62nd HIGHWAY GEOLOGY SYMPOSIUM

www.HighwayGeologySymposium.org

Lexington, Kentucky

July 25th - 28th, 2011

PROCEEDINGS

Hosted By

The Kentucky Geological Survey

The Kentucky Transportation Cabinet, Geotechnical Branch

Coordinated By

University of Kentucky Transportation Center
Technology Transfer Program

UK
UNIVERSITY OF KENTUCKY
Kentucky Transportation Center
Technology Transfer Program

KENTUCKY
TRANSPORTATION
CABINET

UK
UNIVERSITY OF
KENTUCKY
Kentucky Geological Survey

Table of Contents

62nd Highway Geology Symposium

Local Arrangement Committee Members

Henry Mathis – Local Chairman	H.C. Nutting a /Terracon Co.
Tony Beckham	Kentucky Transportation Cabinet
Mike Blevins	Kentucky Transportation Cabinet
Doug Smith	Portland Cement Association
Mike Gragg	Florence & Hutcheson
Craig Lee	Soil and Materials Engineers, Inc.
William Andrews	Kentucky Geological Survey
Dr.John Kiefer	Kentucky Geological Survey-Retired
Mike Lynch	Kentucky Geological Survey
Larry Rhodes	Rhodes Consulting Group
Marwan Ryan	Lexington – Fayette Urban County Government
Richard Wilson	Horn and Associates
Mark Litkenhus	Stantec
Marsha Wilson	Lexington Convention and Visitors Bureau
Lauren Cosentino	Kentucky Transportation Center
Martha Horseman	Kentucky Transportation Center

Special Thanks to

Dr. Jim Cobb	Kentucky Geological Survey
Bart Asher	Kentucky Transportation Cabinet, Geotechnical Branch
Jeff Dean	Oklahoma Department of Transportation
Judy Mathis	

NATIONAL STEERING COMMITTEE OFFICERS 2011

Michael Vierling -
(CHAIRMAN)
Term: 9/2006 until 2010
(YAA) Engineering Geologist
Canal Design Bureau
New York State Thruway Authority
200 Southern Blvd
Albany, NY 12209
E-Mail: **michael_vierling@thruway.state.ny.us**
Phone: (518) 471-4378
Fax: (518) 436-3060

Jeff Dean -
(VICE-CHAIRMAN)
Term: 9/2006 until 2010
Oklahoma DOT
200 NE 21st St.
Oklahoma City, OK 73105
Ph: (405) 521-2677 or (405) 522-0988
Fax: (405) 522-4519
Email: **jdean@odot.org**

Vanessa Bateman -
(SECRETARY)
Term: 9/2006 until 2010
US Army Corps of Engineers,
801 Broadway # A540
Nashville, Tennessee 37203-3816
Ph: (615) 736-7906
vanessa.c.bateman@usace.army.mil

Russell Glass -
(TREASURER)
(Publications & Proceedings)
Retired NCDOT
100 Wolf Cove
Asheville, NC 28804
Ph: (828) 252-2260
Email: **frgeol@aol.com**

NATIONAL STEERING COMMITTEE

MEMBERS 2011

Ken Ashton (Membership)
West VA Geological Survey
P.O. Box 879
Morgantown, WV 26507-0879
Ph: (304) 594-2331
Fax: (304) 594-2575
ashton@geosrv.wvnet.edu

Vanessa Bateman (Secretary)
US Army Corps of Engineers
801 Broadway #A540
Nashville, Tenn. 37203-3816
Ph: (615) 736-3816
vanessa.c.bateman@usace.army.mi

Richard Cross
Golder Associates
RD 1 Box 183A
Solansville, NY 12160
Ph: (518) 868-4820
Cell: (603) 867-4191
dick_Cross@juno.com

John D. Duffy
California Dept. of Transportation
50 Higuera St.
San Luis Obispo, CA 93401
Ph: (805) 527-2275
Fax: (805) 549-3297
John_D_Duffy@dot.ca.gov

Jeff Dean (Vice-Chairman)
Oklahoma Dept of Transportation
200 NE 21st St.
Oklahoma City, OK 73105
Ph: (405) 522-0988
Fax: (405) 522-4519
jdean@odot.org

Tom Eliassen
Transportation Geologist
State of Vermont, Agency of Transportation
Materials & Research Section
National Life Building, Drawer 33
Montpelier, VT 05633
Ph: (802) 828-2516
fax: (802) 828-2792
tom.eliassen@state.vt.us

Marc Fish
Engineering Geologist
WSDOT Geotechnical Division
Environmental & Engineering Programs
Materials Laboratory
PO Box 47365
Olympia, WA 98504-7365
Ph: (360) 709-5498 (office)
(360) 485-5825 (cell)
fishm@wsdot.wa.gov

NATIONAL STEERING COMMITTEE

MEMBERS 2011 CONTINUED

Russell Glass (Treasurer)
(Publications & Proceedings)
NCDOT (Retired)
100 Wolf Cove
Asheville, NC 28804
Ph: (828) 252-2260
frgeol@aol.com

Bob Henthorne
Chief Geologist
Kansas Dept. of Transportation
Materials and Research Center
2300 Van Buren
Topeka, KS 66611-1195
Ph: (785) 291-3860
Fax (785) 296-2526
roberth@ksdot.org

Peter Ingraham
Golder Associates
540 North Commercial Street, Suite 250
Manchester, New Hampshire 03101-1146
Ph: (603) 668-0880
Fax: (603) 668-1199
pingraham@golder.com

Daniel Journeaux
Janod Inc.
34 Beeman Way
P.O. Box 2487
Champlain, NY 12919
Phone: (518) 298-5226
Fax: (450) 424-2614
info@janod.biz

Richard Lane
NHDOT, Bureau of Materials & Research
5 Hazen Drive, Concord
New Hampshire 03301
Phone: (603) 271-3151
Fax: (603) 271-8700
dlane@dot.state.nh.us

Henry Mathis - (By-Laws)
H.C. Nutting A/ Terracon Co.
561 Marblerock Way
Lexington, KY 40503
Cell: (859) 361-8362
hmathis@iglou.com

Harry Moore - (Historian, Appt.)
Tenn. DOT (retired)
Golder Associates
Senior Consultant
3730 Chamblee Tucker Rd.,
Atlanta, GA 30341
Atlanta Office phone:
(770) 496-1893
HLMoore@golder.com

NATIONAL STEERING COMMITTEE

MEMBERS 2011 CONTINUED

John Pilipchuk
NCDOT Geotechnical Engineering Unit
5253 Z-Max Blvd
Harrisburg, NC 28075
Ph: (704) 455-8902
Fax: (704) 455-8912
jpilipchuk@dot.state.nc.us

Nick Priznar
Sr. Eng. Geologist
Arizona D.O.T.
121 N. 21st Ave. 068R
Phoenix AZ, 85009
Ph: (602)712-8089
Fax: (602)712-8138
npriznar@azdot.gov

Erik Rorem
Geobrugg North America, LLC
551 W. Cordova Road, PMB 730
Santa Fe, NM 87505
Phone: (505) 438-6161
Fax: (505) 438-6166
erik.rorem@geobrugg.com

Christopher A. Ruppen (YAA)
Michael Baker Jr., Inc.
4301 Dutch Ridge Rd.
Beaver, PA 15009-9600
Ph: (724) 495-4079
Fax: (724) 495-4017
Cell: (412) 848-2305
cruppen@mbakercorp.com

Stephen Senior
Ministry of Transportation, Ontario
Soils and Aggregates Section
1201 Wilson Ave.
Rm 220, Building C
Downsview, ON M3M IJ6 Canada
Ph: (416) 235-3743
Fax: (416) 235-4101
stephen.senior@mto.gov.on.ca

Michael P. Schulte, P.G.
WYDOT Geology Program
5300 Bishop Blvd.
Cheyenne, WY 82009-3340
Office: (307) 777-4222
Cell: (307) 287-2979
Fax: (307) 777-3994
Mike.Schulte@dot.state.wy.us

Jim Stroud (Appt)
Vulcan Materials Company
Manager of Geological Services
4401 Patterson Ave.
PO Box 4239 Winston-Salem, NC 27105
Office (336) 744-2940
Fax (336) 744-2018
(336) 416-3656 (M)
stroudj@vmcmail.com

NATIONAL STEERING COMMITTEE

MEMBERS 2011 CONTINUED

Steven Sweeney
NYS Canal Corporation
Phone: (518) 334-2511
Email: ssweeney2@nycap.rr.com

John F. Szturo R.G.
HNTB Corporation
KC Office
715 Kirk Drive
Kansas City, MO 64105
Direct Line (816) 527-2275
Fax (816) 472-5013
Cell (913) 530-2579
jszturo@hntb.co

Robert Thommen
Rotec International, LLC
P.O.Box 31536
Santé Fe, NM 87594-1536
Ph: (505) 989-3353
Fax: (505) 984-8868
thommen@www.rotecinternational-usa.com

Michael P. Vierling (Chairman)
Engineering Geologist
Canal Design Bureau
New York State Thruway Authority
200 Southern Blvd
Albany, NY 12209
Phone: (518) 471-4378
Fax: (518) 436-3060
michael_vierling@thruway.state.ny.us

Terry West (Medallion, Emeritus)
Earth and Atmospheric Science Dept.
Purdue University West Lafayette, IN
Ph: (765) 494-3296
Fax: (765) 496-1210
trwest@ purdue.edu

HIGHWAY GEOLOGY SYMPOSIUM
HISTORY, ORGANIZATION, AND FUNCTION

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

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

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

Since the initial meeting, 61 consecutive annual meetings have been held in 32 different states. Between 1950 and 1962, the meetings were east of the Mississippi River, with Virginia, West Virginia, Ohio, Maryland, North Carolina, Pennsylvania, Georgia, Florida,, and Tennessee serving as host state.

In 1962, the symposium moved west for the first time to Phoenix, Arizona where the 13th annual HGS meeting was held. Since then it has alternated, for the most part, back and forth from the east to the west. The Annual Symposium has moved to different location as 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, WV
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX
15th	1964	Rolla, MO	16th	1965	Lexington, KY
17th	1966	Ames, IA	18th	1967	Lafayette, IN
19th	1968	Morgantown, WV	20th	1969	Urbana, IL
21st	1970	Lawrence, KS	22nd	1971	Norman, OK
23rd	1972	Old Point Comfort, VA	24th	1973	Sheridan, WY

25th	1974	Raleigh, NC	26th	1975	Coeur d'Alene, ID
27th	1976	Orlando, FL	28th	1977	Rapid City, SD
29th	1978	Annapolis, MD	30th	1979	Portland, OR
31st	1980	Austin, TX	32nd	1981	Gatlinburg, TN
33rd	1982	Vail, CO	34th	1983	Stone Mountain, GA
35th	1984	San Jose, CA	36th	1985	Clarksville, TN
37th	1986	Helena, MT	38th	1987	Pittsburg, PA
39th	1988	Park City, UT	40th	1989	Birmingham, AL
41st	1990	Albuquerque, NM	41st	1991	Albany, NY
43rd	1992	Fayetteville AR	44rd	1993	Tampa, FL
45th	1994	Portland, OR	46th	1995	Charleston, WV
47th	1996	Cody, WY	48th	1997	Knoxville, TN
49th	1998	Prescott, AZ	50th	1999	Roanoke, VA
51st	2000	Seattle, WA	52nd	2001	Cumberland, MD
53rd	2002	San Luis Obispo, CA	54th	2003	Burlington, VT
55th	2004	Kansas City, MO	56th	2005	Wilmington, NC
57th	2006	Breckinridge, CO	58th	2007	Pocono Manor, PA
59th	2008	Santa Fe, NM	60th	2009	Buffalo, NY
61st	2010	Oklahoma City, OK	62nd	2011	Lexington, KY

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

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

Meeting sites are chosen two to four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the steering committee meeting, which is held during the Annual Symposium. Upon selection as the host state for the next symposium, the Chairman of Local Arrangements becomes a member of the Steering Committee for one year.

The symposia are generally scheduled for two and one-half days, with a day-and-a-half for technical papers plus a full day for the field trip. A banquet generally follows the evening after the field trip. In recent years this schedule has been modified to better accommodate climate conditions and tourism benefits.

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

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

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

In Cody, Wyoming the 1996 field trip visited the Chief Joseph Scenic Highway and the Beartooth Uplift in northwest Wyoming. In 1997 the meeting in Tennessee visited the newly constructed future I-26 highway in the Blue Ridge of East Tennessee. The Arizona meeting in 1998 visited the Oak Creek Canyon near Sedona and a mining ghost town at Jerrome, Arizona. The Virginia meeting in 1999 visited the “Smart Road” Project that was under construction. This was a joint research project of the Virginia Department of Transportation and Virginia Tech University. The Seattle Washington meeting in 2000 visited the Mount Rainier area. A stop during the Maryland meeting in 2001 was the Sideling Hill road cut for I-68 which displayed a tightly folded syncline in the Allegheny Mountains.

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

The New Mexico field trip in 2008 included stops at a soil nailed wall along US-285/84 north of Santa Fe and a road cut through the Bandelier Tuff on highway 502 near Los Alamos where rockfall mesh was used to protect against rockfalls. The New York field trip in 2009 visited the Niagara Falls Gorge and the Devil’s Hole Trail. The Oklahoma field trip in 2010 toured the spectacular geological diversity of south central Oklahoma. The stops included the complex geology of the Arbuckle Mountains, I-35 alignment thru a large cut (150’) of the Arbuckle Mountains, lunch at Lake Murray State Park in the Ardmore Basin and beautiful Turner Falls (77’) Overlook which is one of Oklahoma’s most popular tourist destinations.

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

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

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

A number of past members of the national steering committee have been granted Emeritus status. These individuals, usually retired, resigned from the HGS Steering Committee, or are deceased, have made significant contributions to the Highway Geology Symposium. A total of 30 persons have been granted 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 held in Albany, New York were dedicated to Burrell S. Whitlow (1929 - 1990, Virginia).

EMERITUS MEMBERS OF THE STEERING COMMITTEE

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

(* Deceased)

Medallion Award Winners

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

Hugh Chase*	1970
Tom Parrott*	1970
Paul Price*	1970
K.B. Woods*	1971
R.J. Edmondson*	1972
C.S. Mullin*	1974
A.C. Dodson*	1975
Burrell Whitlow*	1978
Bill Sherman	1980
Virgil Burgat*	1981
Henry Mathis	1982
David Royster*	1982
Terry West	1983
Dave Bingham	1984
Vernon Bump	1986
C.W. "Bill" Lovell	1989
Joseph A. Gutierrez	1990
Willard McCasland	1990
W.A. "Bill" Wisner	1991
David Mitchell	1993
Harry Moore	1996
Earl Wright	1997
Russell Glass	1998
Harry Ludowise*	2000
Sam Thornton	2000
Bob Henthorne	2004
Mike Hager	2005
Joseph A. Fischer	2007
Ken Ashton	2008
A. David Martin	2008
Michael Vierling	2009
Richard Cross	2009
John Szturo	2009

(*Deceased)

HIGHWAY GEOLOGY SYMPOSIUM

FUTURE SYMPOSIUM SCHEDULE AND CONTACT LIST

2012	California	Bill Webster	(916) 227-1041	Bill Webster@dot.ca.gov
2013	OPEN	Mike Vierling Chairman HGS	(518) 436-3197	michael_vierling@thruway.state.ny.us

62nd HIGHWAY GEOLOGY

SYMPOSIUM SPONSORS

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

Michael Baker Jr., Inc.
4301 Dutch Ridge Road
Beaver, PA 15009
Phone: (724) 495-4079
Fax: (724) 495-4112
www.mbakercorp.com

Michael Baker Corporation has evolved into one of the leading engineering and energy management firms by consistently solving complex problems for its clients. We view challenges as invitations to innovate.

Baker has been providing geotechnical services since the mid-1950's. Professional geotechnical engineers and geologists are supported by a staff of highly trained assistants. Expertise covers most major facets of geotechnical investigation and design, including geologic reconnaissance, subsurface investigations, geotechnical analysis and design, and geotechnical construction phase services.

Geobrugg North America, LLC
22 Centro Algodones
Algodones, NM 87001
Phone: (505) 771-4080
Fax: (505) 771-4081
www.geobrugg.com

Geobrugg helps protect people and infrastructures from the forces of nature. These technologically mature protection systems are manufactured in Santa Fe, New Mexico; but are installed world-wide. With its basic component, the high-strength steel wire, our natural hazard mitigation systems offer proven protection against rockfalls, avalanches, mudflows, debris flows, and slope failures. With a bedrock belief in total customer satisfaction, Geobrugg strives to provide exceptional assistance to our customers in mitigating against natural hazard problems.

Golder Associates, Inc.
670 N. Commercial Street, Suite 103
Manchester, NH 03101
Phone: (603) 668-0880
Fax: (603) 668-1199
www.golder.com

Golder Associates is a respected, employee-owned, global company providing consulting, design, and construction services in our specialist areas of earth, environment, and the related areas of energy. From 160 offices worldwide, our more than 7,000 employees work with clients to manage their environmental and engineering activities in a technically sound, economically viable, and socially responsible manner.

Hi-Tech Rockfall Construction, Inc.
PO Box 674
Forest Grove, Oregon 97116
Phone: (503) 357-6508
Fax: (503) 357-7323
www.hitechrockfall.com

HI-TECH Rockfall Construction Inc. specializes in rockfall safety problems on highways, railroads, dams, construction sites and many other locations. The principals of **HI-TECH** have over 50 years of combined experience in the rockfall industry. Our highly skilled employees have extensive experience in all types of rockfall drapery systems, rockfall barriers, debris flow barriers, rock scaling, rock bolting, and shotcrete. Commendations for safety excellence from the Associated General Contractors of America and SAIF Corporation indicates the level of attention that **HI-TECH** imparts to all of our employees in this very specialized field.

Gannett Fleming
601 Holiday Drive, Suite 200, Foster
Plaza III, Pittsburg, Pa. 15220
Phone: (412) 922-5575
Fax: (412) 922-3717
www.gannettfleming.com

Gannett Fleming is a global infrastructure firm focused on planning, design, technology, and construction management services for the transportation, environmental, water, and facilities industries. More than 2,000 employees throughout more than 60 offices worldwide provide customized solutions to improve communities. The firm embraces sustainability and innovation in its projects and activities, finding the best solutions and the most efficient processes while being responsible stewards of the environment. We are proud to be ISO 9001:2008 Certified.

Florence & Hutcheson
Michael Gragg, P.G.
2550 Irvin Cobb Drive
Paducah, Ky. 42003
mgragg@flohut.com
Cell: (859) 236-5776
Phone: (800) 688-3920
www.flohut.com

For more than 40 years, Florence & Hutcheson has been meeting and exceeding client expectations with our first-class service and expertise in Civil Engineering and Environmental Management. Our skilled and dedicated professionals thrive on creating solutions and improvements for every situation. Clients are always impressed with the comprehensive scope of services, value and attention to detail Florence & Hutcheson provides. We specialize in the complete spectrum of civil engineering and environmental services for the infrastructure of the built environment with an integrated sensitivity to the natural environment. The firm combines the technical resources and reaches to implement the most complex and extensive projects for every category of client with the flexibility to ensure rapidly responsive, one-to-one client service and proactive, visionary solutions. Our service lines are Transportation, Design-Build, Construction Engineering & Inspection, Geotechnical Investigation & Engineering, Environmental, Water Resources, Water/Wastewater, Civil/Site, General Aviation, Rail Services, Bridge & Structural and Dams & Levees. Let us put our expertise to work for you.

Zonge International, Inc.
7711 W. 6th Ave, Suite G
Lakewood, CO. 80214
Ph: (720)-962-4444
Ph: (800) - GEOPHYSICS
Fx: (720) 962-0417
www.zonge.com
www.1800geophysics.com

Zonge International is an employee-owned geophysical services company, with more than 39 years experience in the development of geophysical instrumentation for application to environmental, geotechnical and hydrologic problems, as well as mineral and energy exploration. Zonge's advanced geophysical technology is used to provide subsurface imaging, information on rock properties, and detection of anomalous subsurface conditions. Zonge's clients include engineering, hydrology, environmental consulting, mining, petroleum and geothermal exploration companies, and federal, state and local governmental agencies.

S&ME
422 Codell Drive
Lexington, Kentucky 40509
(859) 293-551
(859) 299-2481
www.smeinc.com

S&ME is an award winning engineering and environmental services firm employing 1,000 professional and support staff working from 25 offices in Alabama, Florida, Georgia, Kentucky, North Carolina, South Carolina, Tennessee and Virginia. The firm is ranked 97th in Engineering News-Record (ENR) magazine's Top 500 Design Firms and Top 200 Environmental Firms. We have won more than 70 Engineering Excellence Awards from state chapters of the American Council of Engineering Companies. Expertise includes:

- Geotechnical engineering
- Environmental engineering
- Natural and cultural resources
- Occupational health and safety
- Construction materials engineering, testing and quality control
- Water resources and solid waste engineering

Stantec

1409 North Forbes Road
Lexington, Ky. 40511-2050
Ph: (859) 422-3101
Fx: (859) 422-3100
www.Stantec.com

Stantec provides professional consulting services in planning, engineering, architecture, interior design, landscape architecture, surveying, environmental sciences, project management, and project economics for infrastructure and facilities projects. We support public and private sector clients in a diverse range of markets, at every stage, from initial concept and financial feasibility to project completion and beyond. Our services are offered through approximately 10,500 employees operating out of more than 160 locations in North America.

62nd HIGHWAY GEOLOGY SYMPOSIUM EXHIBITORS

Thanks to all participating exhibitors.

AIS Construction, Inc.
6420 Via Real, Suite 6
Carpinteria, CA 93013
Ph: (805) 684-4344
Fax: (805) 566-6534
www.aisconstruction.com

AMS, Inc.
105 Harrison Street
American Falls, Idaho 53211
Phone: (800) 635-7330
Fax: (208) 226-7280
www.ams-samplers.com

Applied Geomechanics
140 Chestnut St.
San Francisco, CA. 94111
Ph.(415) 861-2245
Fax: (415) 861-1448
www.geomechanics.com

Michael Baker Jr., Inc.
4301 Dutch Ridge Road
Beaver, PA 15009
Phone: (724) 495-4079
Fax: (724) 495-4112
www.mbakercorp.com

Baroid Industrial Drilling Products
P.O. Box 1675
Houston, TX 77251
800-735-6075
Phone: (281) 871-4613
Fax: (281) 871-4621
www.baroididp.com

Central Mine Equipment Company
4215 Rider Trail North,
Earth City, MO 63045
Phone: (800) 325-8827
Fax: (314) 291-4880
www.cmeco.com

Gannett Fleming
601 Holiday Drive, Suite 200, Foster Plaza III
Pittsburg, PA 15220
Phone: (412) 922-5575
Fax: (412) 922-3717
www.gannettfleming.com

Geobrugg North America, LLC
22 Centro Algodones
Algodones, NM 87001
Phone: (505) 771-4080
Fax: (505) 771-4081
www.geobrugg.com

Geokon, Inc.
48 Spencer Street
Lebanon, NH 03766
Phone: (603) 448-1562
Fax: (603) 448-3216
www.geokon.com

Golder Associates, Inc.
670 N. Commercial Street, Suite 103
Manchester, NH 03101
Phone: (603) 668-0880
Fax: (603) 668-1199
www.golder.com

Hi-Tech Rockfall Construction, Inc.
PO Box 674
Forest Grove, Oregon 97116
Phone: (503) 357-6508
Fax: (503) 357-7323
www.hitechrockfall.com

Hayward Baker
Geotechnical Construction
515 Nine North Ct.
Alpharetta, Ga. 30004
Ph: (770) 645-9400
www.haywardbaker.com

Carmeuse Lime & Stone
3600 Neville Road
Pittsburg, Pa. 15225
Ph: (412) 777-0706
www.carmeusena.com

Horn and Associates, Inc.
216 North Main Street
Winchester, KY 40391
Phone: (859) 744-2232
Fax: (859) 744-5892
www.hornandassociates.com

Janod, Inc.
34 Beeman Way,
Champlain, NY 12919
Phone: (518)298-5226
Fax: (450) 424-2614
www.janod.biz

Maccaferri, Inc.
10303 Governor Lane Boulevard
Williamsport, Maryland 21795
Phone: (301) 223-6910
Fax: (301) 223 4590
www.maccaferri-usa.com

Soil Nail Launcher, Inc.
2841 North Avenue
Grand Junction, CO 81501
Phone: (970) 210-6170
Fax: (970) 245-7737
www.soilnaillauncher.com

Monotube Pile Corp.
1825 16th St. N.E., P.O. Box 7339
Canton, Ohio 44705
Phone: (330-454-6111)
Fax: (330-454-1572)
www.monotube.com

TenCate Geosynthetics, N.A.
14064 S. Hagen St.
Olathe, KS 66062
Phone: (913) 909-7150
Fax: (831) 665-5970
www.tencate.com

Terracon Consultants, Inc.
611 Luken Park Drive
Cincinnati, Ohio 45226
Phone: (513) 321-5816
www.terracon.com

Williams Form Engineering, Corp.
8165 Graphic Drive
Belmont Mi. 49306
Phone: (616) 866-0815
Williamsform.com

Zonge International, Inc.
7711 W. 6th Ave. Suite G
Lakewood, Co 80214
Phone: (720) 962-4444
Fax: (720) 962-0417
www.zonge.com
www.1800geophysics.com

Mountain Management / Igor Paramassi
Financial Plaza Bldg.
1135 Terminal Way, Suite 106
Reno, Nevada 89502
Ph: (866) 466-7223
Fax: (450) 455-8762
info@mountainmanagement.biz

Pacific Blasting & Demolition LTD.
3183 Norland Avenue
Burnaby, BC
Ph: (604) 291-1255
Fax: (604) 291-2813
ron@pacificblasting.com

62nd Highway Geology Symposium
Agenda

Monday, July 25th

12:00 AM to 5:30 PM

HGS Registration is open
Location: 2nd Floor top of escalator

12:00 PM - 5:00 PM

TRB Session – Geophysical Explorations, Non-Destructive Evaluation and Monitoring Techniques for Landslides, Rockfalls, and other Geohazards
Location: Bluegrass B

6:30 PM to 8:00 PM

Welcome Reception/ Ice Breaker Social
Sponsored by: Zonge International Inc.
Location: Exhibit Area

Tuesday, July 26th

6:30 AM to 8:00 AM

Continental Breakfast
Location: Exhibit Area

7:00 AM to 12:00 PM

HGS Registration is open

8:00 AM to 8:30 AM

Welcome and Opening Remarks
Henry Mathis-Local Arrangements Committee Chairman
Linda Gorton- Lexington Vice Mayor
Steve Waddle- State Highway Engineer - Kentucky Transportation Cabinet
Location: Salon B and Salon C

8:30 AM – 9:00 AM

Geology of Kentucky - Jim Cobb, Ph.D. State Geologist, Director of the Kentucky Geological Survey

9:00 AM to 4:00 PM

Guest Field Trip: Shaker Village Tour and Dixie Belle River Boat Ride

Location: Side entrance of Hotel at 9:00 AM.

Technical Session I – Geotechnical Asset Management

Moderator: Doug Smith - Portland Cement Association-Consultant

9:00 AM to 9:20 AM

Tennessee's Geohazard Management Program: Moving from Disaster Recovery towards Asset Management - Bateman, Polk, Oliver

9:20 AM to 9:40 AM

Building an Enterprise Geotechnical Database to Support Geologic Mapping Activities- Weisenfluh, Broyles

9:40 AM to 10:00 AM

Geotechnical Asset Management Performance Measures for an Unstable Slope Management Program - Stanley, Pierson

10:00 AM to 10:30 AM

Break

Sponsored by: Hi-Tech Rockfall Construction, Inc.

