideling, we will go south on US 522 to Berkley Springs for some fossil hunting. The third stop will be at the Chesapeake and Ohio (C&O) Canal tunnel across the Potomac River from Paw Paw, West Virginia. The forth and last stop will be on the nearly completed Canal Parkway in Cumberland, Maryland.



QUATERNARY (0-1 mil. yrs.*)
Sand, silt, gravel, clay, and peat. *Sand, gravel, clay, peat.*

TERTIARY (1-63 mil. yrs.*)
Sand, clay, silt, greensand, and diatomaceous earth. *Greensand.*

CRETACEOUS (63-135 mil. yrs.*)
Sand, gravel, silt, and clay. *Sand, gravel, clay.*

TRIASSIC (181-230 mil. yrs.*)
Red shale, red sandstone, and conglomerate. Intruded by diabase sills and dikes (indicated by red). *Clay, shale.*

PERMIAN AND PENNSYLVANIAN (230-310 mil. yrs.*)
Cyclic sequences of shale, siltstone, sandstone, clay, limestone, and coal. *Coal, clay, sandstone.*

MISSISSIPPIAN (310-345 mil. yrs.*)
Red beds, shale, siltstone, sandstone, and limestone. *Crushed limestone.*

DEVONIAN (345-405 mil. yrs.*)
Shale, siltstone, sandstone, limestone, and chert. *Crushed limestone.*

SILURIAN (405-425 mil. yrs.*)
Shale, mudstone, sandstone, and limestone. *Glass sand, crushed limestone.*

ORDOVICIAN (425-500 mil. yrs.*)
Limestone, dolomite, shale, siltstone, and red beds. Slate and conglomerate in northern Harford County. *Crushed limestone, cement, clay, lime.*

CAMBRIAN (500-600 mil. yrs.*)
Limestone, dolomite, shale, and sandstone. *Crushed limestone, cement, lime, lightweight aggregate.*

PALEOZOIC GRANITIC IGNEOUS ROCKS (420-550 mil. yrs.†)
Intrusive rocks: quartz diorite to granite. *Crushed stone, building stone.*

PALEOZOIC BASIC IGNEOUS ROCKS
Intrusive rocks: gabbro, serpentinite. *Crushed stone.*

CAMBRIAN TO PRECAMBRIAN (?)
(South Mountain area) Quartzite, sandstone, shale, and phyllite.

PRECAMBRIAN (?)
(South Mountain area and western Piedmont) Metabasalt, metarhyolite, marble, and phyllite. *Crushed marble, cement, lime.*

PRECAMBRIAN (?)
(Western Piedmont) Tuffaceous and nontuffaceous phyllite, slate, and quartzite.

PRECAMBRIAN (?)
(Eastern Piedmont) Schist, metagraywacke, quartzite, marble, and metavolcanic rocks. *Crushed stone, crushed marble, building stone.*

PRECAMBRIAN BASEMENT COMPLEX (1100 mil yrs.†)
Gneiss, migmatite, and augen gneiss.

Most important mineral products in italics.
* Age ranges from Kulp, J. L., 1961, Geologic time scale: Science, v. 133, no. 3459, p. 1105-1114.
† Radiometric dates made on Maryland rocks.

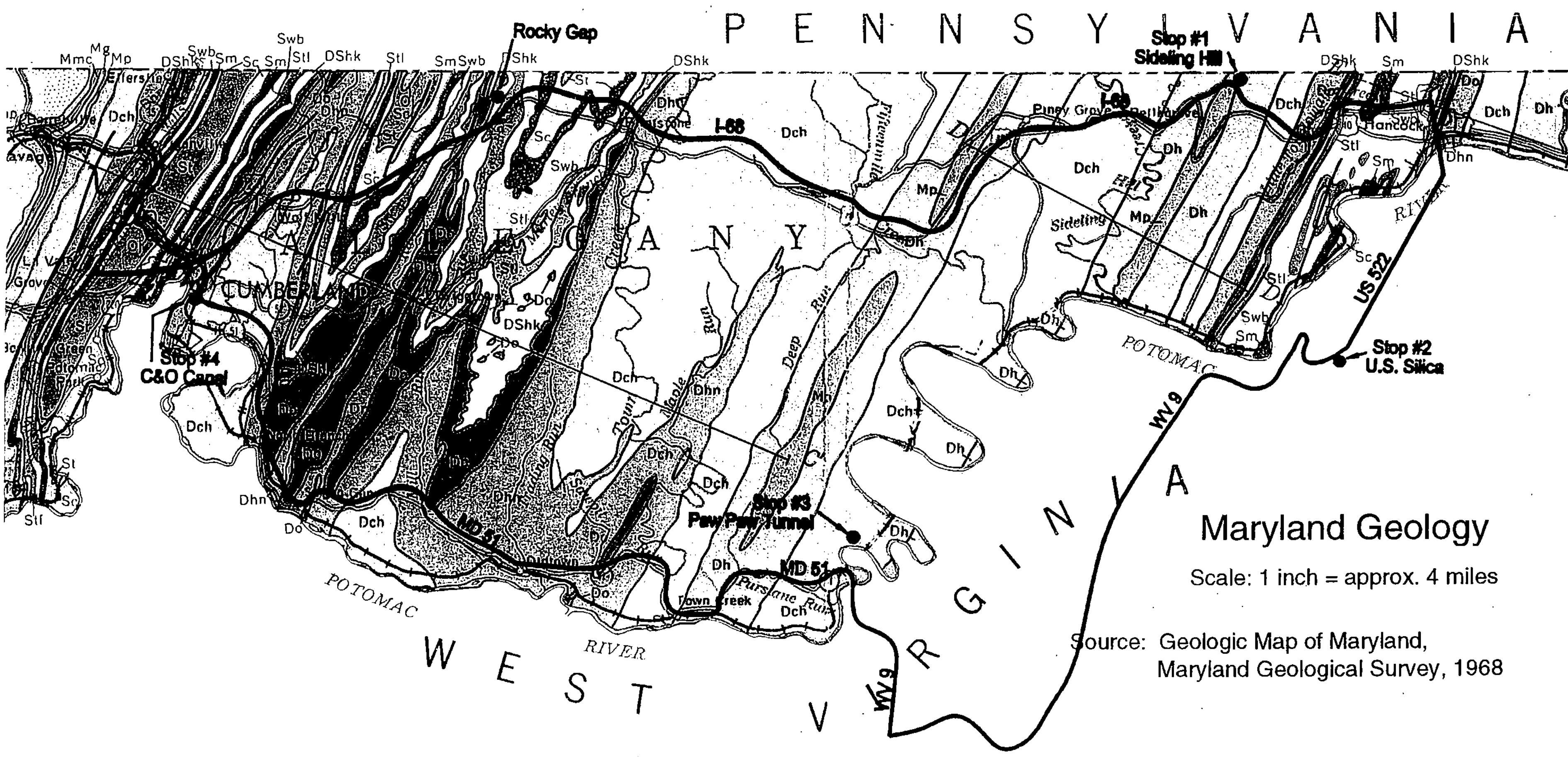
MARYLAND GEOLOGICAL SURVEY
Kenneth N. Weaver, Director