Location: Exhibit Area

Technical Session II – Unique Geotechnical Issues

Moderator: Larry Rhodes - Rhodes Consulting Group

10:30 AM to 10:50 AM

"How Am I Going To Build On That? An Overview of Ground Improvement Techniques"- Marasa

10:50 AM to 11:10 AM

Planning, Design and Geotech Investigation for the South Lawrence Traffic Way – Henthorne

11:10 AM to 11:30 AM

Identification and Mitigation of an Earth Fissure Zone – Neely, Reynolds

11:30 AM to 11:50 AM

Slope Stability Instrumentation and Web Monitoring on I-5 (Portland, OR) Keller, Suszek, Hay

11:50 A.M. to 1:20 PM

Lunch

Sponsored by: S&ME, Gannett Fleming, Inc., Florence & Hutcheson, and Stantec

Location: Magnolia Ballroom

<u>Technical Session III – Application of New Technologies</u>

Moderator: Mike Blevins – Kentucky Transportation Cabinet

1:20 PM to 1:40 PM

Transportation Pooled Fund Project of Highway Geotechnical Applications of Ground-Based LIDAR- Kemeny

1:40 PM to 2:00 PM

Advantages of Using Digital Photogrammetry For Rock Cut Slope Design-An Example in The Central Appalachian Mountains- Roman, Knight, Johnson

2:00 PM to 2:20 PM

Landslide and Rock Slope Movement Evaluation using Surface Seismic Methods – Some Case Studies-Rucker

2:20 PM to 2:40 P.M.

Directional Core Drilling for Tunnel Investigation - Lee

2:40 PM to 3:00 PM

Break

Sponsored by: Hi-Tech Rockfall Construction, Inc.

Location: Exhibit Area

<u>Technical Session IV – Slope Issues</u>

Moderator: Craig Lee – S&ME

3:00 PM to 3:20 PM

Soil Nails at Gateway to Nebraska – Sharma, Sapkota

3:20 PM to 3:40 PM

Landslide Stabilization along the Ohio River Using Cantilevered Stub Piers - Srinivasan, Schroeder

3:40 PM to 4:00 PM

Case Studies in Roadway Landslide Repair along Stream Sides, River Banks, Bluffs and other Sensitive Riparian Areas – Barrett

4:00 PM to 4:20 PM

Geology Field Trip Overview – Andrews

ADJOURN FOR THE DAY

4:30 PM to 6:00 PM

National Steering Committee Meeting

Location: Blackberry Lily

Wednesday, July 27th

6:30 AM to 8:00 A.M.

Continental Breakfast

Location: Exhibit Area

8:30 AM to 5:00 PM

Geology Field Trip
Lunch Sponsored by Geobrugg
Field Trip Refreshments Sponsored by Golder Associates

Location: 8:10 AM. Board coaches at side entrance of Hotel

6:00 PM to 7:00 PM

Social Hour

Sponsored by: Hilton Hotel

Location: Exhibit Area

7:00 PM to 9:30 PM

HGS Annual Banquet
Keynote Speaker: John Nicholson - Executive Director of Kentucky Horse Park – "Impact of the Alltech FEI World Equestrian Games and the future of the Kentucky Horse Park"

Location: Salon A and Salon B

Thursday, July 28th

6:30 AM to 8:00 AM

Continental Breakfast

Location: Salon A and Salon B

Technical Session V – Slope Issues (Cont.)

Moderator: Darrin Beckett – Kentucky Transportation Cabinet

8:00 AM to 8:20 AM

Stabilization of A Large Wedge Failure Utilizing A Passive Anchor System, Interstate 40, North Carolina, Pigeon River Gorge – Kuhne, Clark

8:20 AM to 8:40 AM

Rock Slope Remediation, Chester Vermont, Lessons Learned – Eliassen, Ingraham

8:40 AM to 9:00 AM

Using Tensioned, High Strength Steel, Spiral Rope Net Systems to Mitigate Rockfall Hazards – Wagner, Kane, Hitchcock

9:00 AM to 9:20 AM

Influence of Weak Pennsylvanian System Shale in OH and KY on Transportation Projects – Williams, Kistner, Arduz

9:20 AM to 9:40 AM

Discussion of the Roadway Cut−Section in the Hartshorne Formation – Gowen, Oklahoma - Reidenbach, Nevels

9:40 AM to 10:00 AM

Break

Sponsored by: Michael Baker Jr. Inc.

Location: Exhibit Area

Technical Session VI – Karst / Mining

Moderator: Tony Beckham – Kentucky Transportation Cabinet

10:00 AM to 10:20 AM

Karst Geohazards Along Highways – Identification and Mitigation Moore

10:20 AM to 10:40 AM

Route I-70 & 435 Interchange Mine Remediation, Kansas City, Missouri   Szturo, Martens

10:40 AM to 11:00 AM

Characterizing Karst Geology Beneath Proposed Roadways Using Geophysical Methods – Hebert, Mundell

11:00 AM to 11:20 AM

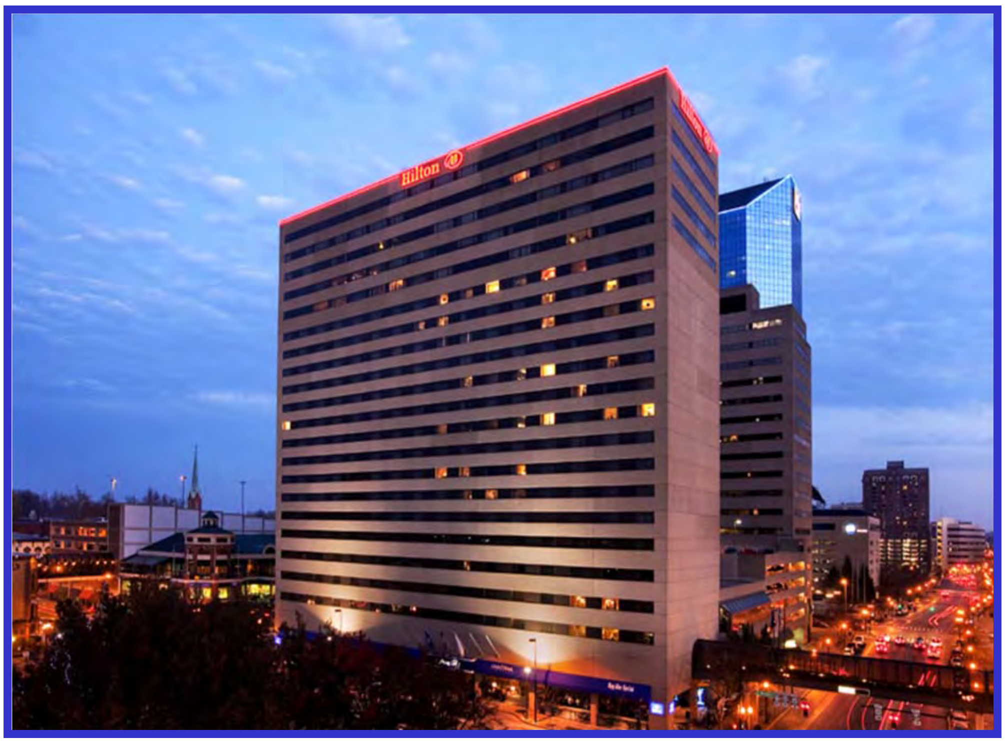
That "Sinking" Feeling: What's Next for Twin Bridges on Extreme Karst   Petrasic, Gaffney, Nevels

11:20 AM to 11:40 AM

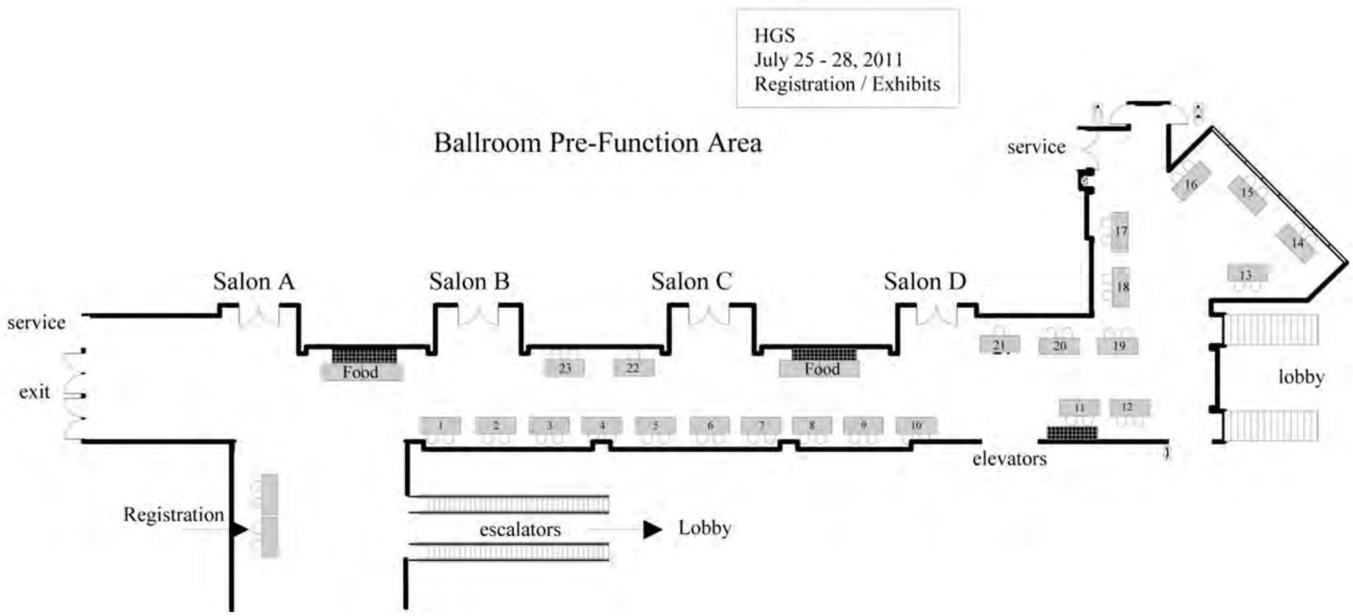
Pavement Settlement Issues and Hydro-Geochemical Water Testing Results for the Cumberland Gap Tunnel – Rister, Dinger

11:40 AM **CLOSING REMARKS-ADJOURN**

Hilton Lexington/Downtown



Hilton Hotel Layout



TRB Mid-year Session at Highway Geology Symposium

Lexington, KY

Monday, July 25th, 2011

"Geophysical Exploration, NDE and Monitoring Techniques for Landslides, Rockfalls and other Geohazards."

Session Description: Geological and geotechnical related hazards often threaten the function of transportation infrastructure and public safety. Assessments of these hazards, and the risks they pose, often start with identifying and characterizing the conditions and mechanisms influencing high-risk events. The objective of this workshop is to share case-studies, methods and fundamental principles of surface and down-hole geophysical exploration, non-destructive evaluation (NDE) and monitoring techniques that can be readily and effectively applied to assist with the identification, characterization and management of conditions associated with landslides, rockfalls and other geohazards.

Sponsoring TRB Committees: AFP10 – Engineering Geology; AFP20 – Exploration and Classification of Earth Materials; AFS20 – Soils and Rock Instrumentation.

Agenda

1:00 PM-1:10 PM

Location: Bluegrass B

Welcome and Introduction

1:10 PM-1:50 PM
Coupled Use of Instrumentation and Geologic History to Assess Movement, Performance, and Stabilization of Large Landslide in Western Pennsylvania- Robert Bachus, GeoSyntec Consultants

1:50 PM-2:30 PM
Induced Polarization and Three-dimensional Seismic at the East Fork Landslide, Wolf Creek Pass, Colorado – Phil Sirles, Zonge Intenational

2:30 PM-2:45 PM - **Break**

2:45 PM -3:25 PM
Instrumentation and Monitoring of a Passive-Anchor Supported Large Wedge Failure, I-40, North Carolina- Jody Kuhne, North Carolina DOT

3:25 PM-4:05 PM
NDE and Performance Monitoring & Rock Bolts- Ken Fishman- McMahon& Mann Consulting Engineers

4:05 PM-4:20 PM - ***Break***

4:20 PM-5:00 PM
Rock Slope Design & Monitoring Utilizing Specialized Instrumentation-Marc Fish, Washington State DOT

5:00 PM - **Adjourn**

Coupled Use of Instrumentation and Geologic History to Assess Movement, Performance, and Stabilization of Large Landslide in Western Pennsylvania

Robert C. Bachus, Ph.D., P.E., Geosyntec Consultants, rbachus@geosyntec.com
Jill Simons, Ph.D., P.E., Geosyntec Consultants
Leslie Griffin, P.E., Geosyntec Consultants

ABSTRACT

A large landslide in Western Pennsylvania adjacent the Ohio River impacted a four-lane highway and three lines of a major railroad. Geotechnical instrumentation has played in significant role in assessing the cause of the failure and in guiding the stabilization efforts that are currently underway. One critical component of the assessment and stabilization effort was to couple the cause of the landslide as well as the stabilization design to the geologic conditions at the site. Assessing and then understanding the site geologic setting was then used to develop the construction phasing strategy. Ongoing construction performance monitoring and the construction performance itself has to date been consistent with the interpreted geologic conditions. This presentation will provide details of the numerous assessments that were made and the results of the performance and construction monitoring activities.

Notes

Induced Polarization and Three-dimensional Seismic at the East Fork Landslide, Wolf Creek Pass, Colorado.

Phil Sirles & James Dean Schofield, Zonge International, Inc.;
And, Khamis Haramy, Central Federal Lands Highway Division/FHWA.

ABSTRACT

A geophysical investigation was undertaken to develop an understanding of the applications of two methods not commonly used on landslides: induced polarization (IP) and three-dimensional (3D) seismic refraction. The underlying goal of the investigation, if successful, is to transfer the technology throughout the FHWA and DOTs. Geophysical surveys were performed over a known landslide on Wolf Creek Pass, Colorado during October 2009 after the most recent movement on the landslide. Movement of the East Fork landslide in May of 2008 reactivated a section approximately 500 meters wide and 800 meters long from within a larger zone of previous displacement. Movement on the East Fork Landslide (EFL) is thought to be primarily a translational mechanism. Induced polarization was tested because clays have a well-known IP response and are often a key component in the behavior and movement of landslides.

Nine IP lines were surveyed transverse to slide movement and modeled in both 2D and 3D with Electrical Resistivity Tomography (ERT). The survey identified many small IP anomalies in a unique arrangement with no discernable pattern. The magnitude of the observed IP anomalies fit into the range of known clay responses. The random distribution of IP features may be the signature of formerly layered material now jumbled through the mass-movement down slope. While the field equipment for the IP survey was in place, resistivity measurements were simultaneously acquired. For the EFL a coherent zone of lower resistivity’s was identified three to fifteen meters below the ground surface which may be indicative of greater water content. The low resistivity zone was seen in the inversions of both the 3D grid and the long 2D profile which extended across the landslide and onto undisturbed ground. Nine seismic refraction lines were surveyed with the geophone locations corresponding to the electrode locations of the IP survey. Seismic travel-times were inverted using seismic refraction tomography.

For the volume of the subsurface analyzed, seismic velocities increased by a factor of six from the near-surface to a maximum roughly 30 meters below the ground surface. A borehole within the slide encountered a breccia at 25 meters below the surface suggesting the maximum velocities found in the inversion were from bedrock. Overall the increase in velocity correlated to increasing depth in a smooth manner; however, several locations had higher gradients and shallower maximums indicating more coherent, solid rock or indurated soils within the body of the landslide. Data processing and interpretation of complex 3D data volumes remain labor intensive however a comprehensive view of the subsurface in a landslide with recent movement appears to be worth the additional effort.

Notes

NDE and Performance Monitoring for Rock Bolts

Kenneth L. Fishman, Ph.D., P.E., McMahon and Mann Consulting Engineers
kfishman@mmce.net,

ABSTRACT

Since the 1970's rock bolts have been used to stabilize and manage the risk of rock falls along the nation's highways. Since many of these installations are approaching service lives of 40 years it is necessary to characterize existing conditions and estimate remaining service life. This presentation will describe implementation of the impulse response (IR) (sometimes referred to as the transient dynamic response (TDR)) technique to probe the lengths of rock bolt installations. This is considered an improvement over the sonic echo technique (SE), which is the existing practice for condition assessment of rock bolts. The IR technique may be useful to identify details of the geometry of an installation as well as material properties including density and dynamic stiffness (i.e. impedance). However, the sonic echo technique only identifies the locations for sources of reflections in the traveling wave, although it can also provide qualitative information on remaining prestress. Implementation of the IR technique for probing rock bolts takes advantage of previous experience by others who have used the IR technique to identify anomalies in drilled shaft installations, but this a novel application for rock bolts. The utility of the method and veracity of the results are demonstrated with respect to a set of specially installed rock bolts whereby installation details are well documented and anomalies are deliberately included. The rock bolts for the test program were recently installed at the site of the Baron Mountain Rock Cut along I-93 near Woodstock, NH. Results from the IR technique are compared to those from the sonic echo (SE). The IR technique appears to be an improvement over the SE technique offering increased resolution and details not available from SE tests. The IR technique identified installation details such as the length of the unbonded zone and relative levels of prestress as well as the locations of various anomalies including voids in the grout along the bonded lengths. This information is useful to describe the vulnerability of the installation with respect to capacity and durability (i.e. corrosion). In situ measurements of corrosion rate and ground resistivity can also be performed to document the performance of existing rock bolts and to estimate durability and remaining service-life.

Notes

Rock Slope Design & Monitoring Utilizing Specialized Instrumentation

Marc Fish, Washington State DOT, fishm@wsdot.wa.gov

ABSTRACT

The Washington State Department of Transportation (WSDOT) has identified over 3300 unstable soil and rock slopes along approximately 7000 miles of its state highway system. Many of these unstable slopes have already been mitigated and designs to mitigate additional slopes are currently being developed. Some of these unstable rock slopes have presented WSDOT's Geotechnical Division with some unique design challenges because of their large slope size, difficult access, and complex rock structure. To develop mitigation designs for these rock slopes we have had to augment our direct on-slope observations and measurements with data collected through the use of long distance ground based lidar and Sirovision. By acquiring data through these techniques, we have been able to obtain block dimensions and discontinuity orientations from inaccessible areas of the slope. This provides us a comprehensive data set to conduct detailed stability analyses and rock fall simulations so more accurate designs and contract documents can be developed. During the construction phase of several projects we have also had to monitor unstable slopes for potential movement. On a major interstate project, we are using ground based lidar in conjunction with remotely accessed in-slope strain gauges and automated motorized total station (AMTS) to measure on-slope survey prisms to identify areas where potential rock slope deformation is occurring. On a second project, we used cross valley radar scanning in conjunction with near slope ground based lidar to monitor for slope movement on a recently activated large scale landslide. We are also using remotely accessed crack gauge data to monitor an open fracture behind a large unstable block and remotely accessed piezometer, inclinometer, and rain gauge data in an attempt to identify relationships between storm events, elevated groundwater levels, and deep seated movement on several known unstable slopes.

Notes

62nd Highway Geology Symposium

Proceedings Paper Abstracts & Notes

Geology of Kentucky

Jim Cobb, Ph.D., Director of the Kentucky Geological Survey, cobb@uky.edu

ABSTRACT

The Kentucky Geological Survey at the University of Kentucky is the state's lead agency on many issues related to rocks and geology. Issues the survey handles include the dissemination of data on mineral resources and geologic maps; and the identification and characterization of karst, seismic, and landslides features for which KGS is building databases. KGS' online web services handles an average of 500 users per day and during a single year provides more than one-million downloads of map, water, resource, publication, and other information to users. Seismic issues are important to Kentucky because of the hazard they pose to human health and safety, and the engineered environment such as highways and bridges. This year is the 200 anniversary of the New Madrid cluster of earthquakes that occurred December 1811 and January and February of 1812. These were large events of M7.0 to perhaps as high as M7.7. Kentucky's current seismicity is low and there are very limited opportunities to see firsthand the effects of large earthquakes. Therefore, KGS has had for six years a scientific exchange program with Gansu Province China. Each year personnel from both countries have made visits to exchange scientific information and observe seismic features in both countries. The 2008 Wenchuan earthquake in Sichuan Province was devastating but did provide an opportunity to see the geologic features it created and the response of the built environment to a large magnitude earthquake.

Dr. Cobb's presentation will cover new technology being employed by KGS to disseminate geotechnical information, current programs being conducted, and some of the geotechnical issues from large magnitude earthquakes in China.

Notes

Tennessee's Geohazard Management Program: Moving from Disaster Recovery towards Asset Management

Vanessa Bateman, US Army Corps of Engineers, vanessa.c.bateman@usace.army.mil
Richard Polk, Tennessee Department of Transportation, GIS Group
Len Oliver, P.E., Tennessee Department of Transportation, Geotechnical Engineering Section

ABSTRACT

Part of the mission of the TDOT's Geotechnical Engineering Section (GES) is to gather and maintain information on geohazards for the Tennessee Department of Transportation department so that these sites may be monitored, repaired or mitigated. Potential geohazards include sinkholes, rockfalls, landslides, settlement problems, potentially acid producing rock and earthquake related problems. This information is used for project design, permitting, mitigation, assistance to maintenance and for TDOT's rockfall mitigation program.

Many stakeholders within the department needed easy and reliable access to the geohazard data maintained by the GES. Unfortunately, until this project began, all of this information other than Rockfall was scattered within paper files. This state of affairs has made the information difficult to access, makes geohazard sites other than rockfall difficult to evaluate, ensuring that the department had to rely on the memories of GES personnel when gathering necessary data and making information sharing extremely difficult and unreliable.

Building on TDOT's Rockfall inventory program, the purpose of the Geohazards Management program is making easy access and visualization of the core geologic hazards available not only to members of TDOT's Geotechnical Engineering Section, but also to other stakeholders throughout the department. Users throughout TDOT can now review information related to specific geologic hazards in Tennessee for field collected measurements / evaluations and where hazard mitigations have been applied or may be necessary using a standard internet browser.

The web browser implementation is supported by an Oracle Spatial database that captures not only inventory information, but stores information of geohazard repair methods and the GES rating of that repair methods' success in the field. It includes links to project photos, reports and assessments. All of which can be requested by the click of a button that sends an e-mail to the relevant GES staff. Specialty maps can be made using GIS programs such as ArcGIS and spatial analysis can also be performed.

While the majority of the records currently within this system are rockfall, all other tracked geohazards are being input into the system as of July 2010 and historical records are being added as time permits. Records displayed by this Geohazard application are stored within an Oracle Spatial Database and are available as read-only layers for use within a full scale GIS program.

Further expansion of the system is expected adding in "pyrite repositories" and other features. Hazard ratings and a "Road Closure Impact Rating," (a measure of the impact on the traffic network during and incident) are also currently supported.

The web browser uses a highway sign motif with signs colored red, yellow and green and sites where mitigation has been performed are easily visible through the use of "badges" on these highway signs. This information will allow the Tennessee Department of Transportation to maintain information on where geohazards have occurred and what it has done to mitigate these sites. It will allow the department to start moving to a more asset management framework where re-inspection of sites occurs systematically; not possible where data on all the sites that need re-inspection is maintained on paper or the memories of staff.

NOTES

Building an Enterprise Geotechnical Database to Support Geologic Mapping Activities

Gerald A. Weisenfluh, Kentucky Geological Survey, jerryw@uky.edu
W.A. Broyles, Kentucky Transportation Cabinet, (Retired)

ABSTRACT

A three year program was conducted to develop an information management system for geotechnical projects and associated data managed by the Geotechnical Branch of the Kentucky Transportation Cabinet. The first objective was to catalog all historical project reports and make them available as electronic documents on the Web. The resulting website contains reports for over 6,000 projects and can be searched by a number of geographic or geotechnical criteria of interest.

The second objective was to develop a data entry system for tracking the progress of ongoing projects. This system gathers information from the KYTC 6-year roadway plan database to initiate new projects and allows for characterization of the type of work being conducted and the organizations and personnel responsible. Dates of completion for a variety of common project tasks can be entered and reports can be generated for managers showing the status of project work. Once projects are completed the final reports are uploaded to the system and become available on the Web.

The third objective was to create a database of drilling and testing information resulting from these projects. A data entry application was developed in the gINT software for managing the data. Project and hole header data is imported from existing electronic files, and additional efficiencies were created by using pull down menus and built in calculations. Once individual projects are completely entered into a gINT database, they are transferred to a SQL-Server Enterprise database for Web distribution using Dataforensics PLog Enterprise software.

NOTES

GEOTECHNICAL ASSET MANAGEMENT PERFORMANCE MEASURES FOR AN UNSTABLE SLOPE MANAGEMENT PROGRAM

David A. Stanley, C.P.G., Alaska DOT&PF, Dave.Stanley@alaska.gov
Larry Pierson, C.E.G. Landslide Technology, Larryp@landslidetechnology.com

ABSTRACT

Geotechnical Asset Management (GAM) is a relatively new concept, first written about ten years ago, but currently receiving renewed attention and interest nationally. Numerous efforts have been undertaken by state and federal agencies to conduct inventories and condition surveys of geotechnical assets such as retaining walls, material sites and unstable slopes. These inventories are consistent with transportation asset management (AM) principles, but most are not integrated within a broader agency AM program and were not seen by the developers at inception in the AM context.

Performance Management (PM) translates strategic agency goals into detailed measures that are tracked to ensure achievement of the goals. Asset Management is a strategic process for operating and maintaining physical assets throughout their lifecycle. These two closely related management frameworks rely on performance measures as essential tools. These measures provide a <u>qualitative</u> means of describing the public or user's perception of asset condition or an agency's service.

For some geotechnical assets, such as buried reinforcements for retaining walls, rock bolts and tie-back anchors, there are on-going efforts consistent with AM principles to predict future performance (remaining service life). In the asset management world, asset condition and performance is compared to performance measures in order to assess alternatives for maintenance, repair or replacement of assets that are not performing. For geotechnical assets, performance measures are largely undeveloped. This presentation provides exemplar performance measures for an unstable slope management program under development for Alaska DOT&PF. The principles and examples presented here can be extended to a broad spectrum of geotechnical assets to assist in development of performance measures that can be integrated with agency AM and PM programs.

NOTES

"How Am I Going To Build On That? An Overview of Ground Improvement Techniques"

Michael J. Marasa, P.E., Hayward Baker Inc.
mjmarasa@haywardbaker

ABSTRACT

Hayward Baker is the largest geotechnical specialty contractor in the country providing services to all types of clients in all types of ground conditions. This topic presents an overview of some of the current geotechnical construction techniques and example applications of various techniques as they relate to site development and construction. Specifically, ground improvement techniques such as vibro replacement, soil mixing, jet grouting and densification will be addressed with an emphasis on recent case histories in the central Kentucky area.

NOTES

Planning, Design and Geotech Investigation for the South Lawrence Traffic Way

Bob Henthrone, P.G., Chief Geologist, Kansas DOT, roberth@ksdot.org

ABSTRACT

Projects for Kansas Department of Transportation, Geotech Section come and go. However one particular project has been around for 25 years. This the South Lawrence Traffic Way Project.

Planning for this project began in 1986. The new roadway was to connect the Kansas Turnpike west of Lawrence to K-10 Highway east of Lawrence. This proposed traffic way would be constructed around the South side of Lawrence. The project became a Legal nightmare almost immediately. The biggest obstacle was the construction of the roadway through a wetlands area. This wetland area is on the eastern half of the project. So we proceeded to construct the west half of the traffic way, thinking the legal battles would soon be over. The west half began construction in 1996 and was completed up to US-59. The bridge over US-59 that was to connect the west portion to the east portion has been termed "The Bridge to Nowhere". The legal issues are proceeding still today.

The Geotech Section was recently given a time frame to complete the investigations for the surficial geology including the wetlands area and the 24 bridge foundations. Given the limited amount of time and workload, major cooperative efforts were undertaken.