GENERALIZED GEOLOGIC MAP OF MARYLAND‡

1967

10 0 10 20 30 40
1 inch equals 25 miles

‡ A detailed Geologic Map of Maryland, 1968 at a scale of 1 inch equals 4 miles, is also available.



PERMIAN

Pd — Dunkard Group
Red and green shale, siltstone, and sandstone, with thin lenticular beds of argillaceous limestone and thin beds of impure coal; thick-bedded, white conglomeratic sandstone at base; thickness greater than 200 feet; occurs only on hilltops in center of Georges Creek Basin.

PENNSYLVANIAN

Pm — Monongahela Formation
Interbedded claystone, argillaceous limestone, shale, sandstone, and coal beds; Waynesburg coal at top; Pittsburgh coal at base; thickness 240 feet in west, increases to 375 feet in east.

Pc — Conemaugh Formation
Includes the rocks between the base of the Pittsburgh coal and the top of the Upper Freeport coal; consists of two unnamed members which are separated by the Barton coal; both members are gray and brown claystone, shale, siltstone, and sandstone, with several coal beds; lower member also contains redbeds and fossiliferous marine shales; thickness 825 to 925 feet.

Pap — Allegheny Formation
Interbedded sandstone, siltstone, claystone, shale, and coal beds; Upper Freeport coal at top; where present, Brookville coal defines base; thickness 275 feet in northeast, increases to 325 feet in south and west.

Pottsville Formation
Interbedded sandstone, siltstone, claystone, shale, and coal beds; conglomeratic orthoquartzite and protoquartzite at base; thickness 60 feet in northeast, increases to 440 feet in southwest.

MISSISSIPPIAN

Mmc — Mauch Chunk Formation
Red and green shale, reddish-purple mudstone, and red, green, brown, and gray thin-bedded and cross-bedded sandstones; thickness 500 feet in west, increases to about 800 feet in east.

Mg — Greenbrier Formation
Upper part red calcareous shale and sandstone interbedded with greenish-gray and reddish-gray argillaceous limestone; Loyalhanna Limestone Member: *Gray to red, cross-bedded, arenaceous calcarenite; total thickness 200 to 300 feet.*

Mp — Pocono Group
Gray, white, tan, and brown, thin- to thick-bedded, cross-bedded sandstone, locally conglomeratic; interbedded gray and reddish-brown shale, mudstone, and siltstone; fragmentary plant fossils. Undifferentiated in Garrett and western Allegany Counties.

Purslane Sandstone
White, thick-bedded, coarse-grained sandstone and conglomerate with thin coal beds and red shales. Eastern Allegany and Washington Counties.

Rockwell Formation
Coarse-grained arkosic sandstone, fine-grained conglomerate, and buff shale; dark shale with thin coal beds near base. Eastern Allegany and Washington Counties.

Total thickness of Group 250 feet in west, increases to 1,700 feet in east.

DEVONIAN

Dh — Hampshire Formation
Interbedded red shale, red mudstone, and red to brown cross-bedded siltstone and sandstone; some thin green shale; greenish-gray sandstone and shale toward top; fragmentary plant fossils; thickness 1,400 to 2,000 feet in west, increases to 3,800 feet in east.

Dch — "Chemung" Formation
Predominantly marine beds characterized by gray to olive-green graywacke, siltstone, and shale; thickness ranges from 2,000 to 3,000 feet.

Parkhead Sandstone
Gray to olive-green sandy shale, conglomeratic sandstone, and graywacke; present in Washington County, identification uncertain in west; thickness averages 400 feet.

Brallier Formation
(Woodmont Shale of earlier reports). Medium to dark gray, laminated shale and siltstone; weathers to light olive-gray; grain size coarsens upward; thickness about 2,000 feet in west, about 1,700 feet in east.

Harrell Shale
Dark gray laminated shale; absent in east where Brallier lies directly on Mahantango, Tully Limestone *lies near base in west, in subsurface of Garrett County; total thickness in west 140 to 300 feet.*

Note: "Chemung," Parkhead, Brallier, and Harrell Formations formerly were designated as Jennings Formation.

Dhn — Hamilton Group

Mahantango Formation
Dark gray, laminated shale, siltstone, and very fine-grained sandstone; thickness 600 feet in west, increases to 1,200 feet in east.

Marcellus Shale
Gray-black, thinly laminated, pyritic, carbonaceous shale; thickness 250 feet in east, increases to 500 feet in west.

Tioga Metabentonite Bed
Brownish-gray, thinly laminated shale containing sand-size mica flakes; thickness less than one foot.

Needmore Shale
Olive-gray to black shale and dark, thin-bedded, fossiliferous, argillaceous limestone; thickness ranges from 70 to 145 feet.

Note: Hamilton Group, Tioga Metabentonite Bed, and Needmore Shale formerly were designated as Romney Formation.

Do — Oriskany Group

Ridgeley Sandstone
White, medium- to coarse-grained, fossiliferous, calcareous orthoquartzite; thickness 160 feet in west. Medium to dark gray cherty, arenaceous limestone in east; thickness 50 feet.

Shriver Chert
Dark gray, brown, and black silty shales, cherty shales, and nodular and bedded black chert; fossiliferous; thickness 170 feet in west, upper boundary gradational with Ridgeley. Thickness 14 feet in east where the lower Shriver intertongues with the Licking Creek Limestone Member of the Helderberg Formation.

DShk — Helderberg Formation
Licking Creek Limestone Member: *(Becraft Limestone of earlier reports.) Present only in east. Medium gray, medium-grained limestone near top; bedded black chert and thin-bedded limestone in middle; silty argillaceous limestone and shale near base; contains tongues of Shriver and Mandata; thickness 110 feet.* Mandata Shale Member: *Dark brown to black, thin-bedded shale; fossiliferous; thickness 20 to 30 feet in west, intertongues with Licking Creek Limestone Member in east.* Corriganville Limestone Member *(Head): (New Scotland Limestone of earlier reports.) Medium gray, medium-grained, medium-bedded limestone, interbedded with chert; fossiliferous; thickness 15 to 30 feet.* New Creek Limestone Member: *(Coeymans Limestone of earlier reports.) Medium gray, thick-bedded, coarse-grained limestone; fossiliferous; thickness 9 to 10 feet. Limestone changes facies eastward into sandstone, the* Elbow Ridge Sandstone Member: *Medium-bedded, medium- to coarse-grained, calcareous sandstone; thickness 10 to 18 feet.*

Keyser Limestone
Dark gray, thin- to thick-bedded, fine- to coarse-grained calcarenite; contains nodular limestone, dolomitic limestone, and calcareous shale; cherty near top; fossiliferous; thickness 200 to 300 feet.

SILURIAN

Stl — Tonoloway Limestone
Gray, thin-bedded limestone, dolomitic limestone, and calcareous shale; thin sandstone member in east 20 feet above base; fossiliferous; thickness 400 feet in east, increases to 600 feet in west.

Swb — Wills Creek Shale
Olive to yellowish-gray, thin-bedded mudstone, calcareous shale, argillaceous limestone, and sandstone; thickness 450 feet in west, increases to 600 feet in east.

Bloomsburg Formation
Bright red, hematitic, thin- to thick-bedded sandstone and shale; some dark sandstone and green shale; Cedar Creek Limestone Member: *Dark gray, fine- to medium-grained argillaceous limestone, occurs in middle part of formation; total thickness 20 feet in west, increases to 200 feet in east.*

Sm — McKenzie Formation
Gray, thin-bedded shale and argillaceous limestone; interbedded red sandstone and shale in east; thickness 160 feet in western Washington County, increases to 300 feet in east and 380 feet in west.

Sc — Clinton Group

Rochester Shale
Gray, thin-bedded calcareous shale and dark gray, thin- to medium-bedded lenticular limestone; thickness 25 to 40 feet.

Keefer Sandstone
White to yellowish-gray, thick-bedded protoquartzite and orthoquartzite; calcareous to west; thickness 10 feet in west, increases to 35 feet in east.

Rose Hill Formation
Olive-gray to drab, thin-bedded shale; some purple shale and gray, thin-bedded sandstone; Cresaptown Iron Sandstone Member: *Purple, hematite-cemented, quartzose sandstone; thickness 5 to 30 feet; occurs in lower half of formation; total thickness 300 feet in east, increases to 570 feet in west.*

St — Tuscarora Sandstone
White to light gray, thin- to thick-bedded, cross-stratified subgraywacke and orthoquartzite; thickness 60 feet in east, increases to 400 feet in west.

LEGEND

DEVONIAN SYSTEM

Dhs Hampshire Formation

Non-marine shales and fine micaceous sandstones, mostly red to brownish-gray, including siltstone, sandstone and conglomerate. Generally distinguishable from the underlying Chemung by non-marine character and red color.

Dch Chemung Group

Gray to brown siltstone and sandstone with shale and conglomeratic interbeds; mainly marine and sparingly fossiliferous; boundaries gradational. Can be divided into the Foreknobs and Scherr Formations along the Allegheny Front. Parkhead Sandstone Member near base.

Db Brallier Formation

Predominantly olive-gray to dark, thickly laminated marine shale, with considerable siltstone and thin sandstone lenses; mainly nonfossiliferous.

Dh Harrell Shale

Dark gray to black thinly laminated to fissile shale. Calcareous shale and limestone lenses near the base (Tully).

Dmt Mahantango Formation

Thickly laminated marine shale, siltstone, very fine sandstone, and some limestone, with an occasional coral reef or biostrome. Contains the Clearville and Chaneysville Siltstone Members of Pennsylvania.

Dm Marcellus Formation

Predominantly gray-black to black thinly laminated noncalcareous pyritic shale. Contains one or more thin-bedded limestones, including the Purcell Member of Pennsylvania.

Needmore Shale

Predominantly dark gray or green, calcitic, mostly nonfissile shale. Gives strong "kick" on gamma ray logs. Tioga Bentonite near the top. Includes the black Beaver Dam Shale Member. Grades westward into the Huntersville Chert. *Not mappable at scale of this map. Included with Dmn.*

Huntersville Chert

Ranges from a nearly pure slightly calcitic or dolomitic chert to an inter-tonguing of such chert and the Needmore Shale. Grades westward in the subsurface to a limestone, commonly considered as "Onondaga". Contains the "glauconitic" Bobs Ridge Sandstone Member. *Not mappable at scale of this map. Included with Do.*

Do Oriskany Sandstone

Sometimes designated Ridgeley in eastern West Virginia. White to brown, coarse- to fine-grained, partly calcareous sandstone, locally pebbly or conglomeratic, and ridge-forming. May be white, nearly pure silica, and a source of glass sand, as at Berkeley Springs, Morgan County.

Dhl Helderberg Group

Mostly cherty limestone, with some sandstone and shale. Contains several named stratigraphic units, including the Keyser Formation, which is partly Silurian and includes the Clifton Forge Sandstone and Big Mountain Shale Members.

Stw Tonoloway, Wills Creek, and Williamsport Formations

Includes the thin-bedded platy argillaceous limestones of the Tonoloway, the thin-bedded shale with fossiliferous limestones of the Wills Creek, the Bloomsburg red clastic facies, and the greenish-brown to white Williamsport Sandstone. The Wills Creek contains anhydrite and rock salt, the latter supplying brine from deep wells along the Ohio River.

McKenzie Formation and Clinton Group

Includes the McKenzie Formation, consisting of shale with thin limestone lenses; the dark Rochester Shale; the white Keefer Sandstone; and the Rose Hill predominantly red shale, with thin sandstone interbeds, some of which are called "iron sandstones" from their reddish-brown color and hematite content.

St Tuscarora Sandstone

Medium- to thick-bedded, white to gray or pinkish sandstone, fine to coarse, quartzitic, ridge-forming. Equivalent to the Clinch Sandstone of Tennessee.

Dmu Middle and Upper Devonian, undivided

This unit is predominantly shale, and includes all Devonian beds above the Onesquethaw.

Dbh

Dmb Millboro Shale

Dark gray to black shale facies of eastern West Virginia.