NOTES

Identification and Mitigation of an Earth Fissure

Scott D. Neely, P.E., Terracon Consultants, Inc., sdneely@terracon.com
Charles E. Reynolds, G.I.T., Terracon Consultants, Inc. cereynolds@terracon.com

ABSTRACT

The proposed alignment of the 4th segment of Arizona State Route 303L (SR 303L) between Glendale and Peoria Avenues, within the Phoenix metropolitan area is crossed by an existing earth fissure. In addition to the known earth fissure, there are several earth fissures that have been mapped by the Arizona Geological Survey, while other unconfirmed earth fissures have been identified by Terracon Consultants, Inc. during site reconnaissance of the project area. The known earth fissure and other unconfirmed earth fissures cross the mainline, and several ramps and cross roads within the project limits. A methodical approach consisting of numerous steps was agreed upon with the Arizona Department of Transportation to determine which earth fissures could be positively identified and which ones could be removed from further consideration on the unconfirmed earth fissure list.

The investigative information considered in our analyses included:

a. literature review,
    i. geology maps showing subsurface stratigraphy
    ii. earth fissure maps
    iii. lineament analyses of historical aerial photographs
b. historical survey information showing subsidence in the area
c. site reconnaissance,
d. seismic p-wave and ReMi field investigation and analyses,
e. earth fissure trench field investigation,discussions with the ADOT Geotechnical group to decide upon the level of risk the state is willing to accept in design and construction of the highway.

Based on the information obtained from the foregoing efforts, those unconfirmed fissures not showing any substantial evidence for their existence were eliminated from the list of unconfirmed earth fissures. For earth fissures for which positive identification or substantial evidence for their possible existence was obtained, measures to mitigate their potential effects to the roadway were modeled by finite element analyses. Fortunately, none of the earth fissures cross any of the four bridge structures on the project, and therefore, only potential effects to roadway embankments and the west drainage channel were considered for design and construction and determination of mitigation measures.

The results of the finite element analyses indicated that a high modulus material is needed to distribute the strain caused by the potential differential movement that could impact the roadway embankments constructed across the location of the earth fissure. Based on our observations obtained from the earth fissure trench field investigation, the strain occurs over about seven feet as observed by the filled aperture and highly fractured broken zone on either side of the filled aperture. A five foot high zone of material having a Young's modulus of at least 10,000 psi

constructed below the embankments will spread 6-inches of potential differential movement occurring over 7 feet beneath the embankment at the fissure, to over 30 feet at the pavement surface. This governed the design of the mitigative measures developed for the project.

NOTES

Slope Stability Instrumentation and Web Monitoring on I-5 (Portland, OR)

Naia Suszek, Applied Geomechanics, Centennial, Co. nsuszek@geomechanics.com
Jeff Keller, Applied Geomechanics, Centennial, Co. jeff.keller@geomechanics.com
Stephen Hay, Oregon Department of Transportation, stephen.hay@odot.state.or.us

ABSTRACT

The I-5 SW Iowa Street Viaduct Bridge Replacement Project is located one mile south of downtown Portland, Oregon. The project will replace two existing bridge structures and construct six retaining walls for future highway widening. The original structure, built in the 1950’s, traverses through steep terrain where soils are predisposed to saturation induced instability; numerous stability measurements were required throughout the original construction process. A number of historic landslides have also been mapped in the region, and seven landslides have taken place since the 1950's construction effort. Such slides endanger not only the highway superstructure itself, but also residential areas on and below the slope.
During pre-construction stability exploration, nine slope inclinometer tubes were installed in identified slide areas. The inclinometer tubes were monitored with a traversing probe for two years. Fully grouted vibrating wire piezometers were also installed with the inclinometer tubes at predetermined depths. Based on historical slide rate data and inclinometer and piezometer readings, site officials chose to implement an Automated Data Acquisition System (ADAS) for the duration of construction. The current ADAS system consists of six In-Place Inclinometers strings, installed in the original inclinometer tubes, the preexisting grouted-in-place piezometers, and a weather station. Inclinometers at each borehole are spaced 4 to 5 feet apart and span from the ground surface to competent bedrock. The complete ADAS provides near real-time slope stability information in a web based format for remote site monitoring. If a sensor exceeds certain user defined thresholds, an e-mail, text message, and web-page notification are immediately sent to appropriate engineering staff for further investigation and/or evacuation. This paper will further explore the design elements behind the aforementioned Automated Data Acquisition System and report on system function and performance.

NOTES

Transportation Pooled Fund Project of Highway Geotechnical Applications of Ground-Based Lidar

John Kemeny, University of Arizona, kemeny@email.arisona.edu
John Combs, University of Arizona, combs.john@gmail.com

ABSTRACT

This paper will present results from a recent Transportation Pooled Fund project concerning highway geotechnical applications of ground-based LIDAR (TPF, 2008). Dr. John Kemeny from the University of Arizona is the PI on this project, and Christ Dimitroplos is the ADOT administrator of this project. State DOTs involved in the study include Arizona, California, Colorado, New Hampshire, New York, Pennsylvania, Tennessee and Texas. Dr. Keith Turner from the Colorado School of Mines is also involved in the project. As part of the pooled fund project, LIDAR scanning is conducted in each state, and the point clouds are analyzed to look at rock mass characterization, rockfall, slope stability and change detection. The purpose of the pooled fund project is to demonstrate geotechnical applications of ground-based LIDAR for highway slopes, and to train state DOT personnel on the use of point cloud processing software.

A number of important concepts and procedures have been developed from this pooled fund project, including:

- Efficient procedures for scanning highway rock slopes and processing the results for rock mass characterization
- Integrating LIDAR results into other geotechnical software, including software for rockfall trajectory analysis and slope stability.
- The introduction of the Overhang Factor (OHF) to analyze rockfall potential, based on vertical cross sections through the LIDAR point clouds
- Developing change detection procedures for multiple scans of a highway slope and processing the results for ground movement and rockfall detection

Some additional topics that were only briefly investigated in the pooled fund project include:

- Integrating LIDAR results into rockfall hazard rating systems.
- Utilizing point clouds from mobile LIDAR units for geotechnical analyses

The paper will provide details on the above plus examples from several different states.

NOTES

ADVANTAGES OF USING DIGITAL PHOTOGRAMMETRY FOR ROCK CUT SLOPE DESIGN—AN EXAMPLE IN THE CENTRAL APPALACHIAN MOUNTAINS

William M. Roman, P.G. Gannett Fleming, Inc. wroman@gfnet.com
Michael A. Knight, P.G., Gannett Fleming, Inc., mknight@gfnet.com
Robert E. Johnson, Pennsylvania DOT, roejohnson@state.pa.us

ABSTRACT

Roadway realignment to improve line of sight along a curved stretch of U.S. Highway 15 over Bald Eagle Mountain necessitated removal of an existing rockfall fence and modification of an existing rock cut near South Williamsport, Pennsylvania. Digital photographs of the existing rock cut were taken with sufficient overlap to permit development of a three-dimensional, digital model, which encompassed a 420-foot-long section of the cut through inclined strata of the Juniata and Tuscarora Formations. A digitizing tool was used to measure the orientation, magnitude, and location of 238 rock discontinuities (bedding planes and joints) visible in the model. A Brunton Geotransit was used to collect additional measurements from behind the rockfall fence and to validate model measurements. Discontinuity measurements were plotted on stereonets to analyze the potential for plane, toppling, and wedge failures along the proposed rock cut. Using digital photogrammetry on this project provided several advantages over traditional methods. Safety was greatly enhanced since discontinuities in higher parts of the outcrop were measured without rappelling or free-climbing. A great volume of data was collected in a short period of time, reducing exposure to rockfalls and traffic. Discontinuities were accurately measured over a greater length, and discontinuities having limited surface exposure were measured by digitizing their trace across the outcrop model. Using digital photogrammetry reduced the potential for human error in the collection, recording, and processing of data, and facilitated performance of the analyses required for the rock cut slope design.

NOTES

Landslide and Rock Slope Movement Evaluation using Surface Seismic Methods – Some Case Studies

Michael L. Rucker, P.E., AMEC Environmental & Infrastructure, Inc., michael.rucker@amec.com

ABSTRACT

In 30 years of using surface seismic methods to support geotechnical investigations, the author has seen several slope failures that occurred during highway construction in areas of low seismic velocities. These situations, where in-situ characterization information was obtained prior to failure, provide an opportunity to learn through experience about slope failure potential. In other situations, initial seismic results indicating potential problem zones have been confirmed through coring during site characterization. Compression wave (p-wave) refraction has been the primary surface seismic method applied, with 12 channel technology with sledgehammer energy source available for geotechnical work since the 1980s. More recently, surface wave technology has become available to assess shear wave (s-wave) profiles. Since 2002, the author has used a combination of both Refraction Microtremor (ReMi) for s-wave and standard seismic refraction for p-wave, using the same field seismograph and geophone array setup, as a standard characterization tool. Strengths and weaknesses of each method are complementary. The inability of seismic refraction to characterize a velocity reversal (lower velocity underlying higher velocity) is overcome by ReMi's ability to make that characterization; this is of special value when a weaker, lower velocity horizon may be an indicator of a slope failure condition. On the other hand, ReMi's inherent constraint as a 1-dimensional vertical s-wave profiling method is supported by seismic refraction's more detailed 2-dimensional (vertical and lateral) p-wave profiling capability that can provide interpretation constraints to ReMi's otherwise non-unique interpretation solutions. Several case studies are presented.

NOTES

Horizontal Directional Drilling For Exploration of Louisville-Southern Indiana River Bridges East End North and South Bound Tunnels

Craig S. Lee, P.E., Senior Engineer S&ME, Inc. clee@smeinc.com

ABSTRACT

S&ME Inc. was retained by the Kentucky Transportation Cabinet to perform directional core drilling as part of the exploration for the proposed East End Tunnel project in Louisville. The East End Tunnel project is part of the larger Louisville Bridges project. Ohio River Bridges Project addresses the long-term cross-river transportation needs in the Louisville-Southern Indiana region. After detailed analysis that included extensive public outreach and involvement, the Federal Highway Administration (FHWA) authorized the project in September 2003. The project is comprised of a new downtown bridge just east of the Kennedy Bridge (I-65); an east end bridge about eight miles from downtown connecting the Gene Snyder Freeway (KY 841) to the Lee Hamilton Highway (IN 265); and a rebuild of the Kennedy Interchange where I-64, I-65 and I-71 converge in downtown Louisville.

The horizontal directional core drilling and sampling will enable the tunnel designer to evaluate the rock conditions along the crown of the entire alignment of both tunnels and the pillar between the tunnels. Originally, the project proposed to use a 12 foot diameter exploratory tunnel, however, the costs were deemed too high and an alternative to the exploratory tunnel was investigated. S&ME will use a conventional horizontal drilling rig along with the DEVICO directional coring system to advance the coreholes. The tunnel will encounter the Louisville Limestone, Waldron Shale and the Laurel Dolomite along the alignment. The primary goals of the exploration program are to assess the presence and location of discontinuities in the formations, significant Karst features, and evaluate the potential for groundwater. We will perform packer testing along the entire coreholes.

NOTES

Soil Nails at Gateway to Nebraska

Lok M. Sharma, M.S., P.E., Senior Principal, Terracon Consultants, Inc.
lmsharma@terracon.com
Binod K. Sapkota, Ph.D., P.E., Geotechnical Engineering Specialist, Bechtel Oil, Gas and Chemicals, Inc., bksapkot@bechtel.com

ABSTRACT

The Gateway to Nebraska project consists of widening Interstate 80 from west of the existing bridge over the Missouri River to 24th Street in Omaha, Nebraska, near 13th Street. The widening project involved cutting into a hillside on the north side of Interstate 80. Cuts on the order of 46 feet were required necessitating the use of retaining walls to stay within the Interstate right-of-way. Comparison of various options of retaining walls resulted in adopting a three-tiered soil nail retaining wall. The soil nail wall had several advantages over other systems including reduced excavation, smaller impact on the existing slope and flexibility in the facing design. Being a top down construction, soil nail wall also provided speed in construction.

The design of a three-tiered wall presented certain challenges at this site. The subsurface conditions largely involved "Peoria" loess deposits. Use of soil nails in loessial deposit is always looked upon with caution. The sensitive nature of loess, especially loss of strength in the presence of water is of major concern. The Peoria loess at this site was found to be very stiff with dense structure.

The three-tiered soil nail wall was designed using the FHWA design guidelines. Starting from the top tier, each tier of wall was considered as a surcharge on the lower wall for global stability analysis using Limit Equilibrium Method (LEM) computer software "SNAIL" developed by CalTrans. The LEM analysis was verified using the "FLAC" computer program. The FLAC analysis model assessed global stability and also calculated the pull out stresses in soil nails. The lengths of the nails were primarily controlled by global stability. During construction of the wall, extensive verification and proof testing of the nails was performed. Some monitoring data indicates deformations of the wall to be within acceptable level.

Due to high visibility, the finished wall has been architecturally treated with sculpted facing and staining. This paper describes the design process and construction of the wall. Results of stability analyses and limited monitoring data are also presented. Use of LEM backed with FLAC analysis for global stability helped in successfully completing the soil nail wall in loess deposits.

NOTES

Landslide Stabilization along the Ohio River Using Cantilevered Stub Piers

Swaminathan Srinivasan, P.E., M. ASCE, Division Manager, H.C. Nutting a Terracon Company,
ssrinivasan@terracon.com
Jess A. Schroeder, P.E., M. ASCE, Office Manager, H.C. Nutting a Terracon Company,
jaschroeder@terracon.com

ABSTRACT

Landslide activity along U.S. 50 in Cincinnati, Ohio has caused roadway damage for decades. After a necessary closure of 3 lanes due to slope movements, emergency stabilization measures were undertaken to protect the roadway by providing a short-term solution necessitated by ODOT budget constraints.

The landslide shear plane was near the top of a sloping bedrock surface as much as 50 feet below grade. Drilled shafts were installed 40 feet downslope of the roadway shoulder. The shafts were heavily reinforced across the shear plane but steel reinforcing did not extend the full length of the shafts and was stopped well short of the ground surface. The goal was to provide shear resistance across the failure plane, forcing the theoretical failure surface higher into the overburden soils. These “Stub Piers” were installed and found to meet all of the project goals.

The stub piers and surrounding ground were instrumented and preliminary analysis of collected data showed earth pressures and horizontal deflections were over-predicted in the original design. However, time-related effects have yet to be evaluated. Recent indications suggest this option offers much more than a short-term solution to the problem and may in fact, offer long-term support.

NOTES

Case Studies in Roadway Landslide Repair along Stream Sides, River Banks, Bluffs, and other Sensitive Riparian Areas

Colby E. Barrett, Soil Nail Launcher, Inc., colby@snl1.com

ABSTRACT

Landslides and severe erosion along our nation's inland waterways coastlines and can present a real challenge to engineers, planners and designers. Not only are the regulatory hurdles associated with construction in sensitive riparian and coastal zones significant, many of the common tools available either present a large environmental impact or are not robust enough to handle the corrosion, scour, rapid drawdown, wave action, and other erosion processes associated with riparian and coastal sites.

In the context of case studies from five riparian/coastal projects across the United States, this presentation outlines some of the new and innovative erosion control and landslide mitigation methods available to designers that are both robust enough to survive riparian/coastal conditions and specific enough to prevent unnecessary environmental impact. Relevant technologies include traditional soil nailing, high capacity tensioned wire mesh with vegetative inclusions and/or seeded turf reinforcement mats, reinforced architecturally sculpted shotcrete, scour micropiles, and fiberglass composite launched (or ballistic) soil nails/horizontal drains.

The case studies for this presentation include a Pacific coastal bluff erosion project using fiberglass ballistic soil nails and architecturally sculpted/stained shotcrete near Crescent City, California; a landslide repair using launched horizontal drains above a sensitive wetland area in Montana; a riparian landslide repair in Virginia; a lakeshore landslide repair along the shores of Lake Tahoe, California, and a roadway landslide repair using soil nails and integrated micropiles for the Kentucky Transportation Cabinet.

NOTES

Geology Field Trip Review

William "Drew" Andrews, Ph.D., P.G., Head, Geological Mapping Section, Kentucky Geological Survey, wandrews@uky.edu

ABSTRACT

GEOLOGY AT CAMP NELSON

The entrenched valley of the Kentucky River near Camp Nelson provides access to numerous distinctive geologic features. Camp Nelson is one of the best areas in which to observe the nature of the Lexington Fault Zone and nearby Kentucky River Fault Zone and associated fracture systems, and it has been the site of several geologic field trips. The superposition of Hickman Creek on the Lexington Fault Zone provided the break in the river gorge and the natural ford that would become the north–south transportation corridor known as U.S. 27 today. Construction of the highway has provided access to the rocks of the High Bridge Group and Lexington Limestone and their interesting juxtaposition along the fault zone. Numerous karst features are exposed in the limestone roadcuts, and deposits of the ancient Old Kentucky River can be found mantling parts of the landscape. In 1838, a 240-foot-long, covered, wooden wagon bridge, called Hickman Bridge (or the Wernwag Bridge), was built across the Kentucky River at the site. The single-span bridge was built without metal and was considered an engineering triumph at the time, but has since been succeeded by two concrete and steel structures, both of which are still extant near the site. Because of the quality of exposures, accessibility, and the importance of the site in research and geologic education, the Kentucky Society of Professional Geologists named the Camp Nelson area as a Distinguished Geologic Site in 2002.

CIVIL WAR HISTORY AT CAMP NELSON

What made Camp Nelson strategic from the 1700's, when settlers first entered the area, until today is the fact that the area provides an important north–south transportation route across the barrier created by the Kentucky River gorge. In contrast to the relatively flat-lying to gently rolling terrain that characterizes the surrounding Inner Bluegrass Region, the valleys of Kentucky River and its tributaries are steep-walled or entrenched, a situation that creates a natural obstacle to transportation and communication. During the American Civil War, the Union Army used the natural defenses at Camp Nelson to shelter a large supply depot and training center on a 4,000-acre peninsula on the north shoulder of the Kentucky River Palisades. Thousands of Union Army recruits trained here, including over 20,000 U.S. Colored Troops. A large blind karst valley provided shelter for the warehouses of the supply depot; an elaborate water supply system drew water from the Kentucky River for the camp. A short line of forts and entrenchments on the north end of the peninsula provided a solid line of defense for the landward approaches to the camp; Camp Nelson was never attacked, despite numerous Confederate raids through central Kentucky.

NOTES

STABILIZATION OF A LARGE WEDGE FAILURE UTILIZING A PASSIVE ANCHOR SYSTEM, INTERSTATE 40, NORTHCAROLINA, PIGEON RIVER GORGE.

Jody C. Kuhne, North Carolina Dept. of Transportation, jkuhne@ncdot.gov
Shane C. Clark, North Carolina Dept. of Transportation, scclark@ncdot.gov

ABSTRACT

On October 25, 2009, a 50,000 yd^3 rockslide blocked all four lanes of I-40 at MM 2.5. Investigation determined that a remaining 310,000 yd^3 potential wedge failure would need to be removed or stabilized. After a failed bid for excavation, a contract for tensioned anchor stabilization under a 60-day contract was let. This article will explain why design and construction shifted to a passive anchor system, the design considerations selected, utilization of SWEDGE software for design, completion of the project and installation of a monitoring system for future slope conditions and movement.

NOTES

Rock Slope Remediation, Chester Vermont, Lessons Learned

Thomas D. Eliassen, P.G., Transportation Geologist, Vermont Agency of Transportation,
tom.eliassen@state.vt.us
Peter C. Ingraham, P.E. Senior Geological Engineer Associate., Golder Associates Inc.,
pingraham@golder.com

ABSTRACT

In the late 1960's the Vermont Agency of Transportation (VTrans) realigned VT-130 near Chester, Vermont to replace an aging bridge and increase sight distance along the roadway. To construct the modified alignment, a steep rock cut was created resulting in persistent steeply dipping foliation joints slightly oblique to the roadway. As the rock cut aged, it required frequent maintenance to clear fallen rocks, some of which were over 30-tons in size. VTrans mitigated the dangerous rock cut in 2010. During design of the slope repairs and construction, several issues arose that are worth discussing. State DOTs often augment their engineering work force by contracting outside firms for developing plans and specifications. In cases where limited in-house expertise is available, these consulting firms can provide additional staff to support the DOT and ensure state-of-the-art investigative, design and inspection services are applied. Paramount to the successful completion of rock slope and other geotechnical projects is continuity of the project team from design through construction. The dynamic nature of rock mitigation projects can lead to compromised project goals and schedule if this continuity is not adhered to. This paper will provide some of the 'lessons learned' during design and construction that may help other DOTs, municipalities and facility owners in their rock slope mitigation planning efforts. Lessons learned are presented that relate to potential contracting roadblocks, site access issues, construction sequencing and project estimating.

NOTES

Using Tensioned, High Strength Steel, Spiral Rope Net Systems to Mitigate Rockfall Hazards

Dane A.Wagner, GIT Hi-Tech Rockfall Construction,
dane.wagner@hi-techrockfallconstruction.com
William F. KANE, Ph.D, P.G., P.E., Kane GeoTech, Inc.,william.kane@kanegeotech.com
Megan M. Hitchock, Kane GeoTech, Inc.,megan.hitchcock@kanegeotech.com

ABSTRACT

Until recently, solutions for stabilizing large blocks of rock to prevent dangerous rockfall, have typically been limited to rock bolting and scaling of the slope. Depending on the site conditions, these solutions can become expensive and/or impracticable. New methods to retain and control rockfall have been developed. These new approaches include stabilizing large block rockfall with tensioned, high strength steel spiral rope nets, which has shown to be a successful alternative.

This pretensioned spiral rope net system utilizes high-tensile strength steel wire in combination with tensioned rock bolts and steel wire rope boundary cables to actively retain blocks on the slope. The dimensioning of this system is based on sound engineering principles and can be accomplished with an available software program developed for that purpose. Design utilizes the shear strength parameters of the rock and the discontinuities defining the block. Pretension forces are transferred from the spiral rope net system to the rock block. The uniform load transfer by the system reduces the number and depth of rock bolts required in comparison with a typical rock bolting pattern. By retaining blocks on the slope and reducing drilling costs the tensioned spiral rope net system can provide significant economic advantages.

Case studies of the stabilization of large blocks using this approach include successful installations in Kentucky, Colorado, and California.

NOTES

Influence of Weak Pennsylvanian System Shale in OH and KY on Transportation Projects

Richard Williams, Ph.D., P.E., Sr. Geotechnical Engineer, Stantec Consulting Services Inc., Columbus, Ohio

Eric Kistner, P.E., Associate, Stantec Consulting Services Inc. Cincinnati, Ohio

Luis Arduz, P.E., Geotechnical Engineer, Stantec Consulting Services Inc., Lexington, Ky.

ABSTRACT

The increased incidence of slope stability problems in Ohio and Kentucky within the excavated and embankment slopes constructed within or from selected shales of the Pennsylvanian System is widely recognized. Not all Pennsylvanian System bedrock units, even some shales, are considered less competent with regard to slope stability; however, the Pennsylvanian redbed formation shales and resultant colluvium, in particular, within the upper series of formations within the Pennsylvanian system are considered notoriously unstable. Understanding the differences in shale structure and the resultant impact of those structure differences on mass shale strength is a crucial part of geotechnical design and construction of transportation infrastructure within the areas of these states underlain by Pennsylvanian bedrock.

The structure of "typical" shales as compared with the structure of weak Pennsylvanian shales will be discussed. The impact of the structure differences on strength will be detailed, with particular attention given to the strain-softening behavior of the redbed shales. The suggested strength parameters for use in design of slopes excavated within selected redbed shales and/or colluviums will be discussed as well as the parameters for embankment design incorporating redbed shales and/or colluviums. Suggested methods for measurement and/or estimation of redbed shale strength will be presented.

The presentation will also include a discussion of a few roadway projects which dissected Pennsylvanian redbed formation shales and colluviums. Projects include SR 7 in Lawrence County, OH; SR 124 in Meigs County, OH; and, I-64 (Industrial Parkway) in Boyd/Carter/Greenup Counties, KY . The analyses undertaken as well as the lessons learned (good and bad) from these respective projects will be presented.

NOTES

Discussion of the Roadway Cut−Section in the Hartshorne Formation Gowen, Oklahoma

Vincent G. Reidenbach, Ph.D., P.E.
James B. Nevels, Jr., Ph.D., P.E.

ABSTRACT

This paper presents the preliminary field work required to access site for a cut−section investigation (a very contemptuous case), the surface soils and site geology, variation in the site geology along the cut−section extent, laboratory analysis of the cut−section core data, and cut−section slope design. Extraordinary steps were made in an effort to accommodate the property owner namely in the redesigns of the alignment and the minmizing of tree cutting and access road building. The surface soils were very shallow residual soils. The underlying geology was first anticipated to be potentially a thick sandstone of the Hartshorne formation as written up in the geologic descriptions; however, this did not prove to be the case. The sandstone as described in the geologic publications was found at the beginning of the project and where the sandstone was exposed on the surface near the east end of the project turned out to be a shallow cap rock. Thick shale sequences were encountered with only minor sandstone layers found. The laboratory testing of the rock core samples included the following: unit weight, point load strength index, unconfined compressive strength, and slake durability. The shale was observed to air slake when exposed to the atmosphere. Test results on the shale proved that the slake durability was a critical issue. The investigation included 20 core borings and refraction seismic surveys. The cut−section slope design was eventually decided on a 3:1 slope ratio because of the slake durability of the shale.

NOTES

Karst Geohazards Along Highways
Identification and Mitigation

Harry Moore, Senior Consultant, Golder Associates, HLMoore@golder.com

ABSTRACT

Geohazards in general continue to cause multimillion dollar damages to not only infrastructure but to private businesses, homes, and can result in loss of life. Earthquakes, flooding, tsunamis, and sinkhole collapse tend to be the major geohazard issues to be confronted.

Karst geohazards that include sinkhole collapse, sinkhole flooding, groundwater pollution, and sensitive environmental populations associated with karst are the major areas of concern when planning, locating, designing, constructing, and maintaining highway systems in karst areas. The collapse of sinkholes along roadways and highway structures continues to be a major safety issue resulting from karst. Encroachment onto land containing geohazards is becoming more common as the geologically better ground is fast being used-up. Approaching karst geohazards should include concepts such as avoidance, minimizing impacts and implementing mitigation measures. Examples of each concept are discussed and illustrated.

NOTES

Route I-70 & 435 Interchange Mine Remediation, Kansas City, Missouri

John Szturo, R.G., HNTB Corporation, jszturo@hntb.com
Jamie Martens, P.E., HNTB Corporation, jamartens@hntb.com

ABSTRACT

The Missouri Department of Transportation (MoDOT) is planning modifications to the existing Interstate 70 (I-70) and I-435 Interchange located in Kansas City, Missouri. The modifications include the addition of two ramps at US Highway 40. Both ramps include new collector distributor lanes extending along the east and west sides of I-435. The existing roadways and proposed improvements are in the vicinity of existing underground limestone mines present on both sides of I-435. The detailed site investigation included a review of available maps and aerial imagery; exploratory borings to identify the presence of the mine space and to characterize the subsurface materials; laser scanning to create a three-dimensional model of the mine space; and mine and surface visits to map existing site features. Mine structure stability was evaluated with respect to overstressing the limestone pillars, overstressing the tensile capacity of the mine roof beam, bearing capacity failure, and kinematic instability. Both numerical modeling and theoretical calculations indicated areas of unstable to marginally stable mine roof conditions. Based on the results of the mine remediation study, plans and specifications were developed for ground improvement grouting and tunnel improvements. Grouting included placement of low slump grout to form a barrier at the perimeter of the remediation area and infilling with high slump grout. Tunnel improvements consisted of installation of rock bolts, welded-wire fabric, and fiber-reinforced shotcrete. Following completion of the rail tunnel improvements from within the mine space, the ground improvement grouting was conducted from the ground surface. Close monitoring of the grouting activities resulted in over $450,000 of savings in grout volume.