Dmn

Onesquethaw ("Onondaga") Group

Includes the Needmore Shale, Huntersville Chert, and "Onondaga" Limestone, which intertongue with one another. *Not mappable at scale of this map. Included in cross sections with Dooh.*

Dooh

Grouped for some of the accompanying cross sections.

Dohl

DlS Lower Devonian and Silurian, undivided

Grouped for some of the accompanying cross sections.

S Silurian, undivided

Grouped for some of the accompanying cross sections.

Sct

West Virginia Geology

CENOZOIC

Qal Quaternary Alluvium

Alluvial deposits of sand, gravel, silt and clay.

MISSISSIPPIAN SYSTEM

Mp Pocono Group

Predominantly hard gray massive sandstones, with some shale. In the Eastern Panhandle, has been divided into the Hedges, Purslane, and Rockwell Formations.

SYMBOLS

Fault (unclassified)

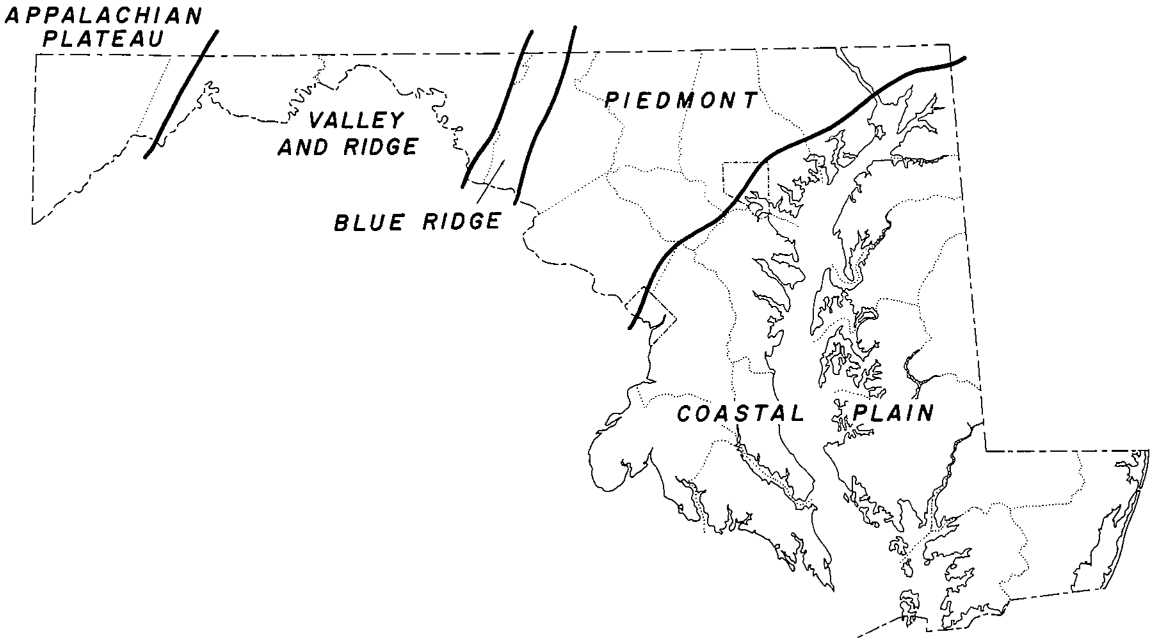
Thrust Fault (teeth on upthrow side)

Anticlinal axis

Synclinal axis

Line west of which structural axes are based primarily on subsurface information.

A BRIEF DESCRIPTION OF THE GEOLOGY OF MARYLAND



Maryland is part of three distinct physiographic regions: (1) the Coastal Plain Province, (2) the Piedmont Province, and (3) the Blue Ridge, Valley and Ridge, and Appalachian Plateau Provinces. These extend in belts of varying width along the eastern edge of the North American continent from Newfoundland to the Gulf of Mexico.

The Coastal Plain Province is underlain by a wedge of unconsolidated sediments including gravel, sand, silt, and clay, which overlaps the rocks of the eastern Piedmont along an irregular line of contact known as the Fall Zone. Eastward, this wedge of sediments thickens to more than 8,000 feet at the Atlantic coast line. Beyond this line is the Continental Shelf, the submerged continuation of the Coastal Plain, which extends eastward for at least another 75 miles where the sediments attain a maximum thickness of about 40,000 feet.

The sediments of the Coastal Plain dip eastward at a low angle, generally less than one degree, and range in age from Triassic to Quaternary. The younger formations crop out successively to the southeast across Southern Maryland and the Eastern Shore. A thin layer of Quaternary gravel and sand covers the older formations throughout much of the area.

Mineral resources of the Coastal Plain are chiefly sand and gravel, and are used as aggregate materials by the construction industry. Clay for brick and other ceramic uses is also important. Small deposits of iron ore are of historical interest. Plentiful supplies of ground water are available from a number of aquifers throughout much of the region.

The Piedmont Province is composed of hard, crystalline igneous and metamorphic rocks and extends from the inner edge of the Coastal Plain westward to Catoctin Mountain, the eastern boundary of the Blue Ridge Province. Bedrock in the eastern part of the Piedmont consists of schist, gneiss, gabbro, and other highly metamorphosed sedimentary and igneous rocks of probable volcanic origin. In several places these rocks have been intruded by granitic plutons and pegmatites. Deep drilling has revealed that similar metamorphic and igneous rocks underlie the sedimentary rocks of the Coastal Plain. Several domal uplifts of Precambrian gneiss mantled with quartzite, marble, and schist are present in Baltimore County and in parts of adjacent counties. Differential erosion of these contrasting rock types has produced a distinctive topography in this part of the Piedmont.

The rocks of the western part of the Piedmont are diverse and include phyllite, slate, marble, and moderately to slightly metamorphosed volcanic rocks. In central Frederick County the relatively flat Frederick Valley is developed on Cambrian and Ordovician limestone and dolomite. Gently undulating plains underlain by unmetamorphosed bedrock of Triassic red shale, siltstone, and sandstone occur in three areas in the western Piedmont.

The Piedmont Province contains a variety of mineral resources. Formerly, building stone, slate, and small deposits of non-metallic minerals, base-metal sulfides, gold, chromite, and iron ore were mined. Currently, crushed stone is important for aggregate, cement, and lime. Small to moderate supplies of ground water are available throughout the region, but favorable geological conditions locally may provide larger amounts.

Unlike the Coastal Plain and Piedmont Provinces, the Blue Ridge, Valley and Ridge, and Appalachian Plateau Provinces are underlain mainly by folded and faulted sedimentary rocks. The rocks of the Blue Ridge Province in western Frederick County are exposed in a large anticlinal fold whose limbs are represented by Catoctin Mountain and South Mountain. These two ridges are formed by Lower Cambrian quartzite, a rock which is very resistant to the attack of weathering and erosion. A broad valley floored by Precambrian gneiss and volcanic rock lies in the core of the anticline between the two ridges.

The Valley and Ridge Province between South Mountain in Washington County and Dans Mountain in western Allegany County contains strongly folded and faulted sedimentary rocks. In the eastern part of the region, a wide, open valley called the Great Valley, or in Maryland, the Hagerstown Valley, is formed on Cambrian and Ordovician limestone and dolomite. West of Powell Mountain, a more rugged terrain has developed upon shale and sandstone bedrock which ranges in age from Silurian to Mississippian. Some of the valleys in this region are underlain by Silurian and Devonian limestones.

For many years the limestone formations have been used as local sources of agricultural lime and building stone. Modern uses include crushed stone for aggregate and cement. A pure, white sandstone in the western region of the province is suitable for glass manufacturing.

The Appalachian Plateau Province includes that part of Allegany County west of Dans Mountain and all of Garrett County, the westernmost county in Maryland. The bedrock of this region consists principally of gently folded shale, siltstone, and sandstone. Folding has produced elongated arches across the region which expose Devonian rocks at the surface. Most of the natural gas fields in Maryland are associated with these anticlinal folds in the Appalachian Plateau. In the intervening synclinal basins, coal-bearing strata of Pennsylvanian and Permian ages are preserved.

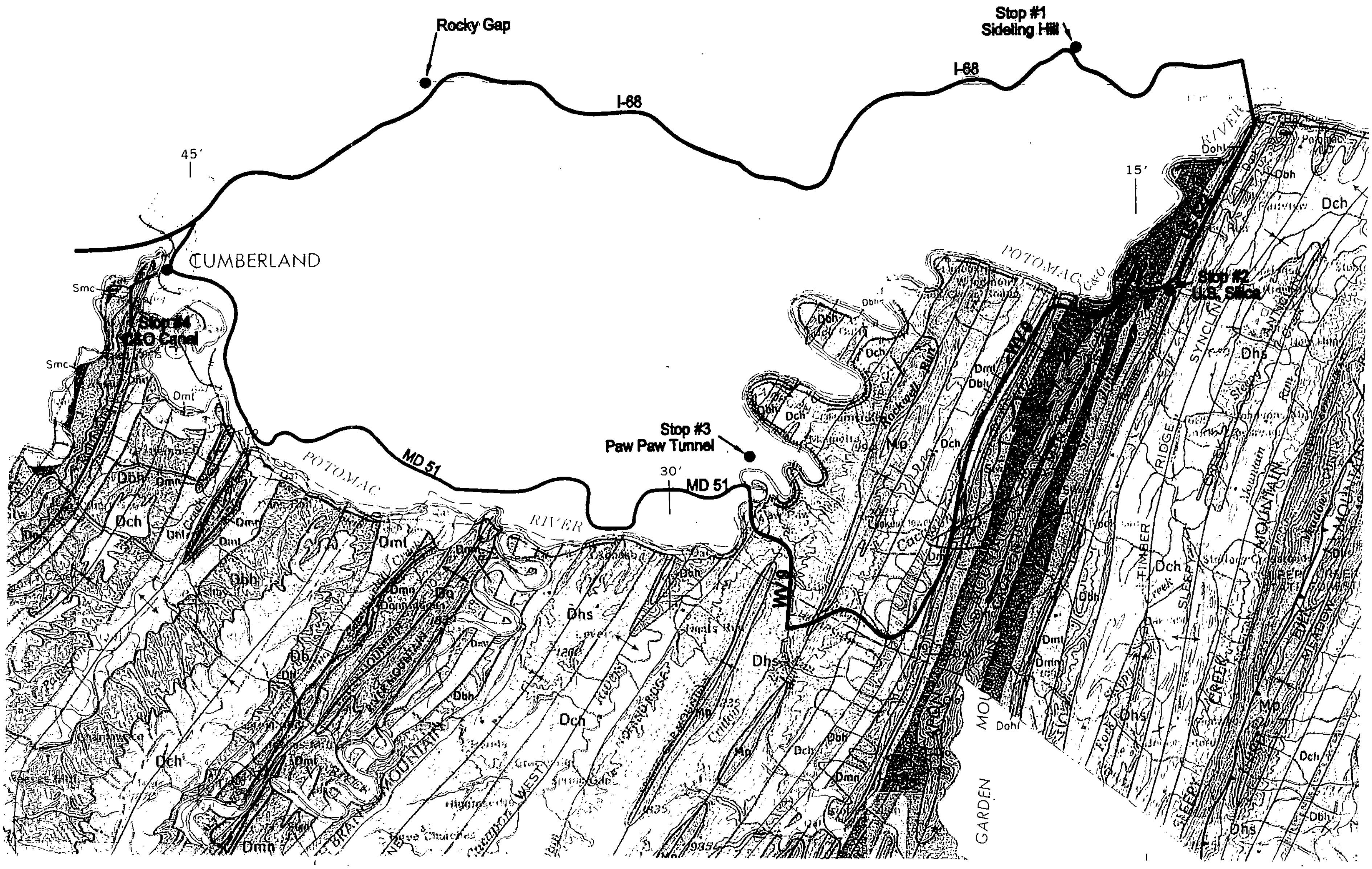
The sedimentary rocks of the Blue Ridge, Valley and Ridge, and Appalachian Plateau Provinces yield small to moderate supplies of ground water. Under favorable conditions large amounts may occur.