NOTES

CHARACTERIZING KARST GEOLOGY BENEATH PROPOSED ROADWAYS USING GEOPHYSICAL METHODS

Gabriel Hebert, Mundell & Associates, Inc., ghebert@mundellassociates.com

ABSTRACT

The development and expansion of highways on top of karst geology always presents challenges and has the potential for catastrophic failure. Recently, a portion of State Road 56 in Orange County, Indiana known to sit atop of potentially karstic limestone was due to be renovated. The presence of several known solution features (including an artesian spring) along this route, led the lead structural engineer on the project to request a subsurface investigation along six portions of the highway (including three bridge locations) totaling 10,800 linear feet, where the proposed roadway would deviate the most from the existing right-of-way. This portion of Indiana, known as the Mitchell Plain, is underlain by Middle Mississippian limestone formations, which are very solution prone and susceptible to the formation of karst features such as voids and sinkholes, which when left undiscovered, can prove disastrous to roadways. Detection of karst features through traditional test boring methods can be extremely costly and time consuming. Thus, given the length of right-of-way to be scanned, the lead engineer proposed to locate potential solution features by measuring contrasts in the physical properties of the subsurface soil and bedrock through geophysical methods. For this project, a preliminary terrain conductivity survey was performed along each of the six areas to yield information regarding the thin residual soils and shallow bedrock, followed by two-dimensional resistivity profiling to detect any karst features deeper within the bedrock. The end result of the geophysical study gave the lead engineer what he wanted – a detailed subsurface investigation which defined site conditions and located potential hazards, all conducted in a timely and inexpensive manner.

NOTES

That "Sinking" Feeling: What's Next for Twin Bridges on Extreme Karst

Kerry W. Petrasic, P.E., Chief Geotechnical Engineer, Penn. DOT, kpetrasic@state.pa.us
Donald V. Gaffney, P.G., Geotechnical Manager, Michael Baker Jr., Inc., dgaffney@mbakercorp.com
Ryan S. Tinsley, P.G., Senior Engineering Geologist, Michael Baker Jr., Inc., rtinsley@mbakercorp.com

ABSTRACT

Local residents and state agencies in Eastern Pennsylvania are familiar with "That Sinking Feeling," as historically, the Epler and Jacksonburg Formations and their associated sinkholes have impacted our transportation network. State Route 33, and its twin bridges over Bushkill Creek in Northampton County, PA, has been plagued by a highly active and deep "Extreme" karst. Replacement single span bridges and approach embankments have experienced significant documented differential settlement and lateral displacement since PennDOT began periodic monitoring in the form of traditional surveys and inclinometers shortly after construction in 2004. Numerous geotechnical, geophysical, and hydrologic investigations over the last 8 years defined the extreme (greater than 400 feet deep) karst and contributing factors in the vicinity of the S.R. 33 Bridges. A plethora of data has been collected to better understand this extreme karst system.

This paper presents recent investigations and refined understanding of the subsurface conditions, contributing factors, and the apparent shifting uncertainties after each investigation as more information became available. The apparent shifting uncertainties created challenges for presenting and discussing this data with individuals outside the geotechnical discipline. After compilation and evaluation of the extensive subsurface data, conceptual "What's Next" alternatives were developed for the S.R. 33 twin bridges. These alternatives are being evaluated to develop practical solutions that minimize future karst impacts on the S.R. 33 bridges.

NOTES

Pavement Settlement Issues and Hydro-Geochemical Water Testing Results for the Cumberland Gap Tunnel

Brad Rister, P. E., University of Kentucky, Kentucky Transportation Center,
brister@uky.edu
James Dinger, Ph.D., Head of Water Resources, Kentucky Geologic Survey,
dinger@uky.edu

ABSTRACT

Both Ground Penetrating Radar (GPR) surveys and Hydro-Geochemical Water Testing (HGWT) have been performed at the Cumberland Gap Tunnel to determine why the reinforced concrete pavement has settled in various areas throughout both tunnels. To date, approximately 7,300 total square feet of pavement surface has voids beneath it that range from 0.05 to 40 inches in depth. Both GPR and HGWT results indicate that approximately 0.75 to 1.5 cubic yards of limestone sub-base material leaves the tunnel in solution form on a monthly basis. Furthermore, HGWT results indicate that the ground water beneath the tunnels is calcium deficient. Thus allowing the water to dissolve the limestone sub-base. Approximately 500,000 to 1 million gallons of water flows through the tunnel's ground water collection system on a daily basis. Attempts to fix/shore-up the settled pavement areas were performed in 2002, 2007, and 2008. In 2002, UreTek foam was placed beneath approximately 2000 square feet of settled pavement for shoring purposes. In 2007, approximately 150 lineal feet of both pavement and backfill were removed and replaced with inert granite backfill material and a new reinforced concrete pavement. In 2008, approximately 51 cubic yards of cement grout material was placed beneath approximately 7,400 total square feet of settled pavement for shoring purposes.

There are several strategies outlined in this report to address both short-term and long-term remediation. However, there are certain strategies that may prevail over others. It is proposed that grout material should be placed beneath the pavement structure, at an estimated cost of $50,000 to $100,000/year, as a short term assurance measure. It is proposed that approximately 2,800 lineal feet of pavement and backfill material be removed in both tunnels and replaced with an inert granite backfill and a new 10 inch reinforced concrete pavement be installed for a long-term remediation (estimated costs $10,000,000).

NOTES

Tennessee's Geohazard Management Program: Moving from Disaster Recovery towards Asset Management

Vanessa C. Bateman*
US Army Corps of Engineers
801 Broadway #A540
Nashville, TN 37203-3816
615-736-7906
vanessa.c.bateman@usace.army.mil

Len Oliver
Tennessee Department of Transportation
6601 Centennial Boulevard
Nashville, TN 37243-0360
615-350-4133
len.oliver@tn.gov

Richard Polk
Tennessee Department if Transportation
James K. Polk Bldg.
505 Deaderick Street, Suite 700
Nashville, TN 37243
615-532-3875
richard.polk@tn.gov

Prepared for the 62nd Highway Geology Symposium, July, 2011

* Formerly of Tennessee Department of Transportation, Geotechnical Engineering

Acknowledgements

The author(s) would like to thank the individuals for their contributions in the work described:

Frederick Barrell - TDOT Geotechnical
Van Colebank - TDOT GIS
Kim McDonough - TDOT GIS
Matthew Lane - TDOT GIS, WebApplication Designer
Olekan Oiende - TDOT GIS, Mobile Form Designer
Tom Strickland - TDOT IT, Project Manager
Harry Moore* - Golder Associates
William Trolinger - TDOT Materials and Tests Division
Paul Degges - TDOT, Chief Engineer

Many people at the Tennessee Department of Transportation directly or indirectly lead to the success of this project to date and with mitigating the rockfall sites mentioned in this paper. Bill Trolinger led the first effort to gather rockfall data at TDOT and Harry Moore was instrumental over the years in completing many geohazard mitigation projects, most often on an emergency basis.

TDOT's Geotechncial Staff have worked on numerous geohazard projects over the years and their efforts have lead to much safer transportation facilities in Tennessee. Their often "unsung" efforts have resulted in repairs to many sites that would otherwise have posed significant threats to safety, transportation and usability of the road network.

TDOT staff in Construction and Maintenance were involved in the mitigation projects supported by this management system and by the original rockfall management system. Without their efforts, these projects could never have moved forward. Without the tireless efforts of Region 2 Staff, the additional work along US 64 during the closure would have never been completed.

Thanks should also go to Paul Degges, TDOT's Chief Engineer who supported the original rockfall research program and led the effort to implement a Rockfall Mitigation program - finding funds in a scarce transportation budget for these projects. He was also directly responsible for supporting the development of the Geohazards Management System, approving the internal project and allowing staff from TDOT Geotechnical, TDOT IT and TDOT GIS time and resources to work on and develop this system.

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

* Formerly of Tennessee Department of Transportation, Geotechnical Engineering

ABSTRACT

Part of the mission of the TDOT's Geotechnical Engineering Section (GES) is to gather and maintain information on geohazards so that these sites may be monitored, repaired or mitigated. Potential geohazards include sinkholes, rockfalls, landslides, settlement problems, potentially acid producing rock and earthquake related problems. This information is used for project design, permitting, mitigation, assistance to maintenance and for TDOT's rockfall mitigation program. Many stakeholders within the department needed easy and reliable access to the geohazard data maintained by the GES. Unfortunately, until this project began, all of this information other than Rockfall was scattered within paper files. This state of affairs has made the information difficult to access, makes geohazard sites other than rockfall difficult to evaluate, ensuring that the department had to rely on the memories of GES personnel when gathering necessary data and making information sharing extremely difficult and unreliable.

Building on TDOT's Rockfall inventory program, the purpose of the Geohazards Management program is making easy access and visualization of the core geologic hazards available not only to members of TDOT's Geotechnical Engineering Section, but also to other stakeholders throughout the department. Users throughout TDOT can now review information related to specific geologic hazards in Tennessee for field collected measurements / evaluations and where hazard mitigations have been applied or may be necessary using a standard internet browser.

The web browser implementation is supported by an Oracle Spatial database that captures not only inventory information, but stores information of geohazard repair methods and the GES rating of that repair methods' success in the field. It includes links to project photos, reports and assessments. All of which can be requested by the click of a button that sends an e-mail to the relevant GES staff. Specialty maps can be made using GIS programs such as ArcGIS and spatial analysis can also be performed.

While the majority of the records currently within this system are rockfall, all other tracked geohazards are being input into the system as of July 2010 and historical records are added as time permits. Records displayed by this Geohazard application are stored within an Oracle Spatial Database and are available as read-only layers for use within a full scale GIS program. Further expansion of the system is expected adding in "pyrite repositories" and other features. Hazard ratings and a "Road Closure Impact Rating," (a measure of the impact on the traffic network during and incident) are also currently supported.

The web browser uses a highway sign motif with signs colored red, yellow and green and sites where mitigation has been performed are easily visible through the use of "badges" on these highway signs. This information will allow the Tennessee Department of Transportation to maintain information on where geohazards have occurred and what it has done to mitigate these sites. It will allow the department to start moving to a more asset management framework where re-inspection of sites occurs systematically; not possible where data on all the sites that need re-inspection is maintained on paper or the memories of staff.

INTRODUCTION

Part of the mission of the Tennessee Department of Transportations Geotechnical Engineering section is to assist the department in identifying, managing and mitigating geohazards that may be encountered along state maintained roadways, including those on the interstate system. Many geohazards have been commonly recognized in Tennessee including numerous sinkhole and karst issues (Moore, 1988) , rockfall (Mauldon, *et. al*, 2007), landslides (Royster, 1974), potentially acid producing rock (Moore, 1992; Gusek *et. al* , 2008), earthquake related issues with the New Madrid Seismic Zone and the East Tennessee Seismic Zone (USGS, 2011), settlement problems and numerous mapped and unmapped springs and seeps. For over 30 years information was gathered on a site by site basis and all records were kept in a “paper database” - with a file number keyed to the location of the site and the date that a project or incident first occurred. For many years, with a steady workforce, and with the significant challenges in the early days of implementing a computer based inventory inventory system, this was sufficient. With a staff containing members with 30+ years of experience at TDOT, information about older sites could easily be obtained by discussions with staff.

However, over the past 10 years, through retirements of the generation that was responsible for building the majority of the Tennessee Interstate System and with an increasingly mobile workforce, this reliance upon memory and experience of long term staff has become increasingly tenuous. Paper records also have a significant disadvantages when compared to electronic records as they are more difficult to retrieve, to back up and require and enormous effort to collate or to perform spatial analyses. Sites cannot easily be compared with one another as historical records may be located in an archive that is difficult to access. Indeed, this proved to be the case for TDOT when the GES was notified after the fact that it’s main archive stored offsite had been accidentally destroyed. Thus many records before 15 years ago including geotechnical reports, drawings, memoranda and information is no longer available. Fortunately, most of the photographs and many of the mylar drawings had not been sent to the archive. However, with retirements and attrition, the original geologists and engineers that supervised much of this work are no longer with the department and the connection to many of these projects has been lost.

Development of TDOT’s Rockfall Management Program

TDOT recognized this issue early on when it started the development of a rockfall inventory, first in paper format through questionnaires sent to maintenance staff and later through a multi-year research project that was later the basis for implementing TDOT”s Rockfall Mitigation Program. Over 1980 sites with potential rockfall hazards were located statewide, these were assessed and included in the hazard inventory stored in an Oracle Spatial Database (Mauldon, et. al, 2007). A GIS interface with the data, including several web mapping implementations were used to give TDOT staff easy access to the data. This effort was successful in allowing the department to take a active approach to rockfall mitigation, with over 34 sites mitigated since the start of the program in 2007 as of April 2011.

Rockfall Mitigation System used during emergencies

The ability to easily locate information on sites proved invaluable during the winter of 2009/2010 when TDOT experienced several major rockfalls, including the Ocoee rockslide which closed US 64 for 5 months, and the "Dragon Tail Slide" on US 321. The rockfall inventory of US 64 was used, along with TDOT Geotechnical and Maintenance Staff in order to plan out mitigations to be performed during this 5 months on other sections of the roadway. With 44 mapped rockfall sites from Parksville Dam to the Ocoee Whitewater center, it was easy to justify the additional effort to mitigate all the sites possible during the closure, not just the two large slides which received all the attention in the press. Significant upgrades and scaling was completed on a number of sites, including the "15 mph" curve - a site where two 18 wheeler trucks could not pass each other safely. Figure 1 below shows a section of this roadway along with mapped rockfall sites before the 2009 rockslide at LM 17.9 "Site 1" across from Ocoee Dam No. 2.

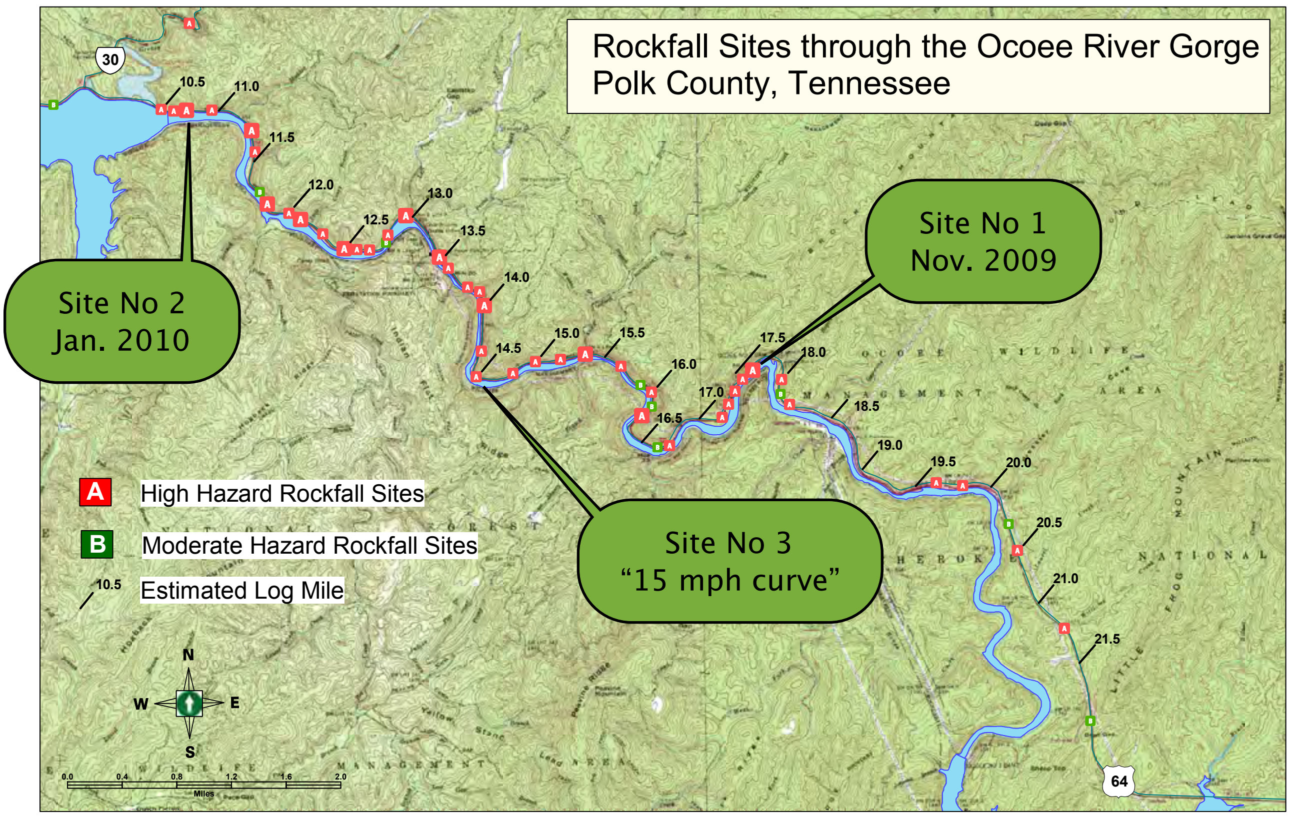


Figure 1. Rockfall Hazards in the Ocoee Gorge

Significantly, sites other than the three shown in figure 1 were chosen not on a "worst-first" basis. Sites were chosen to provide for the highest reduction in hazard given the constraints that the work had to be performed by TDOT Maintenance Personnel and Equipment and could not interfere with the emergency contracts to repair the three major sites and could not extend the time that the roadway was closed. While signifiant rockfall hazards remain on this section of roadway, the efforts of the contractors and TDOT maintenance personnel achieved the goals of hazard reduction and work was completed on more than 15 sites through the gorge while the emergency contracts were on-going. Without the previous work, assessments and inventory, the decision making process and justification for funding significant work through maintenance forces would have been far more difficult. With the Rockfall program TDOT has been able to assess and decide on mitigation strategies more in keeping with the basic principles of Asset Management choosing projects not on an emergency basis, or necessarily sites that are the worst first (FHWA, 2007), but allocating dollars to projects based on a strategic goal. In this case, hazard reduction.

Lessons Learned for the Development of the Geohazards System

All of this experience lead the department to recognize that the same efforts that had been made with rockfall could be expanded to provide a valuable tool to the department for managing other geohazards. Unlike with the rockfall project, a path to the database, GIS layers, server implementation and web mapping application did not have to be blazed. Much of this work had been completed before. While software had continued to improve, it was felt that after discussions with the TDOT GIS group and with personnel from TDOT Geotechnical Engineering that the computer work to accomplish a geohazards system could be completed within the department.

We had moved to an implementation and engineering stage, rather than research. With tighter budgets and very limited research dollars available, this allowed the department to move forward with development using only in-house resources. While it was recognized that this would result in a less comprehensive picture statewide than was provided by rockfall, because the corresponding field work would not be done, it was felt that implementation of the Oracle Spatial database and GIS products would allow the department to have a central storage area where new geohazards could be added as they are identified, and older sites could be added as time and money allowed.

Asset Management Principles and Geohazards

Asset management systems can be put into place in stages and a central system component of a policy based asset management program is the inventory (FHWA, 2007). It is difficult to logically allocate scarce highway funds when there is no complete inventory of assets. Many states have been completing these types of assessments with rockfall inventories over the last 20 years. However, an inventory alone is not sufficient for resource allocation; the next logical step is to provide a program to assess these sites and select mitigation projects which can then be let to contract. With an inventory and a budget, rational site selection becomes possible. TDOT's experience with its rockfall inventory and implementation of a rockfall mitigation program showed that there were significant benefits when an inventory is available to be used to plan out mitigation work based on a known budget.

COMPONENTS OF THE SYSTEM

The central component of the system is an Oracle Spatial database designed to be the central repository of all rockfall, landslides, sinkholes, springs/seeps and settlement areas mapped by the TDOT Geotechnical Engineering Section (GES). All of the rockfall data from previous work was included in the system and the database was expanded to include all of these additional geohazards. Basic information about each geohazard is available, as well as the assessment of the potential threat to the road network.

As was done previously with rockfall all sites were assigned an A, B or C rating depending upon their potential hazard to the roadway network and the traveling public. A sites are those that have a high hazard, B sites have a moderate hazard. C sites are those thought to present little or no hazard to the roadway network. Unlike with the original rockfall inventory, C sites are mapped because they are sites that have been mitigated, are sites that may develop into a problem at some later date or are sites that may require permitting at some point in the future. These sites are marked by a highway sign motif, with each type of geohazard having it's own highway sign. Sites are noted as A, B or C by the red, yellow or green coloring.

Figure 2. Legend showing highway sign motif for geohazards in the system

Location data, GPS coordinates and basic information for what facilities might be affected are the same for all sites. Each geohazard has different information to be gathered for the basic assessment depending upon its' type. For example a sinkhole site would have information on the basic area of the sinkhole, it's depth, geologic setting, whether water was present at the base as well as information on the date of the collapse (if any). The area of roadway or facility affected is also noted. Paper forms were created for each geohazard in the event of a technical difficulty with any of the equipment and to act as backup if an engineer or geologist needed to map a site unexpectedly. All of the data shown on these forms is entered into the Oracle Spatial database one of three ways: 1) data is entered manually through the use of ArcGIS, 2) data is entered manually using ArcPad or 3) data is entered using a GPS form, gathered electronically out in the field and uploaded to the geohazard system when the engineer or geologist returns to the office. Figure 3 shows a typical paper field data gathering form for landslides. A supplementary map showing features and the area of the landslide would also be included.

Landslide Field Data Collection Form

Date		Rater		
GES File No.		LS File No.		

Location	County			County No.		Region	
	Route Number			Co Seq. No.		Sp. Case	
	Begin Log Mile			ADT of Site			
	GPS Coord.	N		Offest From CL in feet			
		W		Reference CL			
Landslide Site Data	Affected Facilities		Road		Bridge Abutment		
			Building		Bridge Pier or Bent		
			Ret Wall		Other	what?:	
	Occurance Date			Length of Slide			
	Depth of Slide			Width of Slide			
	Length of Road			Width of Road involved.			
	Lane Closure		Shoulder	Geologic Formation (s)			
			1 lane				
			2 lanes				
			3 lanes or Total				
	Condition		Active	Slide Type		Fall	
			Reactivated			Topple	
			Inactive			Rotational Slide	
	Slide Notes					Translational Slide	
						Lateral Spread	
						Debris/Mud Flow	
						Complex	
	Water Feature Descriptions						
	Slide Survey Notes						
Photo Notes							

Figure 3. Paper Landslide Field Data Gathering Form

Road Closure Impact Rating

Unlike rockfall, sinkholes landslides and settlement areas do not have a specialized hazard rating assessment unique to the specific geohazard. However, the Rockfall Closure Impact rating (RCI) originally developed for rockfall sites (Bateman, 2004) and used in Tennessee was expanded for use with all mapped rockfall sites. This RCI rating was re-named the “Road Closure Impact Rating” and it is a numerical score that captures the effect of a particular geohazard on the roadway network. Figure 4 below shows the basic rating scheme.

Criteria	Score = 3	Score = 9	Score = 27	Score = 81
ADT (Average Daily Traffic)	Little Traffic ADT>300	Some Traffic ADT 300-1000	Moderate Traffic ADT 1000-3000	Major Traffic ADT >3000
Impedance	Shoulder	1 lane	2 lanes	>=3 lanes or Total
Impedance Duration	Hours	1 Day	Days	Weeks
Detour Length	Very Short <1 mile or lane still open	Short 1-2 miles	Medium 3-4 miles	Long or None >4 miles
Facility Degradation RF-DF= DOF	0	1	2	3

DOF=Degree of Facility (RF=Degree of Road Facility; DF= Degree of Detour Facility)
0 = Local Roads / 1 Lane Road
1 = 2 Lane, no shoulder
2 = 2 lane, adequate shoulder
3 = 3 lane
4 = 4 lane
5 = 4 lane, divided highway, 5 lane highway
6 = Interstate

Figure 4. Road Closure Impact Rating

Each site is rated according to the expected “Impedance,” that is effect on the roadway, the length of time that the roadway is likely to be effected by the geohazard and several factors which capture the significance of a closure at a specific location. The higher the score, the more impact a road or facility closure will have at a particular location. For example an interstate which has a total lane closure lasting weeks that results in a long detour on a 2 lane highway would have the highest score. Where a total closure on a small two lane road with little traffic would have a much smaller score. As geohazard sites are located and mapped, the RCI score is assessed. The total RCI score is the sum of the score for all 5 criteria shown in the chart above. The score is a rating that reflects an geotechnical engineer or geologists assessment of the extent and duration of potential road closures that can be caused by the geohazard.

All rockfall sites continue to have both the TDOT Rockfall Hazard Rating (TRHRS) as well as an RCI score. For landslides, sinkholes and settlement areas basic data for each geohazard is gathered and the RCI score is completed during field assessments.

Superhazard and GIS Mapping

One concept that was significantly different from the original rockfall database and system was the inclusion of the "superhazard." This was a means within the database and within the spatial GIS maps to include a point location for all geohazards that are mapped and to allow for multiple sites to be included as one point on a map.

The new system allows for mapping of sites as points, lines or polygons. Sinkholes for examples are mapped as areas, landslides are mapped as areas (polygons) with features such as scarps mapped as lines. Toe bulges and wet areas are mapped on a landslide as areas, but a spring or seep on a landslide would be mapped as a point. Settlement areas are mapped as polygons and rockfall sites are mapped as lines. This allows for better spatial delineation of the geohazard, however, it means that each geohazard has a different data type. To make display easier, and to allow for grouping of sites we structured the database with a "superhazard" record. This superhazard record captures the spatial location of the particular geohazard (or groups of geohazards), the geohazard type and the A, B or C rating of the site. Each superhazard record can contain only 1 type of geohazard.

This can be particularly useful where there is a long rock cut that presents multiple problems and where the engineer or geologists feels that the site warrants a different hazard rating for different portions of the cut. However, as a practical matter, it is rare to mitigate only one part of a single rock cut, so there is good reason to maintain this "split" site as 1 record on an overview map. Similarly, multiple small sinkholes might be mapped under one "superhazard" to make an overview map less crowded.

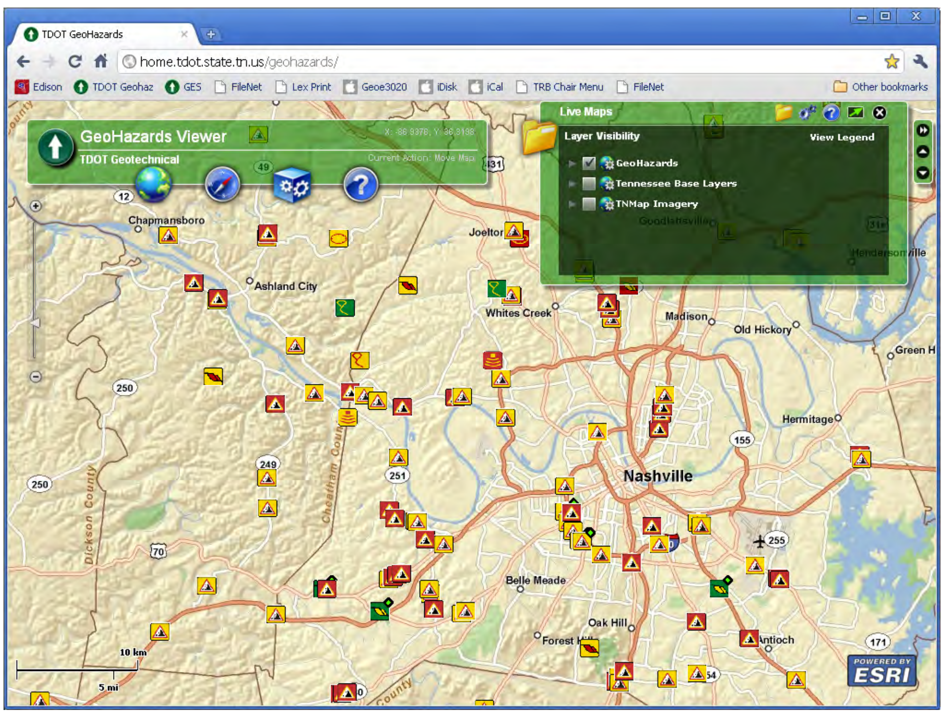


Figure 5 - Geohazards in the Nashville Area. Note the Red/Yellow/Green colors used for geohazards. Each type has its own "highway sign" as shown in Figure 1.