Jonathan Edwards, Jr. 1981
Geologist

STATE OF MARYLAND
DEPARTMENT OF NATURAL RESOURCES

Prepared by the
MARYLAND GEOLOGICAL SURVEY

Baltimore, Maryland 21218



West Virginia Geology

Scale: 1 inch = approx. 4 miles

Source: Geologic Map of West Virginia, W. Va. Geological and Economic Survey, 1968

SUMMARY - FIELD TRIP LOG
HGS FIELD TRIP ITINERARY
MAY 17, 2001

Eastbound from Rocky Gap (RGL)

ITINERARY	CONSTANTS	VARIABLES	ACTUAL TIME	EFFECTIVE STOP TIME HOURS & MINUTES	STOP NUMBER
Board Bus	6:45 a.m.		6:45 a.m.	15 minutes	
Leave RGL*	7:00 a.m.		7:00 a.m.		
Trvl to SHEC*	0 + 37 min.				
Arrive SHEC			7:37 a.m.		
Time at SHEC		1 hour 23 min		0 + 43 min.	**(1st Stop)**
Leave SHEC		**(A)**	9:00 a.m.		
Travel to USS*	0 + 13 min.				
Arrive USS			9:13 a.m.		
Time @ USS		2 hours 12 min		1Hr + 52 min.	**(2nd Stop)**
Leave USS		**(B)**	11:25 am		
Trvl to PPTun*	0 + 43 min.				
LunchBreak			12:08 pm		
Time @ Lunch		1 hour 22 min		1 Hr. 10 min.	**(Lunchtime)**
Leave Lunch		**(C)**	1:30 p.m.	**(D)**	
Tvl to Tun.Ent.		15 minutes			
Arrive PPTun.			1:45 p.m.		**(3rd Stop)**
Time @ Tun.		2 Hrs. 5 min.		2 Hrs 5 min.	
Leave Tunnel		**(E)**	3:50 p.m.		
Tvl to C&O	0 + 31 min.				
Arrive C&O			4:22 p.m.		**(4th Stop)**
Time @ C&O		0 + 18 min.		0 + 18 min.	
Leave C&O		**(F)**	4:40 pm		
Travel to RGL	0 + 20 min.				
Arrive RGL	5:00 p.m.		5:00 pm		
TOTALS	**2 HRS.24MIN.**	**1 HR. 20 MIN.**	**10 HRS**	**6 HRS.7 MN.**	**(4 STOPS)**

(A) Includes 20 minutes to load and leave - 20 minutes to cross pedestrian overpass
(B) Includes 20 minutes to load and leave
(C) Includes 10 minutes leaving bus
(D) Includes 15 minute talk which overlaps last part of lunch at PawPaw tunnel
(E) Includes 20 minute rest stop/drinks and snacks - 10 minute loading
(F) No loading/leaving bus

*** Rocky Gap Lodge**
*** Sideling Hill Exhibit Center**
*** US Silica**
*** PawPaw Tunnel**

SUMMARY - TOUR GUIDES
HGS FIELD TRIP ITINERARY
MAY 17, 2001

Eastbound from Rocky Gap (RGL)

STOP #1 - 43 Minutes

MILEAGE		ESTIMATED TIME		ACTIVITY
(Incremental)	(Cumulative)	Hour:Minute	(Clock)	Location / Direction
0 0	0 0	(-)0:30 (-)0.05	6:30a.m. 6:55a.m.	Buses arrive Rocky Gap Lodge Field Trip Guides monitor loading of buses and make final head counts for their buses
0	0	0	7:00 a.m.	Buses close door, leave RGL
0.1	0.1	0:01	----	Left at Stop Sign
0.3	0.4	0:01	----	Right at Stop Sign; Cross overpass over I-68
0.2	0.6	0:02	7:02 a.m.	Left onto I-68 ramp eastbound
1.1	1.7	0:04	7:04 a.m.	Cross Martin Mountain Devonian Oriskany Formation Anticline Quarry south of I-68
3.8	5.5	0:08	7:08 a.m.	Cross Flintstone Creek; outcrop on left(north) contains stromotoporoid reefs in the Silurian/Devonian Keyser Limestone on Warrior Mtn.
1.2	6.7	0:10	7:10 a.m.	Cross Town Creek which meanders through a broad, picturesque valley underlain by shales and limestones of the Lower Devonian.
1.6	8.3	0:12	7:12 a.m.	Cross Polish Mtn.-Devonian Shales and Siltstones of Greenland Gap Formation in a syncline.
8.3	16.6	0:20	7:20a.m.	Cross Town Hill, a twin syncline to Sideling Hill exposing Mississippian Purslane and Rockwell Formations of shales, sandstones and conglomerates.
4.8	21.4	0:25	7:25a.m.	Cross Sideling Creek, enter Washington County.
0.1	21.5	0:26	7:26a.m.	Devonian Redbeds of the Hampshire Formation outcrop on the left (north) side of I-68.
2.1	24.1	0:29	7:29a.m.	Pass through Sideling Hill Road cut, eastbound.
3	27.1	0:33	7:33a.m.	180 degree turnaround at Woodmont Road. (Exit 77 on I-68)
3	30.1	0:37	7:37a.m.	Arrive Sideling Hill Exhibit Center (SHEC) parking lot. (Westbound)
		0:47	7:47a.m.	Leave bus,enter Exhibit Center, self-guided tour of Center and outdoor waysides.

SUMMARY - FIELD TRIP LOG

HGS FIELD TRIP ITINERARY
MAY 17, 2001

Eastbound from Rocky Gap (RGL)

		STOP#2 - 1 Hr. 52 Min.		
MILEAGE		ESTIMATED TIME		ACTIVITY
(Incremental)	(Cumulative)	Hour:Minute	(Clock)	Location / Direction
		(-)0:30	8:30a.m.	Leave Exhibit Center, cross pedestrian overpass over I-68, walk down path to buses parked eastbound. Field Trip Guides monitor loading of buses.
0	0	0:00	9:00a.m.	Field Trip Guides make final head counts, close doors, leave SHEC parking lot eastbound.
4.2	4.2	0:05	9:05a.m.	Devonian Oriskany Formation outcrops in quarry on right(south)before Sandy Mile Road overpass. Devonian Keyser Limestone outcrops in second quarry past overpass on right(south) of I-68.
2.3	6.5	0:09	9:09a.m.	Right on U.S.522 South exit ramp.
0.8	7.3	0:10	9:10a.m.	Cross Potomac River Bridge, entering West Virginia, leaving Maryland.
2.6	9.9	0:13	9:13a.m.	Right turn to U.S. Silica Company Quarry in the Devonian Oriskany sandstone.
		0:23	9:23a.m.	Leave bus, enter quarry.

SUMMARY - FIELD TRIP LOG
HGS FIELD TRIP ITINERARY
MAY 17, 2001

Eastbound from Rocky Gap (RGL)

			STOP#3 - 2 Hrs. 4 Min.	
		0:10	11:15a.m.	Field Trip Guides (FTG) monitor loading of buses.
0	0	0	11:25a.m.	FTG make final headcounts, buses leave quarry & turn right (south) on 522.
2.4	2.4	0:05	11:30a.m.	Buses turn right (west) on WV State 9 at Berkeley Springs, WV
2.5	4.9	0:10	11:35a.m.	At Prospect Peak overlook on right, see spectacular view of Potomac River, WV to left - MD to right of river.
		0:13	11:38a.m.	Cross green-painted bridge across Cacapon River.
		0:15	11:40a.m.	Enter Great Cacapon, WV
		0:25	11:50a.m.	Cross Cacapon River (2nd time)
		0:30	11:55a.m.	Cross Cacapon River (3rd time)
		0:33	11:58a.m.	Enter Hampshire County, WV from Morgan County, WV
		0:35	12:00p.m.	Turn right (north) toward PawPaw, WV on WV State 9
		0:40	12:05p.m.	Enter Paw Paw, WV
		0:41	12:06p.m.	Cross Potomac River, reenter MD, follow US 51 west.
		0:42	12:07p.m.	Right turn to parking lot for PawPaw Tunn., C&O Canal, Nat'l Park Service.
		0:43	12:08p.m.	Arrive PawPaw Tunnel parking area.
		1:05	12:20p.m.	Leave bus for lunch.
			12:20p.m. 1:30p.m.	Lunch - 1 Hour 10 Minutes
		0:00	1:15p.m.	15 minute talk by Ranger on Tunnel's history during last 15 min. of lunch
		0:15	1:30p.m.	Begin walk to tunnel along C&O Canal towpath.
		0:30	1:45p.m.	Enter tunnel
		0:51	2:06p.m.	Exit tunnel- dip slopes of Foreknobs Formation(Devonian Chemung)

SUMMARY - FIELD TRIP LOG
HGS FIELD TRIP ITINERARY
MAY 17, 2001

Eastbound from Rocky Gap (RGL)

STOP #3 - CONTINUED

		0:56	2:11p.m.	Far (north) end of walkway, NPS rangers will talk about Geology of tunnel.
		1:26	2:41p.m.	Reenter tunnel.
		1:47	3:02p.m.	Exit tunnel.
		2:04	3:19p.m.	Arrive parking lot, take 20 minute rest stop/soft drinks and snacks
	(-)0:10		3:40p.m.	FTG monitor loading of buses.
		0:00	3:50p.m.	FTG make final headcounts, buses leave parking lot.
		0:01	3:51p.m.	Turn right on US51 westbound.
		0:28	4:19p.m.	Turn left onto Virginia Ave. at intersection on Industrial Rd (US51)
		0:30	4:21p.m.	Turn right on Elder Street.
		0:31	4:22p.m.	Turn right at parking lot for SHA Engineers Field Office.

SUMMARY - FIELD TRIP LOG
HGS FIELD TRIP ITINERARY
MAY 17, 2001

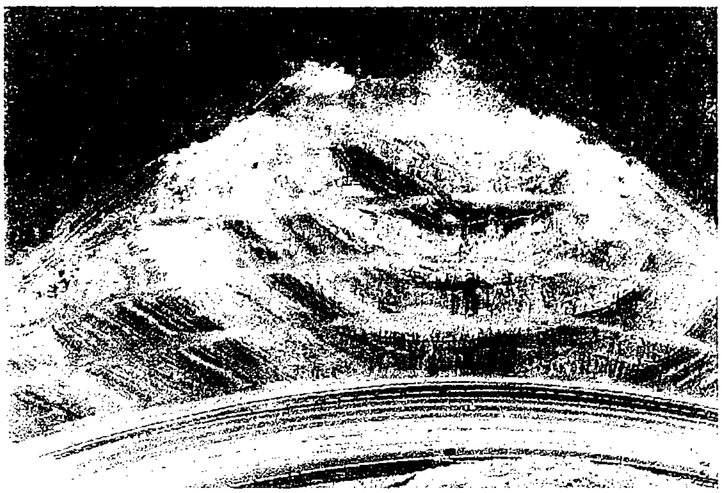
Eastbound from Rocky Gap (RGL)

STOP#4 - (18 Minutes)

0:00	4:40p.m.	Multiple Location Views(from bus) of new highway - Cumberland, MD - Bypass. Leave area from Virginia Ave. and Industrial Road (US51).
0:09	4:49p.m.	Left at Stop Sign.
0:10	4:50p.m.	Right at second Stop Sign.
0:10 (+)	4:50 (+) p.m.	Enter I-68 Eastbound at Exit 43.
0:13	4:53p.m.	Notice Slump Scar on left (north) on slope near Exit 46.
0:18	4:58p.m.	Turn right on I-68 (Exit 50 Ramp), then turn left over overpass to Rocky Gap State Park.
0:19	4:59 (-) p.m.	Turn left at Rocky Gap/Golf Course sign
0:19	4:59 (+) p.m.	Turn right near Rocky Gap Lodge
0:20	5:00p.m.	Arrive Rocky Gap Lodge. Leave Bus.
0:20	5:10p.m.	End of trip - now HIT THE SHOWERS!! "Hope you enjoyed the trip!"

STOP #1: Sideling Hill Road Cut

The Sideling Hill Road Cut along Interstate 68 in Allegany County, Maryland is 360 feet deep and cuts through the eroded trough of a tightly-folding syncline in the Valley and Ridge physiographic province of the Appalachians. Exposed are 850 feet of strata belonging to the Rockwell and Purslane Formations of Mississippian age, about 350 years old. The Rockwell Formation consists principally of interbedded tan and gray-green sandstones, gray-green to dark gray shales, and gray to dark gray silt-stones. The Purslane Formation overlies the Rockwell Formation and consists of gray-green, tan, and white sandstones and quartz-pebble conglomerates with interbedded siltstones, shales, and coaly shales.



Sideling Hill: Elev. 1615, cut on I68, 6 miles west of Hancock, MD. 4.5 million cubic yards of rock were removed from the 380' vertical cut which exposed the 350 million year old section of marine sediments. Younger overlying sediments include thick bedded sandstones, conglomerates, and dark coaly shales.

Photo: Above All Photo Company

The road cut required removal of 4,510,923 cubic yards of stone. Rock from the cut was crushed onsite and used as base material for the roadway. Additional stone from the site was used in the manufacture of 147,000 tons of bituminous concrete used to pave the roadway. Work on the project started in April 1983 and was completed in just under 2.5 years. The cost of the work, including construction of the approach fills, was nearly $21 million. This and other sections of Interstate 68 replaced the historic National Road (Route 40) as the main east/west roadway in western Maryland.