Digital Field Mapping
Digital mapping was implemented with this geohazards system using ArcPad. Special data gathering forms that make data entry simple on a mobile device were designed and implemented for each geohazard type.

Map data including existing mapped geohazards, aerial photography, topographic maps and other spatial data layers can be included as background layers on the mobile mapping units to make digital mapping in the field easier.

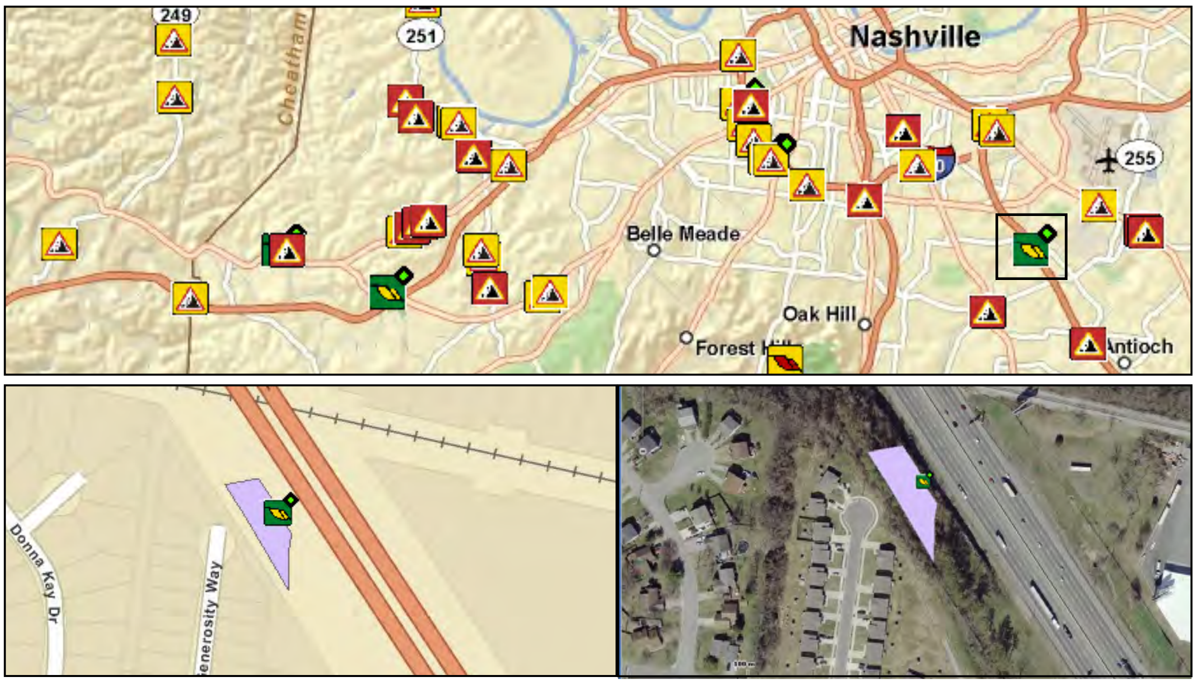


Figure 6 - Landslide along I-24 in Nashville that has been repaired. Area of original slide shown in purple; green Landslide sign indicates site rating of C - low hazard. Example background layers available in ArcGIS, in ArcPad or on the geohazard web application are shown.

Data is "checked out" from ArcGIS which makes a special mobile map (implemented in ArcPad) of the areas of interest for mapping and allows the geologist in the field to map points, lines or polygons. The geologist or engineer can map areas and features of the geohazard which will then be stored in the database and available for display within a GIS program or with the Geohazards Web application also developed for the system. This ArcPad map can be viewed on any Windows or Windows CE device that has ArcPad installed. Thus mapping can be done from Windows CE based GPS unit, with a tablet computer or with a desktop computer where hand notes on paper maps are transferred manually into the computer.

Data entry forms were designed for ArcPad to make data gathering as simple and error free as possible. Drop down boxes are used for selections and the spatial data at the site is automatically entered, with a user over-ride possible if the data requires any modification. The forms enter the county, route ID, log mile and other spatial data based on the GPS coordinates of the site.

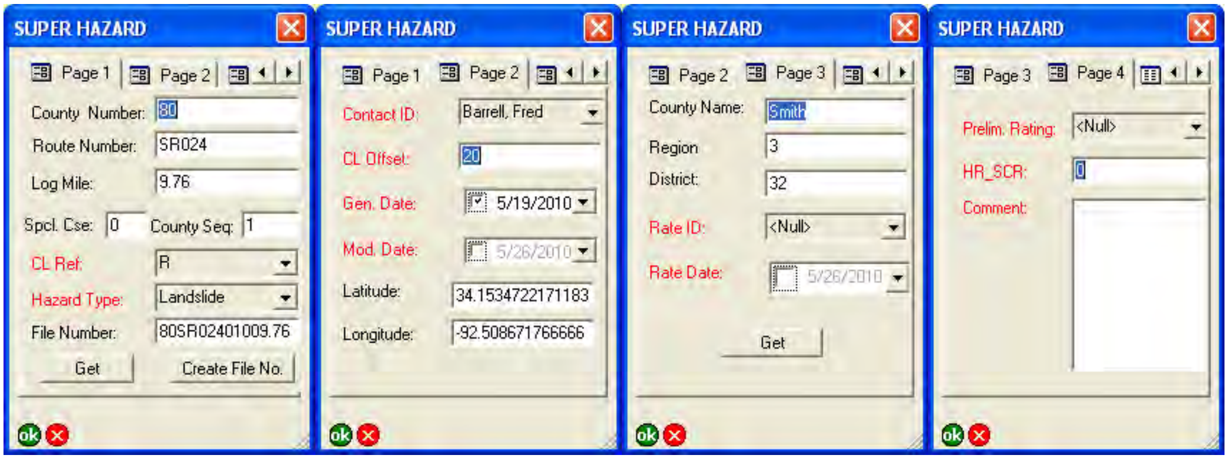


Figure 6 - ArcPad Form Example: “Superhazard” record for a landslide. Data shown in red require data entry from the user. Other data such as route number, file number, county name etc. is entered automatically by the system based on the GPS reading at the site, but can be changed if data is incorrect or needs to be revised.

Mitigated Sites

Another development of the geohazards mitigation system is to easily track sites where mitigation has been performed. Landslide repairs, rockfall fences, rock bolts, sinkhole mitigations etc. can all be thought of as assets and sites that have been repaired need to be tracked and revisited over time. This is tracked through the record in the database, but is displayed on the web application and in ArcGIS as a “badge” added to the site. A yellow badge indicates a partial mitigation of the site and a green badge indicates a “complete” mitigation of the site.

B site, partial mitigation B site, complete mitigation C site, complete mitigation

Figure 7 - Rockfall Site icons with badges to show mitigation projects.

Sites which have been mitigated to the point that they no longer pose a hazard to the traveling public are classified as C sites. A landslide with a rock buttress replacement, for instance would be a C site with a green badge. A complete mitigation at the site is a project that provides a substantial upgrade in safety at the site. It may not remove all hazard. For example, a rockfall fence may provide significant safety upgrades to a site, but may not stop all potential rocks from striking the road. A partial mitigation would be a project that provides safety improvements, but is expected to be a more temporary measure. For instance, a scaling project or cleaning out of catchment ditches. Figure 8 below shows a some rockfall mitigation sites from a recent project along I-40, Figures 5 and 6 also show mitigated sites.

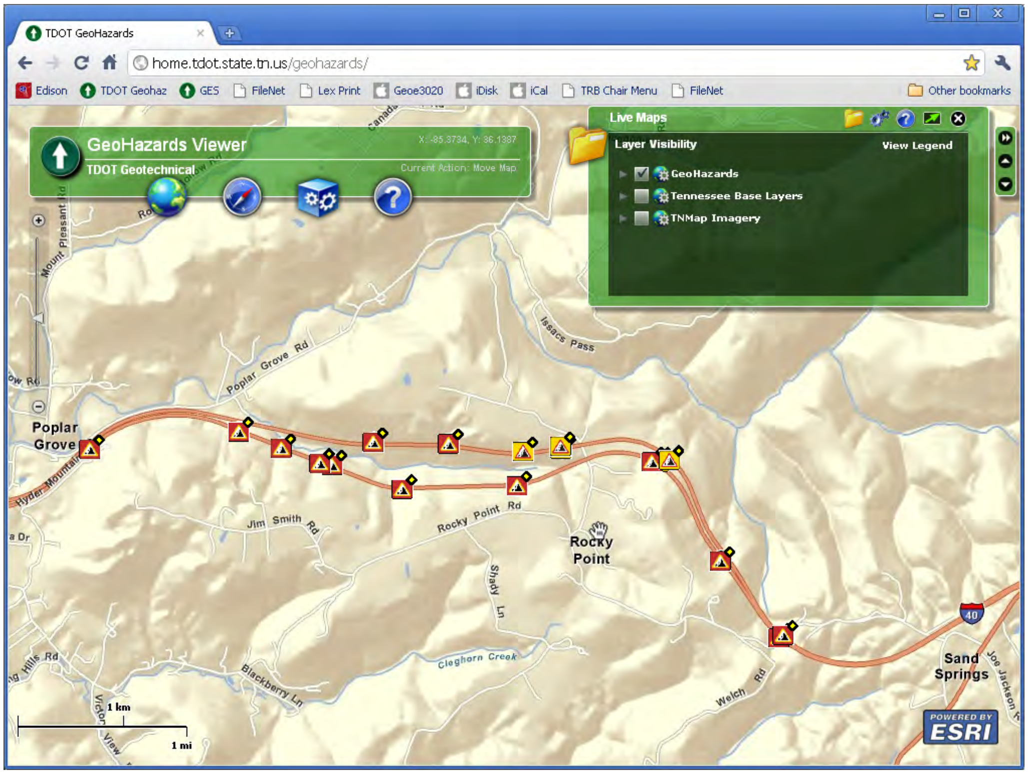


Figure 8. View of Rockfall Sites where mitigation has been performed. The project ended on August 2010. Note that many of these sites are still tagged as "A" and have a partial mitigation badge. All rockfall projects must go through one winter before the rating can be changed, they are re-rated after 1 year. Thus you can see that work has been performed, but the original rating has not yet been changed. Most of these sites will change to "B" sites with partial or complete mitigation badges based on this evaluation scheduled for August 2011.

Rockfall mitigation projects are re-rated after the site has experienced at least one winters weathering period. Typically, this is done one year after completion of the mitigation project. Winter has historically been the time of greatest frequency of rockfall in Tennessee and sites need to experience freeze thaw and significant rains before we can judge the success of the work. Sites which still pose a moderate threat after this re-assessment would still be marked as B sites and colored yellow even if the badge indicates green informing the user that a significant safety improvement had occurred at the site.

Asset Management, Mitigated Sites and Rockfall Repair

For all projects, if unlimited money were available, our ultimate goal would to be “change the color” of all sites to a green sign with a green badge, thus indicating that we feel that no more work is required and the site poses little hazard to the public. However, as budgets are more limited, sites are chosen to get the greatest safety improvements within a given budget year. This more policy based asset management program has been implemented with rockfall as there is a set annual budget for repair. Another strategic goal of the program in it’s first five years was to “front load” the projects with those that could be completed with a minimum of environmental and right of way lead time. This was done to get projects started quickly and to give the department time to learn the most efficient contracting methods for projects. It allowed TDOT to start with projects that could be taken to contract quickly, preferably within one year of initiation. Emergency projects are still let and treated as needed. But the decision was made to approach rockfall in a more systematic manner wherever possible. The goal is to eventually reduce the need for emergency rockfall projects. B sites are repaired along with A sites in a single project where they are closely located and can be reasonably included in a contract. For example, all the sites in Figure 8 were repaired in a single contract that mitigated 18 sites.

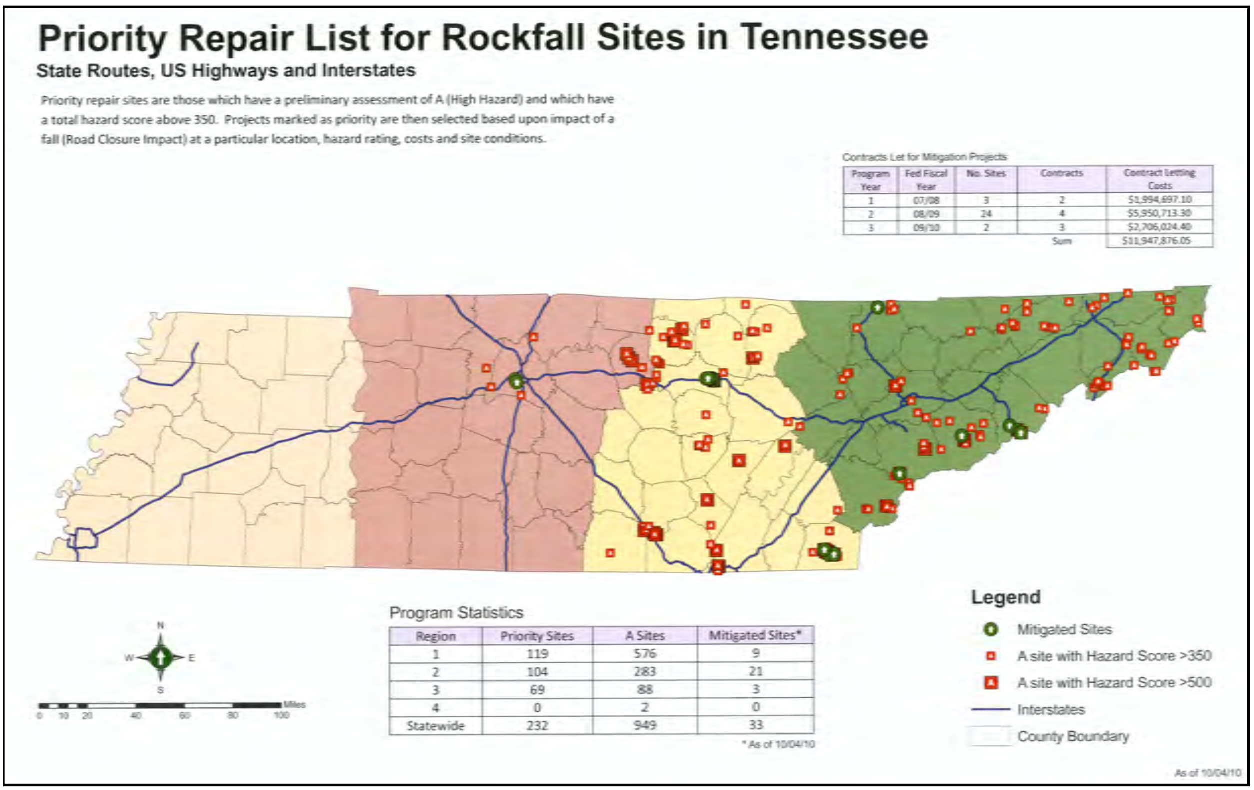


Figure 9. Priority Rockfall Sites in Tennessee - potential repair sites are shown along with sites that have been mitigated as of October 2010. Note that the original inventory identified over 1980 rockfall sites statewide and originally had 982 identified as A sites. Map produced using ArcGIS and TDOT’s geohazard data layers.

While the remainder of the geohazards that are mapped and included in the system do not have a set annual budget, as with rockfall, these sites are repaired through the letting of maintenance projects. As sites are identified during exploration, or are noted by TDOT maintenance or construction staff and brought to the attention of the GES, these sites are included in the system. This does not allow TDOT to date to take as comprehensive an approach to mitigation as has been done with rockfall. However, since implementation of the system, all sites are included and over time this will allow the department to take a more systematic and rational view of all of the geohazards managed by TDOT and to plan mitigation projects as efficiently as possible.

The main challenge for this implementation is simply field work time to gather data and include it in the system. Sinkholes are typically repaired as they are noted within either construction projects or through maintenance funds, often with state forces. However, landslides and settlement areas are more problematic as they are usually considerably more expensive and much harder to absorb through general maintenance funds. The next logical step for the system, if money is available, is to initiate the field work required to gather the data set for landslides. This would allow the department to deal with these very expensive sites in much the same way as is being done with rockfall and allow for more rational planning, rather than only emergency planning of these repairs. The RCI rating gives a numerical comparison between sites based on their impact to the roadway network. The goal would be to initiate landslide mitigation projects before they must be let on an emergency basis - almost always the most expensive method of contracting.

Web Application

As the department learned with rockfall, a simplified interface to the data with some set queries can be invaluable. GIS programs can allow a geologist or engineer to produce excellent spatial analyses or maps, but they are not always the most user friendly. A typical computer user will need specialized training to make full use of these programs as they have their own logic that may not be consistent with how the users has learned that computer programs generally work. For instance a user familiar with CAD programs or with some typical graphics programs that have been used in the past will find that there are many new skills and techniques that need to be learned to make effective use of a full-scale GIS program. It is for this reason that the original rockfall project within TDOT included a web application and this web application was actually implemented in two different versions.

Experience in implementing this smaller data set as a web application directly informed the software choices and decisions made for the Geohazard project. We chose to use ArcMap as it was compatible with ArcGIS which we were using also because of the mobile mapping application ArcPad. One GIS software family was used to make the underlying programing and implementation easier. This was also done to make upgrades to the system in the future much simpler. There are other applications that could likely do the job very well, including some non-proprietary software that can be programmed to produce a web application and indeed this was done at TDOT with an earlier version of the rockfall data. The ArcGIS family of programs was already supported at TDOT and our GIS staff already had excellent familiarity with the software, thus reducing the needed development time. The underlying database however, does not restrict future versions from migration to another software platform.

The web application shows all of the geohazards in the highway sign motif and uses check boxes and layers to restrict what data is shown based on user chosen criteria. For example, only landslides could be shown. The user can zoom in to fine detail and use either the initial roadway base map or choose base maps such as topographic maps, aerial photography or the "pyrite layer" which shows potentially acid producing rock formations. Figures 5, 6 and 8 show views of the data from within the web application and demonstrate some of the base layers that are available. Figure 10 shows some mitigated rockfall sites within the system on a topographic map background.

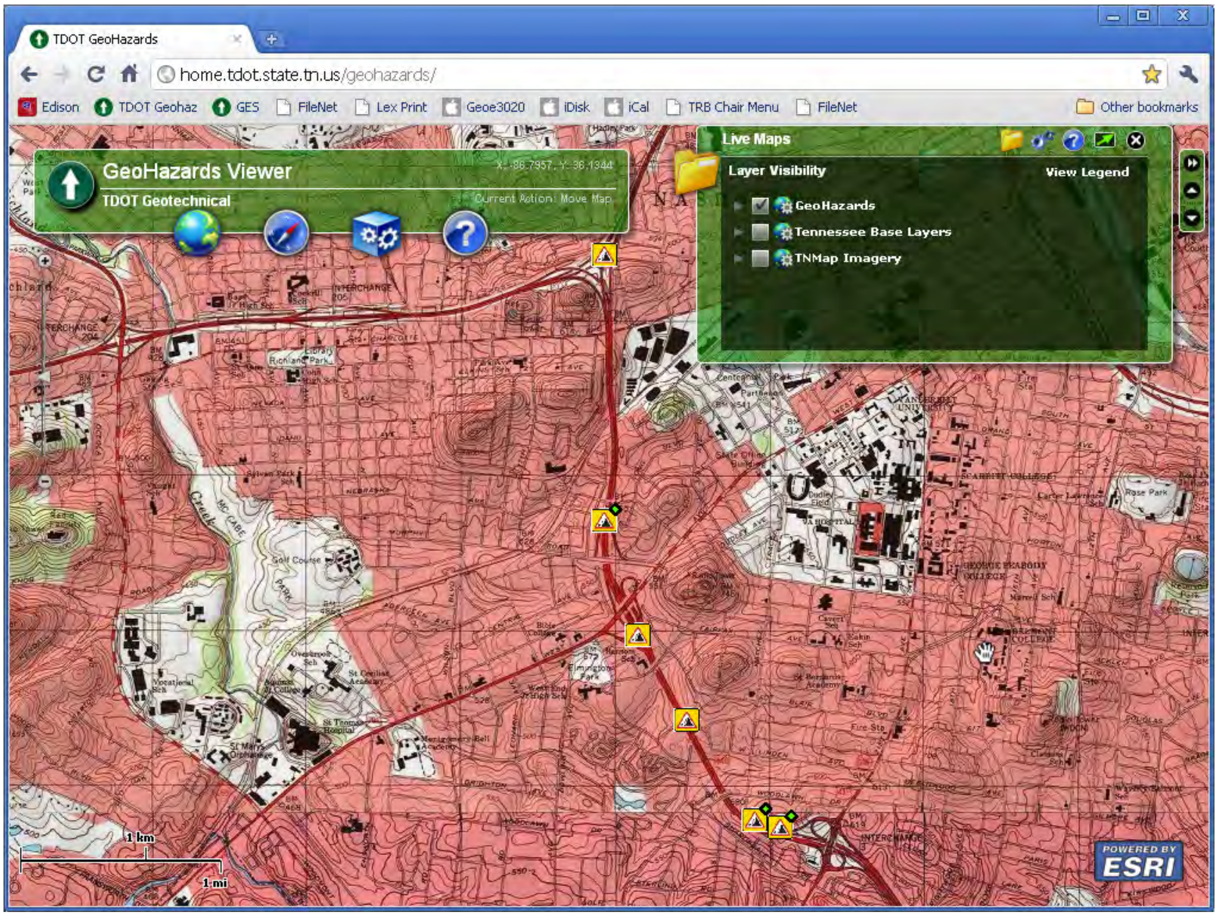


Figure 10. Geohazards on a topo map base layer.

Querying the Data in the Web Application

The web application allows the user to look at sites in a map view, but also contains connections to the details of each record. The user can use an information button to hover over a site which will then allow access to the complete record in the database for that site. Rockfall hazard ratings, RCI ratings and geohazard assessments for all other project types can be viewed within the system. Figure 11 shows one of the information tables that can be viewed within the system.

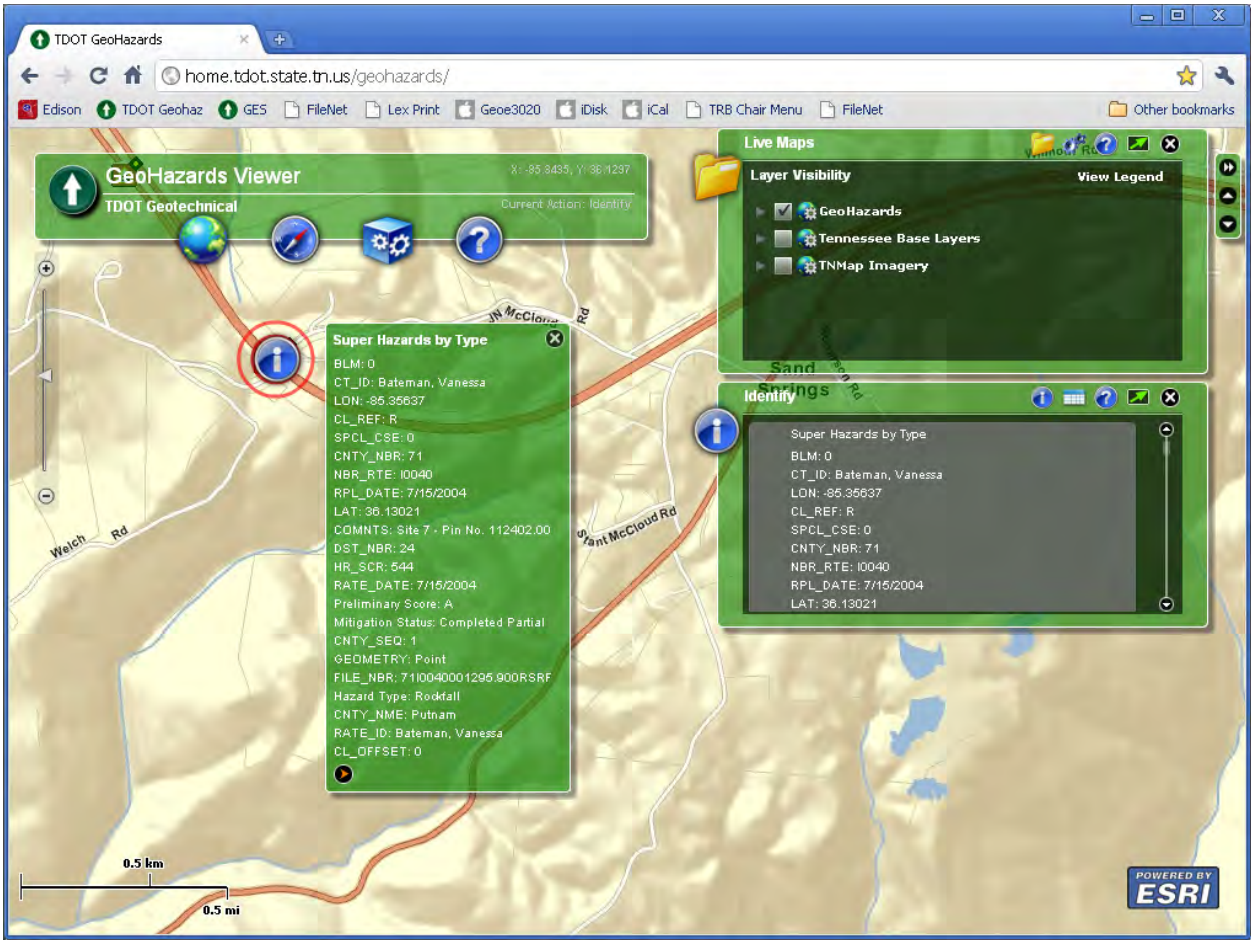


Figure 11 - Information Query for a specific site using the “Identify” feature. The record shown gives the user immediate access to the underlying data should it be needed. This record is a rockfall site that has a preliminary rating of A, has a rockfall hazard score of 544 and has had a partial mitigation completed at the site.

Historical Document Access

Selection of a button also allows the user to bring up a document list that details geotechnical reports, drawings, photographs and plans that have been completed for a site. This may be of some importance during an emergency situation as it allows the user to immediately assess what work may have been done in the past. For previously slow moving landslides a geotechnical investigation with drilling and design work may have been completed before a larger drop at the site occurred. Immediate access to this information saves time and prevents duplication of effort. Figure 12 shows a view of this list.

At this time a true “document management” system has not been implemented, though all reports, photographs and drawings are stored on TDOT’s geotechnical server and are indexed. If at some point in the future the GES implements a document management system as is being done within other areas of the department, instant access to this data is available through the use of a hyperlink in the document list.

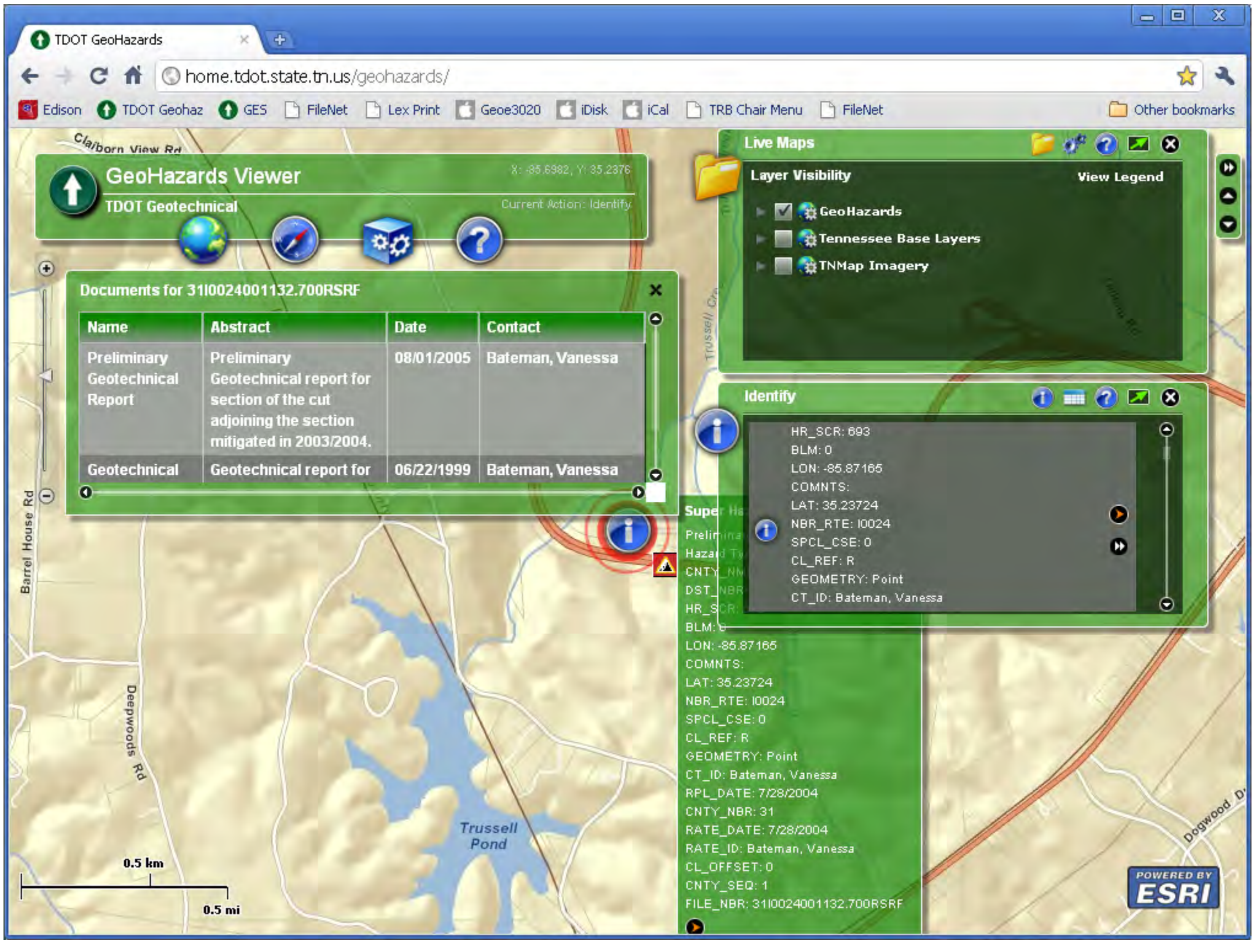


Figure 12. View of Document List for Geohazard Site.

Ease of Use

An extensive help file is also available to the user in the web application. With the advent of Google Maps, Bing, Mapquest and other online mapping programs, users are familiar with the basic navigation methods needed and can learn the online access to the system very quickly. One of the more surprising aspects of the system has turned out to be that the TDOT geologists and engineers learned quickly that statewide topo maps were available within the system and could be printed for any particular location, regardless of any geohazard that might be at the site. The system is being used as a primary point for topographic data for all types of geotechnical projects and investigations.

CONCLUSION

While TDOT does not have a true overall "Asset Managment" program that covers all geohazards, work in recent years had moved the department closer to the goal. An extensive inventory of rockfall has been completed, a mitigation program now in it's 4th year is demonstrating the usefulness of a statewide approach and the system has now been expanded to include other major geohazards. All landslides, sinkholes, rockfall, settlement areas and identified springs/seeps can now be mapped and stored in one central location. The use of an underlying spatial database to store all this data means that the data can be used with any spatial analysis software, preventing one GIS software vendor from having TDOT data in a proprietary format that cannot easily be viewed using other software at a later date. All the data can be viewed, analyzed and maps can be generated using a full scale GIS program. Input into the system can be done manually, where needed or preferably using the digital mapping tools developed with this initiative. Easy access to the system is maintained by the use of a web mapping application, this gives users a simplified interface, but with deep access to all the data gathered at the sites. While a statewide inventory of all of these geohazards has not yet been completed, this gives the department a central location for all data and will allow it to build this view over time. Further work on the system includes additional field work as well as adding in "pyrite repositories," i.e. encapsulation sites built by the department to mitigate potentially acid producing rock. Base maps can be added to the system as they become available. One obvious need is a 1:24 000 scale statewide geology map. A 1:250 000 scale map is included as a basemap, but as there is no comprehensive GIS data layer at the quad map level, it was not included in the system.

With limited budgets and uncertain funding future at the federal level for building an maintaing transportation assets it is vital that we make as an efficient use of the available dollars as possible. Emergency response to projects are necessary, but if we can manage our overall risks by targeted mitigation programs over time, it will allow for better use of available funds. Asset management principles are not just for pavements. These principles can be applied to hazard reduction and sites can be chosen based on these hazards, impacts and cost/benefit analyses in a rational manner, rather than only on an emergency basis. However, to get there, we must have an inventory of hazards that need to be managed, easy access to our data that allows for multiple types of analyses and in the end a budget so we can strategically plan out how to achieve the best cost/benefit on a statewide rather than a site by site basis.

REFERENCES

Bateman, V.C. Rockfall Closure Impact and Economic Assessments for Rockfall Sites in Tennessee. *Proceedings of the 55th Highway Geology Symposium,* Kansas City, Missouri, 2007.

FHWA, *Asset Managment Overview.* FHWA-IF-08-008. Office of Asset Management, Federal Highway Administration, US Department of Transportation, Washtington, D.C., December 2007, 43 pages.
Accessed April 30, 2011, http://www.fhwa.dot.gov/infrastructure/asstmgmt/assetman.cfm

Gusek, J., Moore, H.L., Bateman, V.C., Ozment, J., Oliver, M.L., Kathman, D., Waples, J., Rutkowski, T., Bowden, W. and Reither, A. A New Guidance Document for Mitigating Impacts from Acid Producing Rock Formations in Tennessee Road Construction Projects. *Proceedings of the 59th Highway Geology Symposium*, Sante Fe, New Mexico, 2008.

Mauldon, M.; Drumm, E.; Dunn,W. M; Bateman, V.B.; Rose, B. and Kim, M. *Rockfall Management System for Tennessee: Final Report*. Research Report RES1189. Tennessee Department Of Transportation, Long Range Planning Division Research Reports, Nashville, TN, December 2007, 301 pages.
Accessed April 10, 2011, http://www.tdot.state.tn.us/longrange/reports/res1189.pdf

Moore, H.L., “Treatment of karst along Tennessee Highways.” American Society of Civil Engineers, Geotechnical Special Publication 14, 1988, pgs. 133-148.

Moore, H.L., “The Use of Geomembranes for Mitigation of Pyritic Rock,” *Proceedings of the 43rd Highway Geology Symposium,* Fayetteville, Arkansas, August 1992.

Royster, D.L., “Highway Landslide Problems Along the Cumberland Plataeu in Tennessee,” AEG Bulletin, Vol. X, No. 4, 1974.

USGS, *Tennessee Seismic Hazard Map.* Accessed April 10, 2011, http://earthquake.usgs.gov/earthquakes/states/tennessee/hazards.php

Building an Enterprise Geotechnical Database to Support Geologic Mapping Activities

G. A. Weisenfluh
Kentucky Geological Survey
228 Mining and Mineral Resources Bldg.
University of Kentucky
Lexington, KY 40506-0107
Telephone: (859) 323-0505
Fax: (859) 257-1147
e-mail: jerryw@uky.edu

W.A. Broyles
Division of Structural Design (retired)
Kentucky Transportation Cabinet
Frankfort, KY 40601

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

This paper reprinted with minor modifications with permission by Digital Mapping Techniques 2009.

Weisenfluh, G.A., and Broyles, Bill, in press, Building an Enterprise Geotechnical Database to Support Geologic Mapping Activities, in Soller, D.R. (ed.), Digital Mapping Techniques 2009 -- Workshop Proceedings: U.S. Geological Survey, Open-File Report.

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior

ABSTRACT

A three year program was conducted to develop an information management system for geotechnical projects and associated data managed by the Geotechnical Branch of the Kentucky Transportation Cabinet. The first objective was to catalog all historical project reports and make them available as electronic documents on the Web. The resulting website contains reports for over 6,000 projects and can be searched by a number of geographic or geotechnical criteria of interest.

The second objective was to develop a data entry system for tracking the progress of ongoing projects. This system gathers information from the KYTC 6-year roadway plan database to initiate new projects and allows for characterization of the type of work being conducted and the organizations and personnel responsible. Dates of completion for a variety of common project tasks can be entered and reports can be generated for managers showing the status of project work. Once projects are completed the final reports are uploaded to the system and become available on the Web.

The third objective was to create a database of drilling and testing information resulting from these projects. A data entry application was developed in the gINT software for managing the data. Project and hole header data is imported from existing electronic files, and additional efficiencies were created by using pull down menu’s and built in calculations. Once individual projects are completely entered into a gINT database, they are transferred to a SQL-Server Enterprise database for Web distribution using Dataforensics PLog Enterprise software.

INTRODUCTION

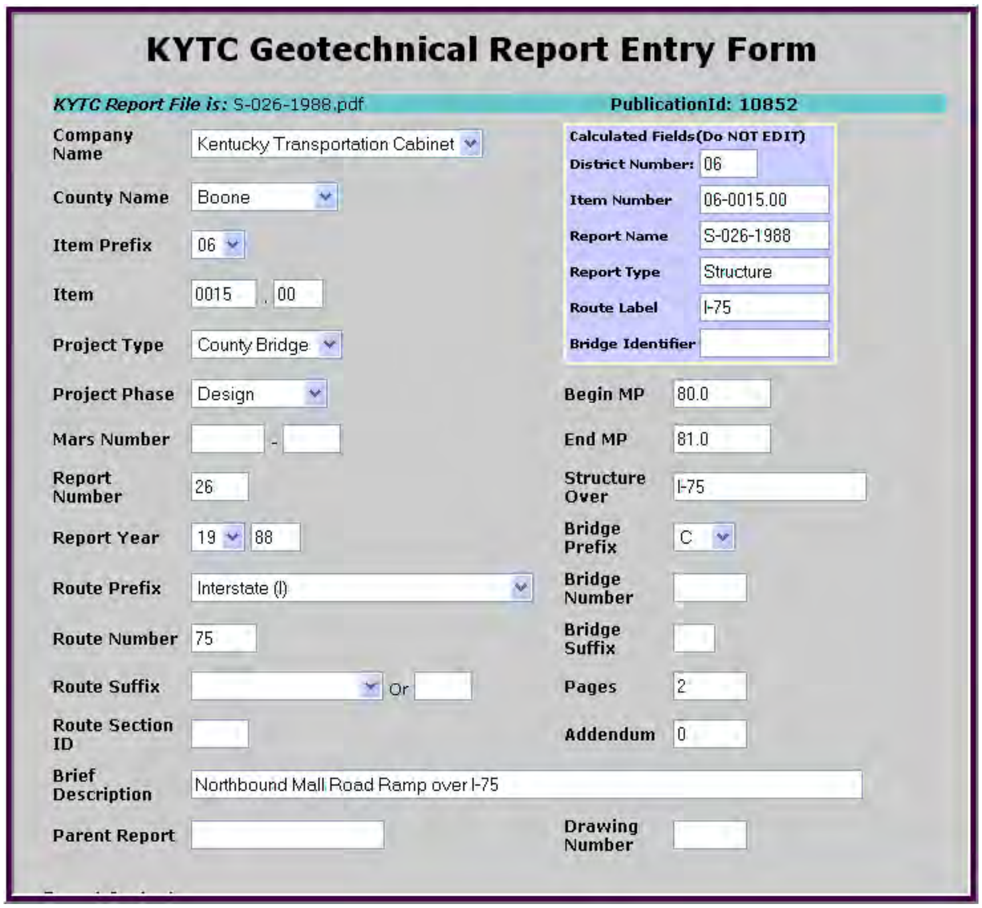
The Geotechnical Branch of the Kentucky Transportation Cabinet (KYTC) employs engineers and geologists to prepare design plans for roadway cuts, fills, and associated structures. Almost all projects involve site investigations that require drilling, sampling, and testing of geologic materials, including rock and soil. These data are reviewed by project staff, and standard engineering practices are used to create design drawings and recommendations for construction work. Drawings are prepared in CAD software and then combined with text documents to produce the final issued reports. Historically, the reports were produced as paper documents, and approximately 6,000 were filed in the Branch library. Supporting data, such as drillhole logs and lab analysis results, were loosely and inconsistently filed by county of location.

Public access to the reports, mainly by engineering consulting firms, was through verbal or written requests; however, there was no catalog of projects from which to identify reports of interest. Branch staff would respond to requests by making copies of reports or by scanning them into electronic files. Supporting data were generally not released because they were not easily accessible. Reductions in staffing levels at KYTC made it increasingly difficult to provide this information, and the system was vulnerable to losing important historical records.

In 2006, KYTC approached the Kentucky Geological Survey (KGS) for assistance with transitioning their records system from paper to electronic. KGS had extensive experience with similar document conversion processes, and had a Web system for disseminating geologic information. KGS geologists would benefit from easier access to the KYTC data for geologic mapping projects, since the KYTC project reports contain information about depth to bedrock, soil properties and classifications, rock descriptions and physical properties, water table data, and fracture measurements. KYTC would benefit from a reduced effort for distributing their data, and could save significant financial resources by reusing drillhole data and analyses from historical projects. A 3-year program was initiated; the objectives were to make all historical reports available to the public through a Web service, create a Web-based management system for tracking the activities and characteristics of future projects, develop a software system for entering borehole and lab analyses into a relational database, and serve these supporting data to the public on the Web.

ARCHIVING LEGACY GEOTECHNICAL REPORTS FOR WEB DISTRIBUTION

The first phase of the project dealt with scanning, cataloging, and distributing the historical reports. KGS developed a database and corresponding data entry form based on interviews with KYTC staff familiar with the content of the reports. The goal was to attribute those characteristics that would be likely search criteria. Figure 1 shows the upper part of the form, which includes information about who conducted the geotechnical study, the location of the study in reference to the transportation network, what type of construction activity it was related to, when it was done, and an assortment of identifiers that relate to the Cabinet’s central tracking database of road plans. To minimize typing errors and inconsistent entries, some data were harvested from the Cabinet’s 6-year roadway plan database, pull-down menus with standard lists were implemented, validation rules were applied where appropriate, and formatting of certain concatenated text values was done programmatically. Administrative functions were designed into the form so that users could update the interface (for example, to add new values to pull-down menus) without the assistance of a programmer. Figure 2 shows the lower part of the form, where contents of the report are described; this information was specified by the KYTC engineers and geologists to facilitate finding reports that deal with specific topics or engineering practices. Check-box controls were used to speed the data-entry process. A free text field was included to add items not covered on the lists, and periodically these nonstandard items were reviewed, and some were eventually included as standard check boxes.


KYTC Geotechnical Report Entry Form

KYTC Report File is: S-026-1988.pdf
PublicationId: 10852

Company Name: Kentucky Transportation Cabinet
County Name: Boone
Item Prefix: 06
Item: 0015 . 00
Project Type: County Bridge
Project Phase: Design
Mars Number: -
Report Number: 26
Report Year: 19 88
Route Prefix: Interstate (I)
Route Number: 75
Route Suffix: Or
Route Section ID:
Brief Description: Northbound Mall Road Ramp over I-75
Parent Report:

Calculated Fields(Do NOT EDIT)
District Number: 06
Item Number: 06-0015.00
Report Name: S-026-1988
Report Type: Structure
Route Label: I-75
Bridge Identifier:

Begin MP: 80.0
End MP: 81.0
Structure Over: I-75
Bridge Prefix: C
Bridge Number:
Bridge Suffix:
Pages: 2
Addendum: 0
Drawing Number:

Figure 1. Upper part of project data-entry form, showing the variety of descriptive information stored for each report.

Report Contents

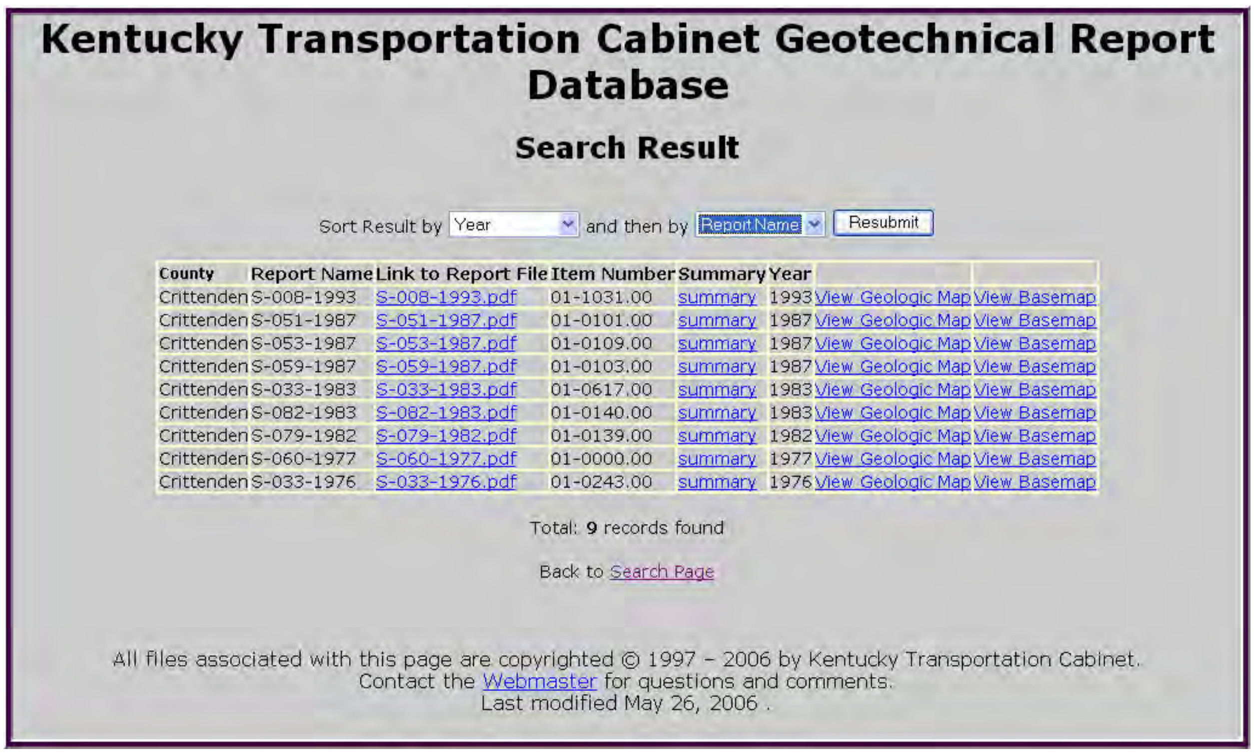
Cut Slope Designs		Friction Piles	☐
Rock Fall Fence	☐	End Bearing Piles	☐
Wire Mesh	☐	Black shale remediation	☐
Back Stowing	☐	Mining	☐
Shape Ditches	☐	Geophysics	☐
Soil Modification		Instrumentation	☐
Dynamic Compaction	☐	Seismic design	☐
Wick Drains	☐	Litigations	☐
Surcharging	☐	Lightweight fill applications	☐
Special Structures		Shotcrete	☐
Gabian Baskets	☐	Excess Materials Sites	☐
RSS Slopes	☐	Chemical Stabilization	☐
Tunnels	☐	Sinkholes	☐
Tied Back Walls	☐	**Sheet Types**	
Soil Nail Walls	☐	Project Layout	☐
Cantilever Wall	☐	Location Map	☐
Cantilever H-Pile Wall	☐	Subsurface Data Sheet	☐
Cantilever Railroad Steel Wall	☐	Soil Profile	☐
MSE Wall	☐	Geotechnical Notes	☐
Drilled Shafts	☐	Cut Stability	☐
Settlement Platform	☐	Embankment Stability	☐
Rock Bolts	☐	Loading Diagrams	☐
		Coordinate Data Sheet	☐

Other Content Items: (Separate by ;)

Figure 2. Lower part of project data-entry form, showing check-box contents that can be associated with a report.

Two other key elements were included at the suggestion of KGS. The first was the ability to assign one or more geologic units to a report. A search form was designed to look up standard stratigraphic names from the KGS geologic mapping database and translate them to alpha-numeric codes. The second allows for locating KYTC projects on a geographic base. Although many projects had geographic descriptions, such as route numbers and milepoint designations, these are not easily translated to geographic coordinates and moreover can change when roads are realigned. KGS developed an internet map service that included a function to zoom to route milepoints or road intersections so that Branch staff could verify the project location on a topographic or photographic map base. Once the location was determined, a tool was used to define a bounding rectangle for the project, and the coordinates for that rectangle were added to a database table and its primary key was assigned to the report. This facilitated both drawing the project extents on a map as well as searching for reports using geographic criteria. A second way of locating projects was implemented for recent work in which lists of surveyed hole locations were available. These lists could be uploaded to the map service, posted for verification, and then used to define the project extent.

At the end of each data-entry session, the electronic report is selected on the user's computer; the report is uploaded to the KGS Web server for public dissemination and the attribute data are submitted to the database. Public access to these data is provided by an active server pages (ASP) Web service hosted by KGS (http://kgs.uky.edu/kgsweb/KYTC/search.asp). Users can search for reports by any combination of attributes that are included in the database. The result of the search (Figure 3) is a simple list of projects identified by county of origin, date of completion, project number, and KYTC item number that relates to the State's 6-year roadway plan. A number of links (formatted in blue) are also provided. One is to a summary page for the project giving more detailed information about the report and another is to the online version of the report. Two other links lead to Internet maps zoomed to the extent of the project—one a topographic map with other transportation information provided and the second showing the geologic context of the project with access to a variety of KGS geologic site data and descriptions (Figure 4).



Kentucky Transportation Cabinet Geotechnical Report Database

Search Result

Sort Result by Year and then by Report Name Resubmit

County	Report Name	Link to Report File	Item Number	Summary	Year		
Crittenden	S-008-1993	S-008-1993.pdf	01-1031.00	summary	1993	View Geologic Map	View Basemap
Crittenden	S-051-1987	S-051-1987.pdf	01-0101.00	summary	1987	View Geologic Map	View Basemap
Crittenden	S-053-1987	S-053-1987.pdf	01-0109.00	summary	1987	View Geologic Map	View Basemap
Crittenden	S-059-1987	S-059-1987.pdf	01-0103.00	summary	1987	View Geologic Map	View Basemap
Crittenden	S-033-1983	S-033-1983.pdf	01-0617.00	summary	1983	View Geologic Map	View Basemap
Crittenden	S-082-1983	S-082-1983.pdf	01-0140.00	summary	1983	View Geologic Map	View Basemap
Crittenden	S-079-1982	S-079-1982.pdf	01-0139.00	summary	1982	View Geologic Map	View Basemap
Crittenden	S-060-1977	S-060-1977.pdf	01-0000.00	summary	1977	View Geologic Map	View Basemap
Crittenden	S-033-1976	S-033-1976.pdf	01-0243.00	summary	1976	View Geologic Map	View Basemap

Total: **9** records found

Back to Search Page

All files associated with this page are copyrighted © 1997 - 2006 by Kentucky Transportation Cabinet.
Contact the Webmaster for questions and comments.
Last modified May 26, 2006 .

Figure 3. Example project search result page with links to the online report and to Internet maps that show the geographic and geologic context of the projects.

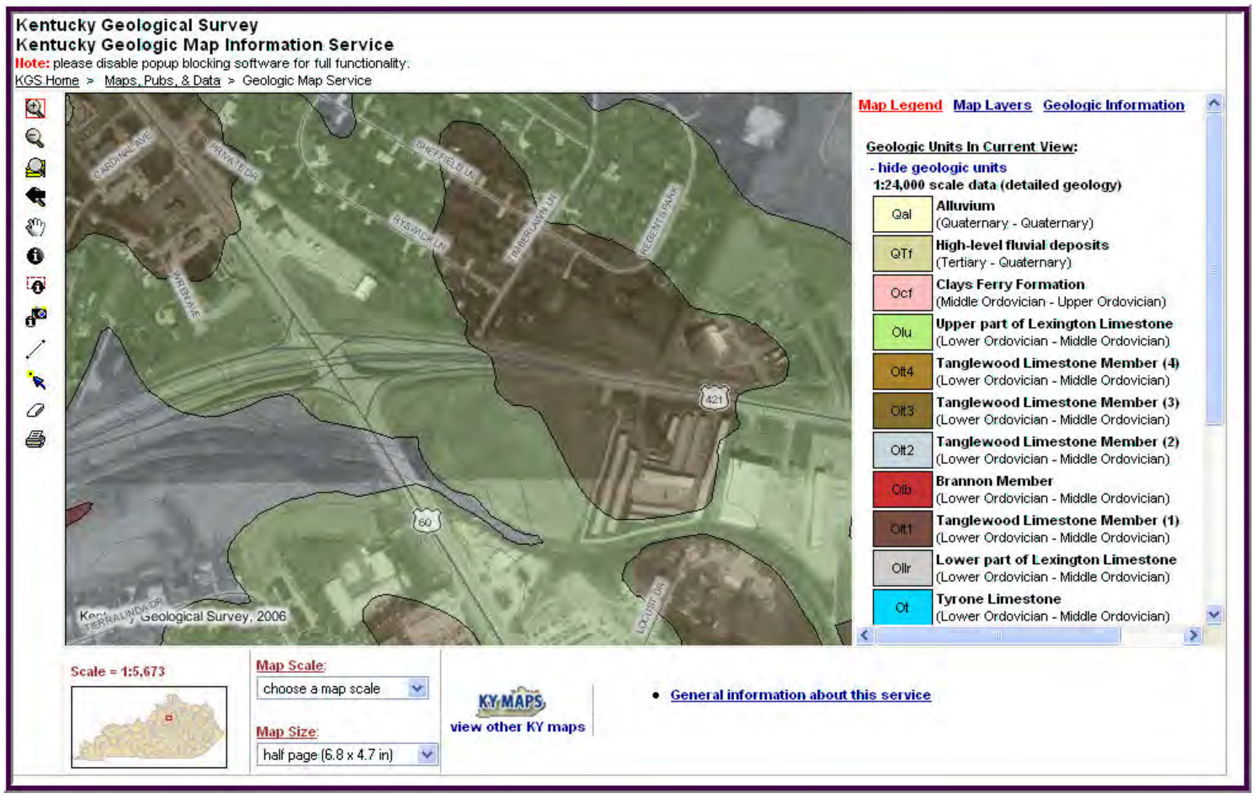


Figure 4. Geologic map zoomed to the extent of a linked project, showing formation contacts along a major route intersection.

The development of this phase of the project and the scanning of documents began in late 2006, and all the historical reports were cataloged and publicly available by fall of 2007.

MANAGING GEOTECHNICAL PROJECT ACTIVITIES

Once phase I was accomplished, the next phase of the project addressed management of active projects. In many ways this was similar to cataloging historical reports, except for the timing of data entry. Rather than beginning with a finished report, new project data are entered over time as the project work proceeds. Once the project is complete, the final report is issued and uploaded to the system and the project status is changed from active to complete, making the information available to the public. The cataloging program was modified to reflect this workflow, and an additional set of functions was added to track project activities. These included a form to record the dates of completion for project milestones such as completion of drilling or issuance of various documents. This is an extremely useful function for the Branch manager, because from 100 to 200 projects may be active at a given time. Standard reports were designed for various aspects of projects, such as those related to a geologist's or driller's activities, or specific types of projects such as landslide mitigations or structural designs. The manager can now create a report to quickly assess the status of activities to identify impediments to project completion.

Another function was added that analyzes the catalog to find relationships between projects. For example, planning projects usually precede roadway designs, and a number of bridge and culvert structure designs may relate to a roadway project. This function builds the complete hierarchical relationship tree beginning with any specified project and looking upward and downward in the tree.