Sideling Hill Road Cut: One of the best rock exposures in the northeastern U.S. is located 6 miles west of Hancock, Washington County, MD on U.S. Rt.I-68. Nearly 850 feet of strata is visible, with an interpretive center and observation area for better understanding.

Photo: Jeff Brant

STOP #2: U.S. Silica Quarry

The U.S. Silica Quarry and the associated processing facility in Berkeley Springs, West Virginia produces high quality silica sand of various textures, much of which is used by optical-glass and foundry customers. The sand comes from the Oriskany Sandstone of early Devonian age. The sandstone forms Warm Spring Ridge which runs from Virginia north/northeasterly through West Virginia and into Pennsylvania. At the quarry, the unit dips steeply to the east.

Looking northeast in the U.S. Silica Quarry at Berkeley Springs, West Virginia. This site is currently being reclaimed with backfill.

As the quarry is approached, two shales are exposed. The first is the thick and tightly-folded Marcellus Shale, and near the quarry proper, the unfolded Needmore Shale which overlies the Oriskany Sandstone. Trilobites can be found in slabs of shale (along the bedding planes) that have slumped down the quarry wall. Specimens of brachiopods (sea-shells) can be found in the Oriskany Sandstone, and with luck, some crinoids (sealilies) and gastropods (snails) might also be found.

A closer view of the Quarry. The southeast dipping sandstones of the Oriskany Formation may be seen in the distance.

STOP#3: Paw Paw Tunnel

Completion of the Paw Paw Tunnel was the final link in the construction of the Chesapeake and Ohio Canal that connected Georgetown to Cumberland, Maryland. The tunnel was necessary to navigate the six-mile stretch of the Potomac River known as the Paw Paw Bends. The original contract called for the construction of the tunnel in two years beginning in 1836 and at a cost of $33,500. Instead, it took 14 years to complete at a cost of $600,000.

Paw Paw Tunnel: The tunnel is three-fifths of a mile in length with a 5-foot wide towpath. By cutting through the mountain, the canal avoided a bend in the Potomac River and saved five miles.

Photo: Jeff Brant

The tunnel was driven from both ends simultaneously, and vertical shafts sunk from the hilltop provided extra faces in each direction. Boring through the Brallier shale was more difficult than anticipated. Advancement of 7 to 8 feet per day was estimated using black powder as the explosive; instead, full crews working three shifts a day achieved 10 to 12 feet per week. Beyond the upstream end of the tunnel, a cut was made through the steeply dipping shale. In this area, freezing and thawing action caused large slabs of rock to slide into the canal and made rock bolting necessary, a practice that has not been fully effective.

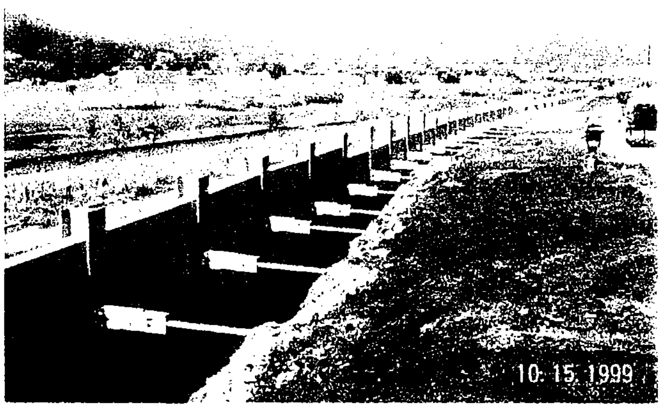
An early photo of the tunnel. The boats used their horns to warn other boats travelling in the opposite direction when they were passing through the tunnel. An incident occurred when 2 boats met inside the tunnel and both captains refused to give passage. They were steadfast for several days before one captain finally conceded to let the other pass.

The brick-lined tunnel is 3,118 feet in length, and its use ceased in 1924 when the canal was abandoned. The walk through the tunnel normally requires 20 minutes each way, not including a delay to study the rock cut.

STOP #4: Canal Parkway

The Canal Parkway is a roadway project constructed through the cooperative efforts of the Maryland State Highway Administration, the National Park Service, the Federal Highway Administration, and CSXT Railroad. It is constructed on land that was originally occupied by the western terminus of the Chesapeake and Ohio (C&O) Canal.

The roadway project is envisioned to address several needs. It will improve access to south Cumberland, it will improve access to the Cumberland Regional Airport, and it is one of the several Canal Development Projects that are designed to improve tourism in the area.



Looking northwest over the tied back wall "C", the Potomac River is visible beyond the old canal towpath. In the distance, you can see the "Narrows" which was the gap in the mountains for the roads and railroads west.

The C&O Canal was originally planned to connect the Potomac River Valley with the Ohio River Valley. Its construction in the mid 19th century coincided with the development of the railroads. By the time the 184 miles of canal were completed from Washington, D.C. to Cumberland, Maryland, the railroads had successfully connected both Baltimore and Washington to the Ohio River Valley, and it was not economically feasible to continue construction of the canal further west.

The canal operated primarily as a coal transporter until the 1920's when flooding of the Potomac River caused great damage. Since the coal mines serviced by the canal were beginning to play out, the railroad that had acquired ownership of the canal could not justify the needed repairs and operations were terminated.

Looking north towards the city of Cumberland, MD. The south ends of completed walls "C" and "B" are shown. Wall "C" is to the left.

STOP #4: Canal Parkway - Cont'd

In 1956, the U.S. Corps of Engineers constructed a flood control program for the City of Cumberland, which consisted of the demolition of the Potomac River dam that supplied the canal, and a realignment of part of the river downstream of the city. The materials excavated from the river were used as backfill for the basin area at the canal terminus. In the 1960's, the C&O Canal became part of the National Park System.

Towards the north end of the project, wall "C", on the left, is a retaining wall supporting the roadway, wall "B" is a barrier wall to keep motor vehicles separated from the railroad traffic.

The 1.6-mile long Canal Parkway is placed near the original canal alignment between the river and the CSXT rail yard. It is designed as a parkway and truck traffic is prohibited with the exception of emergency vehicles. Due to the need for many retaining structures, the $21 million project includes $14 million in structures. Its presence on and near the National Park property dictates that extensive measures would be taken to create an atmosphere consistent with the history of the area. Ultimately, the Corps of Engineers plans to construct a recreation of the old canal at the base of the roadway retaining walls.