BUILDING AN ENTERPRISE DRILLHOLE DATABASE

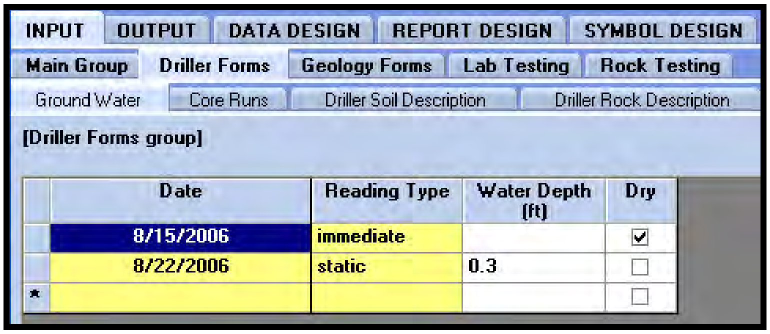
Almost every KYTC geotechnical study involves drilling holes—rock soundings, soil sample holes, and rock coreholes—to obtain samples for material testing and to visually assess those materials. Structure projects typically require from 2 to 12 holes, whereas roadway projects may have as many as 300 holes. Over 2,300 holes were drilled in 2007, and more than half of those had samples that were laboratory tested. Historically, such drillhole data were used solely for the project on which they were obtained, but given the substantial costs of acquiring the data, KYTC was interested in developing a database that would enable them to reuse data for future projects conducted at nearby locations. A historical database of drilling and testing data would also facilitate characterization of rock and soil intervals over larger areas than are typically studied for site assessments—information that could greatly enhance geologic and soils mapping of Kentucky. Therefore, the third phase of this project was intended to develop an enterprise software system for storing drillhole data and a Web interface for exploring the data to find holes that could be useful for new design projects or for regional analysis.

Geotechnical projects are conducted by teams that include engineers, geologists, lab technicians, drillers, and CAD operators. Standards for data collection, analysis, and reporting have been developed over the years at KYTC to maintain consistency in results and report formats. The design requirements for the data entry system included forms that modeled the workflow of team members, and extensive controls to adhere to standard vocabulary, methods, and calculations. Another objective was to minimize data entry by providing functions to import data from external databases, using pull-down menus, and by calculating numerical values programmatically wherever possible. The gINT software suite (www.gintsoftware.com[1]) was selected to develop the data-entry application, because of its capabilities for customization and powerful report development modules. gINT uses Microsoft Access for its data storage format and Visual Basic for Applications for program customization. It includes modules for building data-entry applications, for designing text or graphical output, and for import and export of data. At the time of development for this project, gINT software did not have the capability to build and maintain an enterprise database. It was designed for the consulting industry, which typically performs separate, discrete projects; therefore, each project's data are stored in a separate database. Consequently, PLog Enterprise (www.dataforensics.net) was acquired to convert the gINT projects to a SQLServer database for permanent archival and to facilitate the most efficient mechanism of Web distribution.

Project description information is already stored in the project management system described above; therefore, it can be imported directly into gINT to initiate a new database. Similarly, KYTC surveyors are required to prepare a list of hole locations in a standard format so that these data can be imported as well. Once these sets of data are imported, the project manager

[1] Since this project was completed gINT was acquired by Bentley Systems (www.bentley.com)

adds other details about the characteristics of each hole to prepare it for further data entry by team members. Drillers enter information about groundwater levels, core recovery, rock intervals, and soil sample intervals. An example form (Figure 5) shows entry of both immediate and static groundwater readings for a hole. Geologists enter the lithologic data for coreholes and a variety of rock assessments that depend on the type of project. Figure 6 shows depth to rock and RDZ, a parameter related to weathering of rock material, along with geologic unit designations. Finally, lab personnel enter all the results of soil and rock analyses, including moisture, size analysis, California bearing ratio tests, and a variety of strength tests. Figure 7 shows the summary table for soil classifications, in which raw size data are used to derive a variety of soil classifications required for engineering designs.

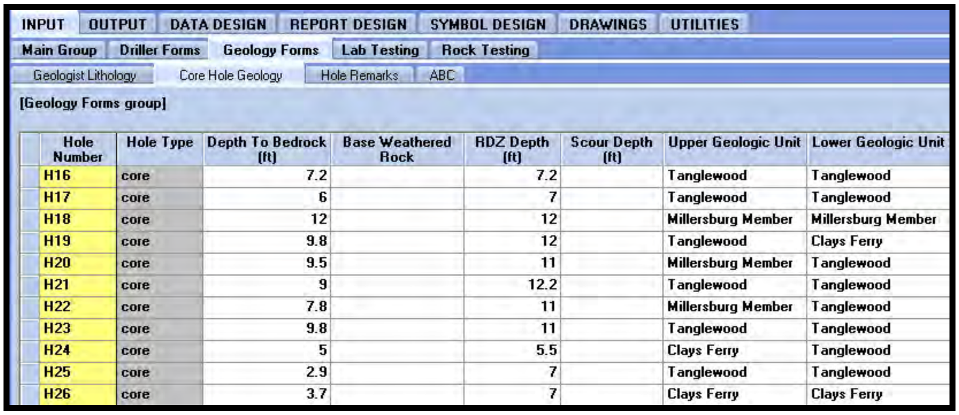


INPUT | OUTPUT | DATA DESIGN | REPORT DESIGN | SYMBOL DESIGN
Main Group | Driller Forms | Geology Forms | Lab Testing | Rock Testing
Ground Water | Core Runs | Driller Soil Description | Driller Rock Description

[Driller Forms group]

	Date	Reading Type	Water Depth (ft)	Dry
	8/15/2006	immediate		☑
	8/22/2006	static	0.3	☐
*				☐

Figure 5. Data-entry form in gINT software, for groundwater readings.

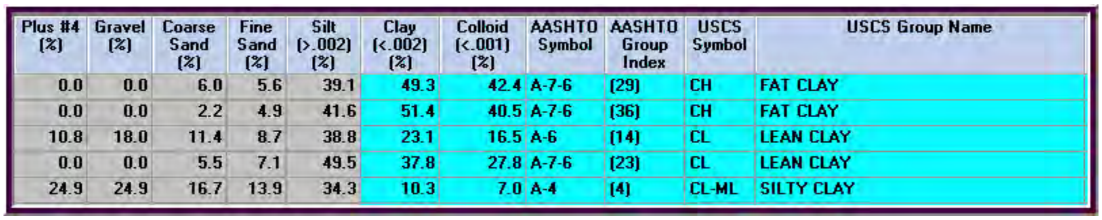


INPUT | OUTPUT | DATA DESIGN | REPORT DESIGN | SYMBOL DESIGN | DRAWINGS | UTILITIES
Main Group | Driller Forms | Geology Forms | Lab Testing | Rock Testing
Geologist Lithology | Core Hole Geology | Hole Remarks | ABC

[Geology Forms group]

Hole Number	Hole Type	Depth To Bedrock (ft)	Base Weathered Rock	RDZ Depth (ft)	Scour Depth (ft)	Upper Geologic Unit	Lower Geologic Unit
H16	core	7.2		7.2		Tanglewood	Tanglewood
H17	core	6		7		Tanglewood	Tanglewood
H18	core	12		12		Millersburg Member	Millersburg Member
H19	core	9.8		12		Tanglewood	Clays Ferry
H20	core	9.5		11		Millersburg Member	Tanglewood
H21	core	9		12.2		Tanglewood	Tanglewood
H22	core	7.8		11		Millersburg Member	Tanglewood
H23	core	9.8		11		Tanglewood	Tanglewood
H24	core	5		5.5		Clays Ferry	Tanglewood
H25	core	2.9		7		Tanglewood	Tanglewood
H26	core	3.7		7		Clays Ferry	Clays Ferry

Figure 6. Form for entering a geologist's rock evaluations in gINT software.



Plus #4 (%)	Gravel (%)	Coarse Sand (%)	Fine Sand (%)	Silt (>.002) (%)	Clay (<.002) (%)	Colloid (<.001) (%)	AASHTO Symbol	AASHTO Group Index	USCS Symbol	USCS Group Name
0.0	0.0	6.0	5.6	39.1	49.3	42.4	A-7-6	(29)	CH	FAT CLAY
0.0	0.0	2.2	4.9	41.6	51.4	40.5	A-7-6	(36)	CH	FAT CLAY
10.8	18.0	11.4	8.7	38.8	23.1	16.5	A-6	(14)	CL	LEAN CLAY
0.0	0.0	5.5	7.1	49.5	37.8	27.8	A-7-6	(23)	CL	LEAN CLAY
24.9	24.9	16.7	13.9	34.3	10.3	7.0	A-4	(4)	CL-ML	SILTY CLAY

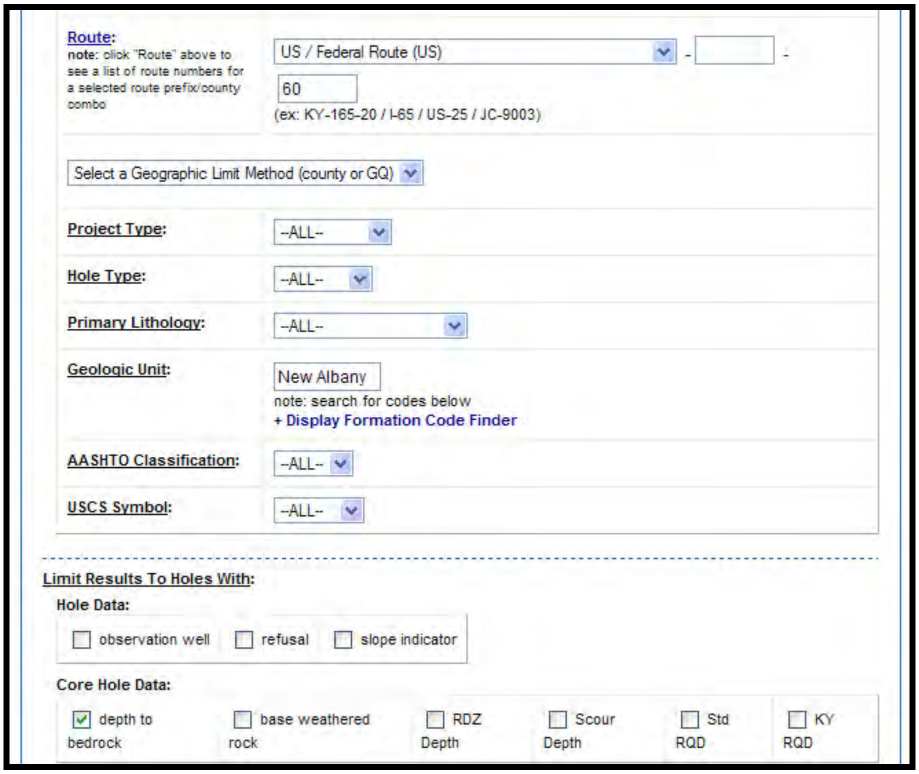
Figure 7. Soil classification form in gINT software, showing soil classifications calculated from raw input data.

Each data-entry form represents a table in the Access database, and may have one or more Visual Basic subroutines assigned that are executed at different times relative to saving the dataset. These procedures are used extensively to validate data entry, to initialize new records in related tables, to perform calculations, and to issue error messages.

Use of the gINT software product allowed KYTC to maintain standardization throughout the database. It also provided a mechanism to enforce standards for projects that are subcontracted to consultants. All consultants are required to use the same software, and KYTC supplies each vendor with a preformatted database that includes the list of holes to be drilled. The contractors enter the data when holes are completed and return the gINT database to KYTC at the end of the project.

Since the gINT application was completed in early 2007, 351 projects with 5,512 holes have been entered and archived in the KGS SQLServer database using the PLog software. Currently, a system is being implemented that will allow KYTC personnel to remotely upload the completed project files into the KGS database.

The final task of this phase was to develop Web services to search the drillhole database and return a variety of tabular and formatted reports. These services were developed using ASP, and are available to the public (http://kgs.uky.edu/kgsmap/gINT/gINTSearch.asp). Figure 8 is the search form that permits queries ranging from any information for holes drilled by a specific project, to all holes containing a specified soil type. The criteria can be geographic, stratigraphic, the presence of specific soil or rock types, or particular kinds of analyses. The result of a query is a list of all projects that contain at least one hole that met the criteria (Figure 9). The project summary line contains a link to the final report as well as a hole summary report that contains descriptive information and geotechnical values such as depth to rock and allowable bearing capacity. Each project line can be expanded to show a list of holes, and each hole has a page containing links to available reports such as the geologist's log and any soil or rock test results (Figure 10).



Route:
note: click "Route" above to see a list of route numbers for a selected route prefix/county combo

US / Federal Route (US) - -

60

(ex: KY-165-20 / I-65 / US-25 / JC-9003)

Select a Geographic Limit Method (county or GQ)

Project Type: --ALL--

Hole Type: --ALL--

Primary Lithology: --ALL--

Geologic Unit: New Albany
note: search for codes below
+ Display Formation Code Finder

AASHTO Classification: --ALL--

USCS Symbol: --ALL--

Limit Results To Holes With:

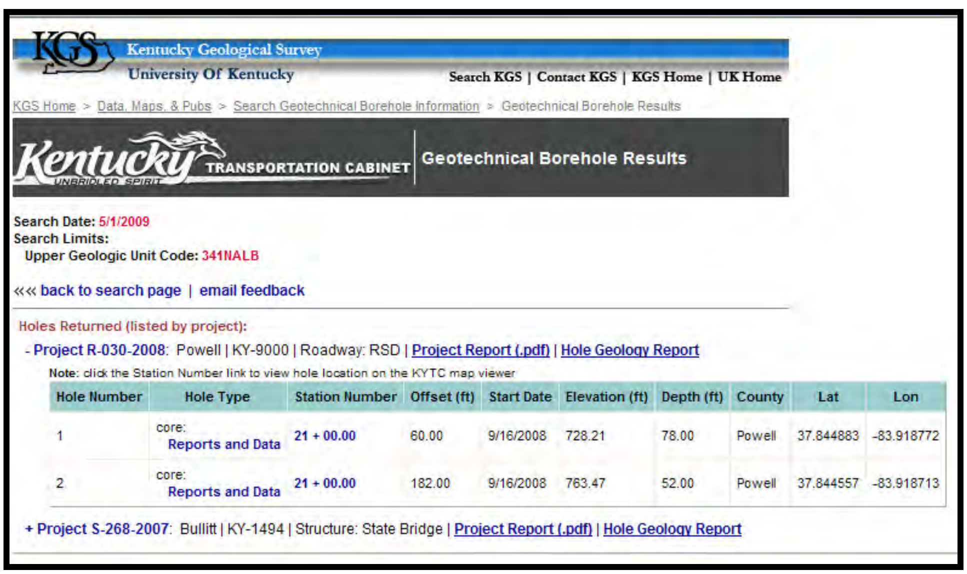
Hole Data:

☐ observation well ☐ refusal ☐ slope indicator

Core Hole Data:

☑ depth to bedrock ☐ base weathered rock ☐ RDZ Depth ☐ Scour Depth ☐ Std RQD ☐ KY RQD

Figure 8. Online search form for finding holes that contain specified kinds of information.



Kentucky Geological Survey
University Of Kentucky
Search KGS | Contact KGS | KGS Home | UK Home

KGS Home > Data, Maps, & Pubs > Search Geotechnical Borehole Information > Geotechnical Borehole Results

Kentucky UNBRIDLED SPIRIT TRANSPORTATION CABINET Geotechnical Borehole Results

Search Date: 5/1/2009
Search Limits:
Upper Geologic Unit Code: 341NALB

««« back to search page | email feedback

Holes Returned (listed by project):

- Project R-030-2008: Powell | KY-9000 | Roadway: RSD | Project Report (.pdf) | Hole Geology Report

Note: click the Station Number link to view hole location on the KYTC map viewer

Hole Number	Hole Type	Station Number	Offset (ft)	Start Date	Elevation (ft)	Depth (ft)	County	Lat	Lon
1	core: Reports and Data	21 + 00.00	60.00	9/16/2008	728.21	78.00	Powell	37.844883	-83.918772
2	core: Reports and Data	21 + 00.00	182.00	9/16/2008	763.47	52.00	Powell	37.844557	-83.918713

+ Project S-268-2007: Bullitt | KY-1494 | Structure: State Bridge | Project Report (.pdf) | Hole Geology Report

Figure 9. Primary search result report, listing all projects that met the hole search criteria.

Kentucky Unbridled Spirit TRANSPORTATION CABINET | KYTC Information Service Kentucky Geological Survey

KGS Home > Data, Maps, & Pubs > KYTC Data Results

KYTC Reports and Data (Project: R-065-2007 | Hole: 69)

Logs (sample):
- Geologist Log (.pdf)
- Driller Log (.pdf)

+ Soil Summary:

Note: instruction place holder

Depth	Sample Type	Sample Number	Penetrometer	Description	Sieve Type	Proctor	CBR	Consolidation Report	QU Report	UU Report	CU Report
2.5 ft	ST	1	2.5	Dark tan & gray, elastic silt.	With Gravel	NO	NO				
7.5 ft	ST	2	3	Brown & tan, fat clay with sand.	With Gravel	NO	NO				
12.5 ft	ST	3	4	Reddish brown & yellow, fat clay with sand.	With Gravel	NO	NO				
17.5 ft	ST	4	4.5	Yellowish tan, fat clay.	With Gravel	NO	NO				REPORT - B (.pdf)
22.5 ft	ST	5	4.5	Yellowish & copper, lean clay with sand.	With Gravel	NO	NO				
27.5 ft	ST	6		No Specimen	None	NO	NO				

Figure 10. Summary of soil samples for a single drillhole, with links to online reports containing analytical results.

INTEGRATING GEOTECHNICAL DATA WITH GEOLOGIC MAPS

Future additions to this system will include search functions to return tabulated test results of specified parameters for regional analysis and modeling. For example, all strength tests for clay soil types could be extracted to facilitate comparison to mapped soil or geologic units. Because all the holes are geographically referenced in the database, it will be possible to integrate information from these holes with the statewide geologic map database developed from Kentucky's 7.5-minute geologic map series. This online, interactive geologic map site (http://kgs.uky.edu/kgsmap/KGSGeology) has the ability to display derivative classifications of geologic units. The availability of this extensive geotechnical data will provide opportunities to add new classifications that will facilitate the use of geologic maps by engineers.

GEOTECHNICAL ASSET MANAGEMENT PERFORMANCE MEASURES FOR AN UNSTABLE SLOPE MANAGEMENT PROGRAM

David A. Stanley, C.P.G.
Chief Engineering Geologist
Alaska DOT&PF
5800 E. Tudor Road
Anchorage, AK 99507
Dave.Stanley@alaska.gov

Lawrence A. Pierson, C.E.G.
Senior Associate – Landslide Technology
10250 SW Greenberg Rd., Suite 111
Portland, OR 97223
Lawrencep@landslidetechnology.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

Performance Management (PM) translates strategic agency goals into detailed measures that are tracked to ensure achievement of the goals. Transportation Asset Management (TAM) is a strategic process for operating and maintaining physical assets throughout their lifecycle. These closely-related management frameworks are used by numerous transportation agencies throughout the US and around the world, sometimes individually, sometimes in concert. Together, these systems rely on performance measures as essential tools for assessing asset conditions and an agency's service delivery. Performance measures monitor progress toward a goal and may be defined most simply as indicators of the quality of work performed and results achieved. For asset management programs, asset condition and performance is compared to performance measures and the required level of service in order to assess an agency's delivery of services and to evaluate alternatives for maintenance, repair or replacement whenever these assets are not meeting established performance standards.

Geotechnical Asset Management (GAM), a relatively new entry into the TAM world in the last ten years, is beginning to receive national attention and interest. Asset inventories, condition surveys and hazard ratings for geotechnical assets such as retaining walls, material sites and unstable slopes are being conducted as first steps of asset management programs, but most efforts have not been integrated with TAM programs or made consistent with agency strategies and goals for performance management. For geotechnical assets where inventories and condition surveys are fairly common, it is the performance measures that are generally missing or largely undeveloped. This paper provides example performance measures and levels of service for an Unstable Slope Management Program (USMP) that is being developed by the Alaska DOT&PF to support future decision-making for alternative actions. The principles and examples presented here can be used as a starting point for integrating a broader spectrum of geotechnical into agency PM and TAM programs.

INTRODUCTION AND BACKGROUND

In 2008, the Alaska Department of Transportation & Public Facilities (AKDOT&PF), having recognized the risks associated with unstable slopes and the potential for acting proactively to manage slopes, rather than react to failures, allocated funding for research and literature review to kick off development of an Unstable Slope Management Program (USMP). A subsequent research effort completed by the University of Alaska Fairbanks and published in a 2009 report provided the framework for an unstable slope rating system. Also in 2009, AKDOT&PF initiated a three-year long Statewide Transportation Improvement Program USMP implementation project. These combined research efforts have resulted in development of the USMP. Currently the USMP project has nearly completed the development phase and is partially deployed with inventory and condition surveys underway for the most critical slopes affecting the transportation system.

The USMP was developed in part along the models of hazard rating for rockfall hazards. The principal model for the USMP hazard rating architecture is the "Rockfall Hazard Rating System" created in the 1990s. (*2, 3*) In addition to the technical aspects informed by the work of Pierson, et al., the USMP is founded on the principles of transportation asset management and performance measurement. The goals of the USMP include:

- Providing safer and more cost effective transportation system;
- Demonstrating to the public that a specific program is in place to address possible safety concerns and inconvenience due to unstable slopes and to assure the public that these safety problems are being proactively addressed in a timely manner;
- Providing a basis for AKDOT&PF maintenance and geotechnical staff to consistently manage this geotechnical asset;
- Providing AKDOT&PF managers with current information that is regularly reviewed and updated by appropriate technical staff in order for them to make informed project development and mitigation decisions;
- Forming the basis for consistent and improved working relationships within the department and in particular between DOT maintenance and geotechnical personnel; and
- Resulting in a reduction in maintenance and operations, design, and construction life-cycle costs related to unstable slopes.

Performance Measures, Performance Management, Transportation Asset Management and Geotechnical Asset Management are all elements of management systems that have become essential to the operation of modern transportation agencies across the country and around the world. As transportation systems become larger, more complex, and more expensive to build and preserve, management systems become more important to maintain control over function and cost of these infrastructure networks. Many agencies look to performance management as a means to manage transportation systems and activities, and to provide the tools needed to properly manage these physical assets.

Transportation agencies at the local, state and federal level have embraced the concepts of Performance Management (PM) and Transportation Asset Management (TAM) as a way to link agency goals with resources and results and improve long-term management of government assets. (*4, 5, 6*) Overall, both concepts are still developing - some states and agencies have a few well-established programs, while others are just beginning the process. There is no one-size-fits-all format and each agency's unique mix of goals, strategies, resources and other imperatives drive the design of its management programs. (*7*) The success of both Performance Management and Transportation Asset Management is gauged by observing or measuring established performance standards. These standards include both qualitative and quantitative measures. Qualitative measures may describe the public's perception of asset condition (e.g., rating pavement ride as good, adequate or poor), or how well an agency delivers a particular service (e.g., wait times at the Department of Motor Vehicles). (*7*, at Ch 5) An example of a quantitative performance standard is highway safety as measured by the yearly rate of traffic fatalities.

Performance Management is by intention an iterative process that translates strategic goals into relevant and detailed measures and targets used to guide investment of both human and capital resources. (*8, 9*) Performance Management forms the framework for an agency to set strategic agendas and motivate its staff. It improves the business processes of the agency and focuses efforts on accountability with stakeholders and customers. (*6* at pg. 2 and 9)

Transportation Asset Management addresses the physical assets of an agency. It is "a strategic and systematic process of operating, maintaining, upgrading and expanding physical assets effectively throughout their life cycle. It focuses on business and engineering practices for resource allocation and utilization, with the objective of better decision making based on quality information and well defined objectives." (*7* at pg. 1-12) The purpose of Transportation Asset Management includes three key concepts: "to meet a **required level of service**, in the most **cost effective manner**, through the management of assets for **present and future customers**." (*7* at pg. 1-12; bold in original)

In this paper we address performance measures and the related concept of "level of service" as they are being developed by the AKDOT&PF. Level of Service (LOS) is a classification or standard that defines the quality expectations for a specific facility or service, against which performance can be measured. Meeting the LOS goals is determined by how well or how often the evaluation standards are satisfied.

Performance measures are indicators of the value of the service provided by the transportation system to users. They may be known as objectives, metrics, targets, etc., but they are collectively referred to as "performance measures." Performance measures may be employed at all levels within an agency, from the highest strategic and policy level to the program or project working level. Strategic performance measures may include general condition, life cycle cost, safety, mobility, accessibility, reliability, comfort or convenience, external effects, risk, and preservation, among other concepts (*7* at Chapter 5). More sharply focused numerical measures can include concepts such as: sign retro-reflectivity, bus ridership level, percentage of structurally deficient bridges, and average International Roughness Index for pavements.

STRATEGIC PRINCIPLES, GOALS, POLICIES AND PROGRAMS

A detailed look at the Department's long-range plan and strategic goals illuminates several key policies for PM and TAM. The AKDOT&PF 2008 Strategic Plan (2008 Strategic Plan) (*10*) provides a framework for the agency's transportation goals (see Table 1, *11*). These goals directly apply to the desired outcomes for the USMP as noted.

Table 1: AKDOT&PF 2008 Strategic Plan

Strategic Plan (2008)

Goal	Summary of Aspects Relevant to the USMP
1. **Safety**: Improve the safety of the transportation system	Fewer lives lost, roads upgraded to national highway standards, airports open 24 hours.
2. **Economic Development**: Develop a transportation system that supports and promotes economic development	Ready road system for gas line construction and build new roads for access to resources.
3. **Performance**: Improve our individual and collective performance	Provide customers with best possible service.
4. **Openness and Honesty**: Conduct business in an open and honest manner	Promote understanding by public of process to construct, maintain and operate transportation system through a dialogue with the public.
5. **Staff Improvement**: Promote career growth and safety for all staff	Provide opportunity for advancement and promote safety for all employees.

The more forward-looking, federally-mandated AKDOT&PF Statewide Long-Range Transportation Policy Plan - Let's Get Moving 2030 (2030 Plan) (*11*), expresses several guiding principles resulting from a dialogue with stakeholders, technical analyses, and recognition of the transportation challenges that face Alaska. The guiding principles include: getting the most value possible from transportation funding; conducting statewide planning for resource allocation; recognition of statewide priorities for preservation, operation and future development of the system; managing the system efficiently; and optimizing the use of new technologies to improve performance and increase efficiency. The 2030 Plan has enumerated fourteen policies that guide the development of strategies and actions for the Department. Several of these have direct application to transportation asset management, as set out in Table 2. The Strategic Plan goals shown in Table 1 and the 2030 Plan policies shown in Table 2 are directly related.

These goals enumerated in these plans interact with the AKDOT&PF Performance Electronic Tracking System (PETS) that is currently under development. When fully deployed, this tracking system will be used to create and revise performance measures for all levels of the Department, as discussed below under Performance Management.

Table 2 - Relevant Policies: Alaska's Statewide Long-Range Transportation Policy Plan

Theme/Policy Number	Policy	Summary of Aspects Relevant to the USMP
System Development	Develop the multi-modal transportation system to provide safe, cost-effective, and energy-efficient accessibility and mobility for people and freight	Apply cost-effectiveness criteria to project recommend for funding.
System Development	Establish statewide strategic priorities for transportation system development funding	Continue modernizing the Highway System to meet current standards for safety and connectivity
System Preservation	Apply best management practices to preserve the existing transportation system	Use life cycle management practices and management systems to support asset management practices.
System Preservation	Increase understanding of and communicate ADOT&PF's responsibilities for system preservation as the owner of highways, airports, harbors and vessels	Monitor and report annually the condition and value of assets; the anticipated level of service and predict future system conditions based on allocation of preservation and maintenance funds
System Management and Operations	Ensure the efficient management and operation of the transportation system	Preserve transportation corridors and pursue corridor management
System Management and Operations	Use cost effective technology and Intelligent Transportation Systems to ensure the efficient operation of the transportation system, accessibility, and customer service	Apply research results and technology transfer to our design, construction and maintenance practices to reduce costs and improve efficiency and safety
Economic Development	Preserve and operate Alaska's multimodal transportation system to provide efficient reliable access to local, national, and international markets	Preserve and modernize the system; recognize the need for continued system development; maintain and operate the system to provide reliability and performance
Safety	Increase the Safety of the transportation system for users of all modes:	Implement strategies in safety plans; ensure safe transportation by means of timely compliance with national safety standards.
Good Government: Openness and Accountability for Transportation System Performance	The statewide plan will provide the analytical framework:	Monitor, forecast, and report system performance through management systems; provide information to support performance-based planning and budgeting.

PERFORMANCE MANAGEMENT

Performance Management Policies and Goals

Agencies establish mission statements and values to focus their actions and behaviors. Through a very conscious process, they define goals that reflect department values and provide a target for planning and guiding agency actions. In turn, goals require policies to establish the rules of engagement for employee performance and how work is accomplished. At the highest policy setting level, long range and strategic planning is used to develop systems and implementation programs that will satisfy agency objectives. The expression of these combined objectives occurs at the System level where broad performance measures are defined. Program managers evaluate the level of success with a set of detailed performance metrics designed to optimize available resources for attaining these program level performance goals.

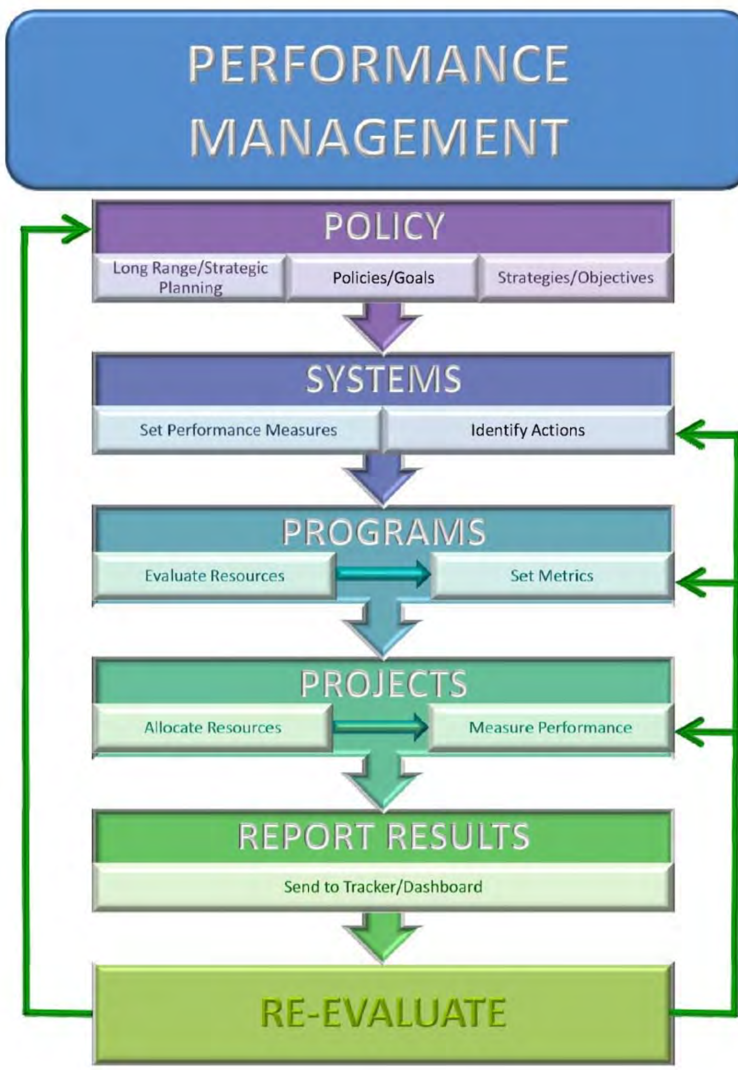


Figure 1 - Performance Management Flowchart

At the project level, performance metrics are used to assess the health of a project's performance and specific physical assets, and to gauge how well a project meets customer requirements and provides the intended value. Project managers allocate the resources and perform the actions needed to meet the metrics and measure performance.

The results of these evaluations are reported to management and collated in a tracking database which connects with a web-based tracking system, referred to as a scorecard or dashboard. The dashboard is a public and transparent expression of the agency's performance that also serves to inform the executive whether the goals and objectives are being met.

Throughout this process, the policies, measures and metrics are re-evaluated to ensure they are adequate to characterize performance and that the program and project resources are appropriate for achieving the goals and objectives set by the executive level.

Performance Electronic Tracking System

A performance management system defines and tracks the relationship between strategic goals and performance results. AKDOT&PF has established a Performance Electronic Tracking System (PETS) as a performance management tool. The system is still in the development stage, but it is being tested as a means to create performance measures (metrics) and scorecards for a variety of management and functional groups within the Department.

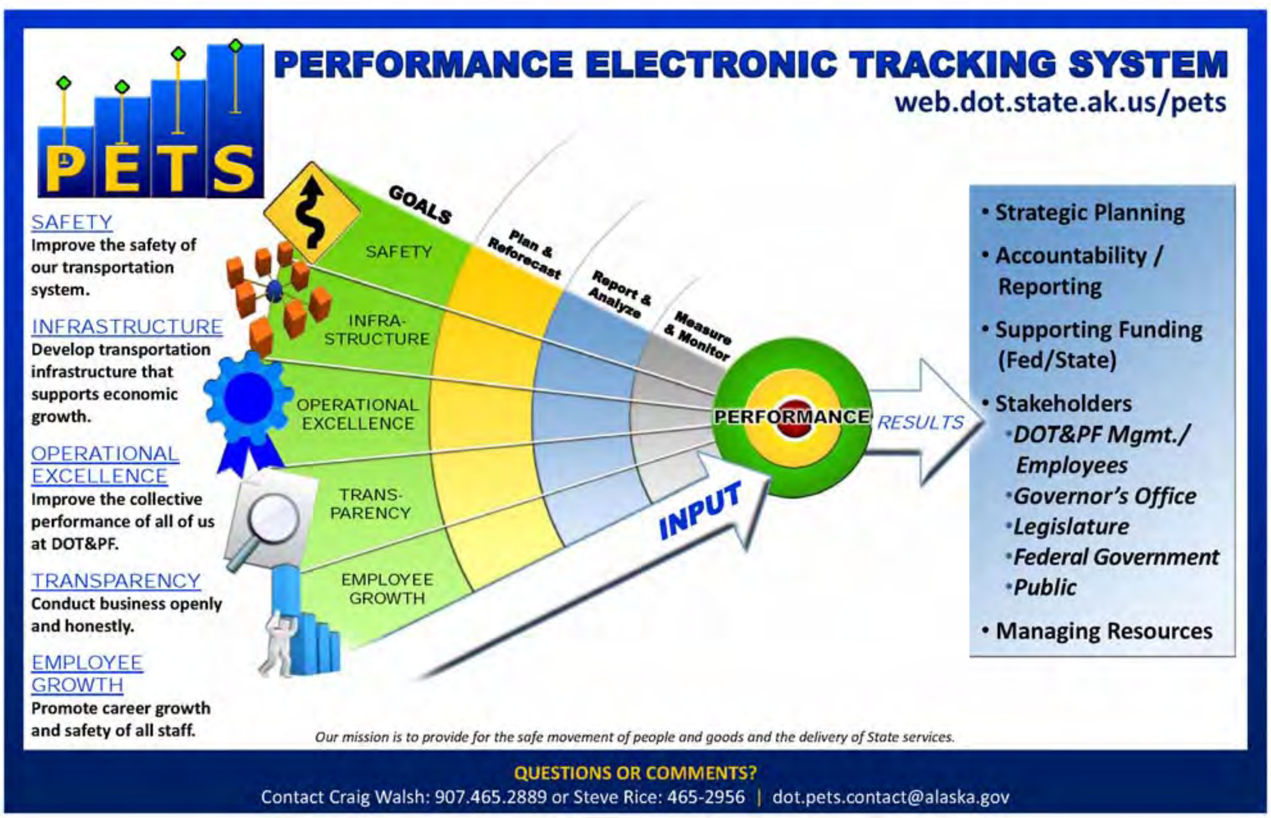


Figure 2 - Performance Electronic Tracking System
(AKDOT&PF Internal Intranet PETS Website)

AKDOT&PF uses PETS to create and summarize the results of performance metrics and to provide reports and visual representations of those results. The visual summaries indicate level of compliance with goals and can be in the form of a scorecard represented as bar graphs or "dashboards" that uses clusters of "gauges." PETS can report on both qualitative measures such as customer satisfaction with wait time at Department of Motor Vehicles as well as quantitative measures such as traffic fatality rates or percentage of structurally deficient bridges.

In Figure 3 below captured from the AKDOT&PF internal intranet PETS website, several metrics are listed on the left side. On the right side, a history bar chart for a single element (structurally deficient bridges) is shown for a particular reporting period. Once fully operational, PETS will allow administrators, technical staff, legislators, the public, and other customers to get a quick view of AKDOT&PF performance and a detailed look at the performance of specific agency functions. This transparency will improve our connection to customers and stakeholders.

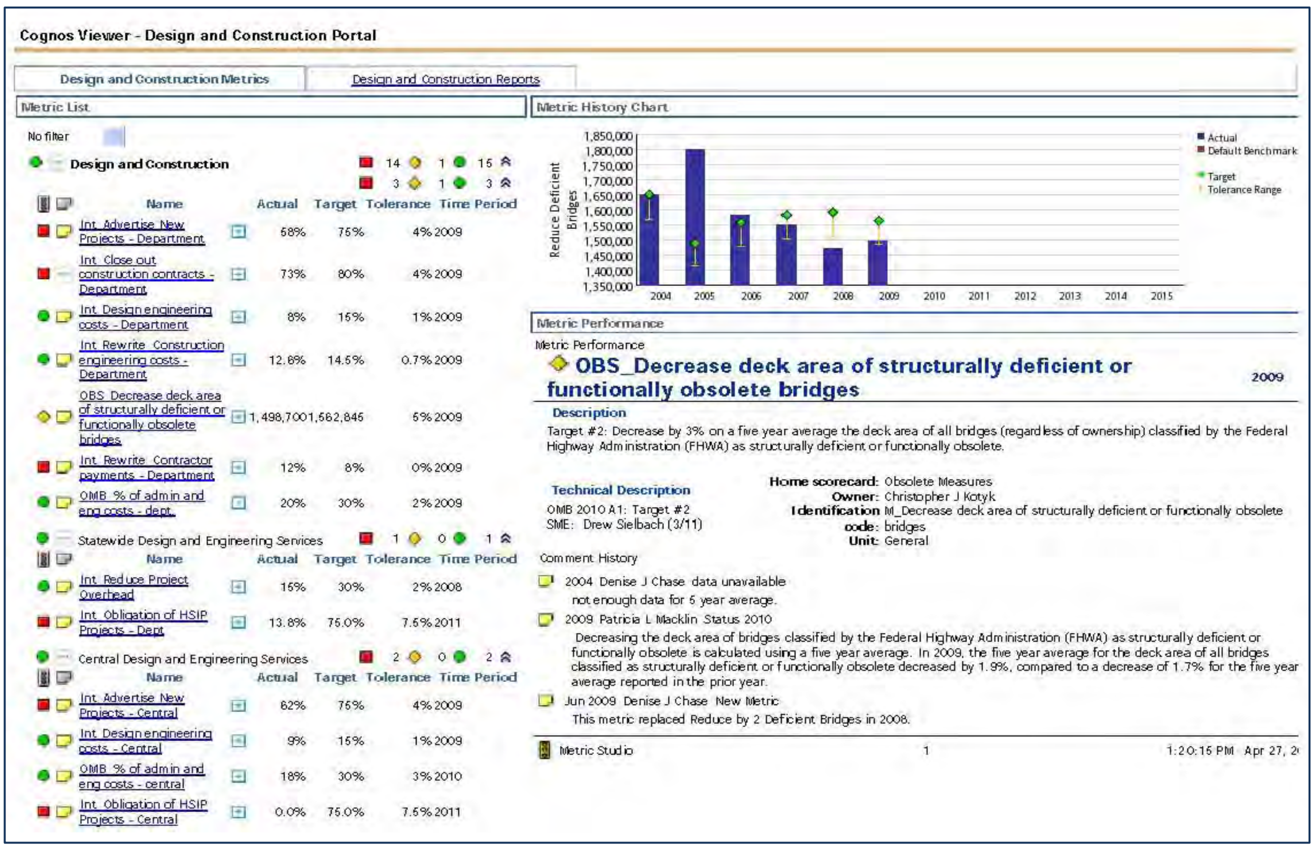


Figure 3 - Performance Electronic Tracking System Report
(AKDOT&PF Internal Intranet PETS Website)

As noted, the PETS program is still under development and is not available for public viewing at present. As AKDOT&PF is re-evaluating its strategic planning goals, the performance measures contained in the PETS system will likely undergo modification before public release.

Dashboards

The commercially available PETS software provides the options to report results in a dashboard format similar to those shown on Figure 4. The final appearance of this reporting format for AKDOT&PF has not been determined but it will likely be similar to this example. This simplified reporting format will broaden the appeal for the public to keep up with the agency's performance. In addition to the public, this readily accessible visual format should make it quicker for all involved to stay abreast of progress.

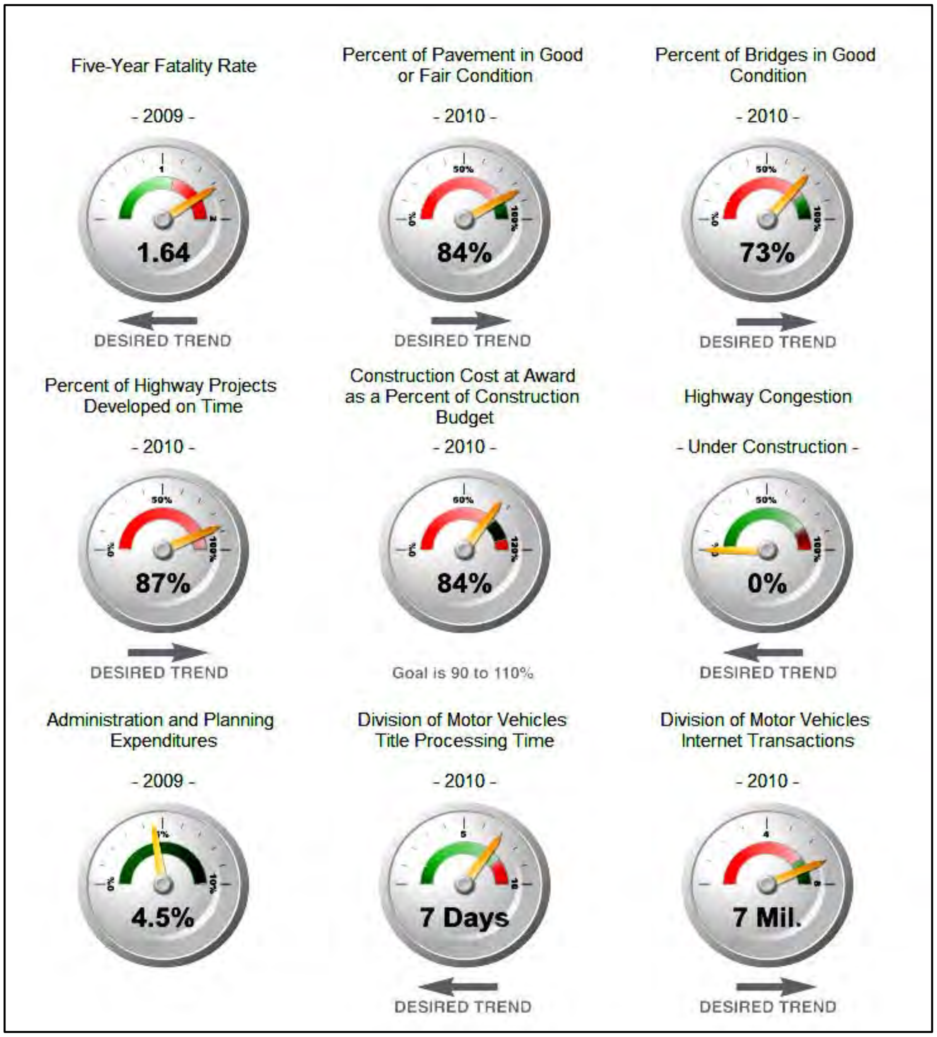


Figure 4 – Reporting Dashboard
(Idaho Transportation Department, 2010)

AKDOT&PF TRANSPORTATION ASSET MANAGEMENT

Policy Framework

AKDOT&PF has made a commitment to adopt Asset Management as a means of conducting its business. While Alaska is just beginning the process of organizing asset management as an overall guiding principle, many aspects of AM have been utilized in the past in the Department's pavement, bridge and data management programs. However, these efforts have been largely independent with little interaction within the organization on a broader AM perspective. In the future, the Department will need to break down organizational boundaries in order to satisfy the strategic goals and policies set by the AKDOT&PF Commissioner and Alaska's Governor to fully integrate AM principles.

The general scheme of Asset Management Policies and Goals is shown in the flow diagram in Figure 5. This diagram illustrates the relationship between the AKDOT&PF Strategic Plan and the 2030 Plan, and the Performance Management (PETS) program and how these key elements relate to TAM and Geotechnical Asset Management.

Figure 5 – Goals and Policies Relationships to Asset Management

This flow diagram shows the framework of present AKDOT&PF agency goals applicable to performance management and asset management as related to the nascent geotechnical asset management program. Nearly all agency goals and policies predate the Department's commitment to AM implementation. [Note: The AKDOT&PF Strategic Plan is undergoing revision at the time of this writing and the agency is in the early stages of implementation of TAM and PM. Some of the performance measures detailed in this paper will likely be revised in response to new strategic initiatives. However, this iterative process is considered a best practice within the AM community].

The Department's Strategic Plan is a brief but critically important document. As introduced above and shown in Table 1, it enumerates five goals for the AKDOT&PF and establishes performance measures to meet those goals. The plan does not mention asset management specifically, but the goals contain performance measures and targets that directly relate to performance management and asset management principles and a geotechnical asset management program such as the USMP.

In addition, the 2030 Plan provides agency goals, principles and policies to guide the Department. Like the Strategic Plan, the 2030 Plan does not explicitly reference the integration of asset management into the Department either, but it does include significant language derived from asset management principles that can guide efforts to deploy and integrate AM. Neither plan contains a stated period of performance, but the Strategic Plan is implicitly shorter term than the 2030 Plan.

AKDOT&PF GEOTECHNICAL ASSET MANAGEMENT

Geotechnical Asset Management is an underdeveloped sector of Transportation Asset Management. (*12*, *13*) However, agencies have begun to sharpen their focus on geotechnical assets since so many other transportation assets including the roadway itself are directly related to geotechnical elements. The list of geotechnical assets is quite extensive. Important examples include: embankments, rock and soil slopes, material/quarry sites, retaining walls and their buried reinforcing elements, bridge foundations, rock bolts, horizontal drains, tie-back anchors and other buried structural components. GAM is new compared to longer-standing Pavement Management and Bridge Management programs, but, due to its critical relationship to roadway serviceability and the required capital investment and performance of the overall transportation system, it is now receiving the attention it deserves as an important part of an overall asset management program.

The AKDOT&PF Geotechnical Asset Management program is in the initial stage, and research initiatives are underway for the development and deployment of various GAM programs. Importantly, funding requests for a larger overall program are under consideration. A larger and more complete GAM program will eventually be implemented, but its scope and funding level are not yet defined.

To date, the GAM program includes a Material Site Inventory Program and the USMP. Both are being implemented in accordance with the AKDOT&PF policies and strategic goals and the principles of TAM and PM. As part of a phased research project, the USMP to date includes a defined rating system and an inventory database for unstable slopes. Current work includes conducting condition surveys of the “Top 100” maintenance-identified unstable slopes affecting the AKDOT&PF transportation system. However, since so many of the identified sites have been subdivided for a variety reasons, the database contains over 190 unstable slopes including both landslides and rockfalls.

The USMP will include performance measures, levels of service (LOS) and other asset and program performance concepts in order to fully function as a TAM/GAM program. The LOS targets and performance measures are required for properly managing and investing in these unstable slopes. This will provide the detailed performance information AKDOT&PF managers need to make well supported decisions on the repair, rehabilitation or replacement of these assets.

LEVELS OF SERVICE AND PERFORMANCE MEASURES

Good asset management requires that the right information is available to the right levels of the organization at the right time (*7* at Ch. 5). As these management systems develop and mature, the appropriate flow of information about performance of assets and the agency’s delivery of services becomes more complete and efficient. Levels of Service and Performance Measures are used throughout all levels of a Performance Management system as tools to allow creation of targets and for monitoring the agency’s progress over time (See Figure 1 - Performance Management).

At strategic policy/system levels, measures and metrics are created based on guidance from strategic policies and goals. The measures are created in the PETS system and imparted to staff at the program/project levels. At the program/project levels, staff measures performance compared to required levels of service and collects the results for reporting to the management levels. This process creates a circle of responsibility and communication throughout the Department. Results of performance measurements returning from the program/project levels are aggregated and transformed to scorecards, report cards, website dashboards and other means to track agency performance. This information is shared both within and outside the agency as performance reports, which provides a clear picture of how well the agency is meeting required levels of service and its strategic goals and policies.

Goals for Levels of Service and Performance Measures

The USMP Levels of Service are derived from a set of consolidated goals derived from the Strategic Plan and the 2030 Plan that include:

- Safety
- Infrastructure Development
- Efficient Management & Operations
- System Preservation & Asset Condition
- System Mobility & Reliability

(*10*, *11*)

Levels of Service and Evaluation Methods

Levels of Service (LOS) are key elements of performance management and asset management. They are classifications or standards that define quality for the services offered to transportation system users (*7* at Ch. 5). Levels of Service may be viewed as customer-related LOS or technical LOS. <u>Customer</u> levels of service relate to how the customer perceives the service in terms of tangible and intangible, but generally qualitative, criteria. However, at this stage of transportation asset management/performance management development, AKDOT&PF does not have data on customer expectations for USMP-related LOS or performance measures. <u>Technical</u> levels of service are usually expressed in quantitative terms used by technical staff. For purposes of this initial development, LOS will be simplified without the distinction. As the TAM/GAM programs mature, finer divisions may be made supported by an increased ability of AKDOT&PF to collect and analyze performance data and customer feedback.

Specific LOS and performance measures must satisfy the overarching policies and strategic goals of the agency. At this development stage of the Unstable Slope Management Program generalized LOS and related performance measures are needed. LOS must be objectively measurable (S.M.A.R.T. - specific, measureable, achievable, relevant, time-bound), so that performance can be audited and overall system improvements resulting from Asset Management efforts can be monitored (*7*, at § 5.2.3.1) in order to determine whether the LOS have been met.

Within this framework of goals and LOS, the USMP provides nine services that can be monitored and tracked by establishing and monitoring performance toward meeting target levels. (*14*) These services and the basis for measurement include:

> **Service 1.** Roadway safety from earth movements whether from rockfalls or landslides will be addressed within the limited resources available to the agency in a proactive manner, statewide. Performance based on a running total of projects advanced by each region to address slope or embankment stability issues with the overall goal of addressing all unstable slopes within 50 years;
>
> **Service 2.** Maintenance forces will monitor and patrol historically unstable areas in order to keep roadways clear of debris. Performance based on the number of hours each maintenance section spends on slope stability road patrols divided by the number of USMP sites within their section times the number of road miles within their section
>
> **Service 3.** If additional equipment is required, debris will be removed and the roadway passage reestablished within 8 hours of any event if it is deemed safe to enter the affected area and the amount affecting the roadway is less than 3,000 cubic yards of material. Evaluated based on the number of road closure days related to slope failures each year per Region or Maintenance District.
>
> **Service 4.** Regional (based on organizational boundaries) geotechnical personnel will be notified the same day of all road closing events lasting more than one hour or for any event that results in damage to a vehicle or personal injury. Evaluated based on the number of notifications made versus the number of measureable events that occur.

Service 5. If notified and there was no vehicle involvement or personal injury, the region geotechnical person will review the area in person as soon as possible and always within one week of the event. Evaluated based on the percentage of qualified events that are inspected by region geotechnical staff within the assigned time.

Service 6. If working conditions are deemed unsafe by maintenance, if an extended road closure in excess of one day is anticipated, or if an accident has resulted, the region geotechnical person will visit the site within 24 hours to provide technical guidance and direction. Performance based on the percentage of qualified events that are inspected by region geotechnical staff within the assigned time.

Service 7. Whenever a site is visited, the region geotechnical person will reevaluate the site using the USMP to determine if the assigned section rating and site priority should be adjusted. Performance based on the number of updated ratings compared to the number of sites visited.

Service 8. Public satisfaction with the GAM efforts and results related to unstable slopes will improve. Evaluated based on level of improved public perception, as verified by opinion polls/surveys or by a measureable reduction in the annual complaints received.

Service 9. The AKDOT&PF maintenance expenditures related to unstable slopes in each region will decrease 5% per biennium allowing more of the operations budgets to be expended on other agency needs. Performance based on downward trend of operating costs related to proper investments in unstable slopes.

Numerous different services could be evaluated besides those listed here, but these provide an initial step towards identifying criteria for the USMP that can clearly demonstrate improvements in how the state addresses road serviceability related to unstable slopes. Levels of Service will vary widely between individual assets, asset classes, and different agencies. LOS established initially must be periodically re-evaluated to ensure that they adequately characterize performance and are properly serving the needs of the agency.

Rating Service Performance

The basis for evaluating these services relies on a basic grading system. The grades are generic and would not be amended but grading criteria could be adapted for various rating schemes as appropriate. Measurement and evaluation methods could include numeric ratings (e.g., 0% - 100%), subjective ratings systems (e.g., excellent, good, adequate, poor, and unacceptable <u>or</u> graded, e.g., A, B, C, D, F) and other rating schemes. In other words, the rating system must be capable of integration into an indexed rating.

The USMP levels of service discussed below will be rated using the five grades as described in Table 3 – Level of Service Rating System. These grades will represent an evaluation of the agency's USMP performance measures.

Table 3 - Level of Service Rating System

LOS Number Grades of Service	LOS Grade	Explanation
A	Excellent	100% compliance with service criteria
B	Good	95% - 100% compliance with service criteria
C	Adequate	90% - 95% compliance with service criteria
D	Poor	80% - 90% compliance with service criteria
F	Unacceptable	< 80% compliance with service criteria

LEVELS OF SERVICE AND PERFORMANCE MEASURES FOR AN UNSTABLE SLOPE MANAGEMENT PROGRAM

Table 4, below, provides a summary of recommended USMP-related Goals, Levels of Service, Performance Measures and Methods of Evaluation. This table shows the relationship between an agency's goals and policies and methods of establishing expected performance levels and measuring how these targets are met, including generalized means to conduct the measurement. The evaluation will utilize the rating system from Table 3, above.

As noted above, all aspects of the relationships shown are subject to re-evaluation and change over time. The Performance Management System flowchart presented in Figure 1, at page 7, explicitly includes a feedback loop for the technical aspects of setting metrics, measuring performance and reporting, as well as re-evaluation of agency goals and policies. It can be particularly important in development of performance measures to identify processes or data that are meaningful and can be measured. To serve as a warning, it is also established in transportation asset management processes that you can collect too much data and/or data that does not accurately reflect the performance of the asset under consideration.

The levels of service and associated performance measures shown below should give a good overall picture of unstable slope health. The measures include technical aspects directly related to the condition of the slopes, operational elements that will address the agency's reaction to slope failures, reliability- and customer relations-related measures of how well the agency keeps roadways functioning, and finally to track the operational and performance information related to the cost of unstable slopes.

Table 4 - Levels of Service and Performance Measures for USMP

Applicable Goals and Policies	Level of Service	Performance Measures	Method of Evaluation
Safety, Efficient Management & Operations, System Preservations & Asset Condition, System Mobility & Reliability	Service 1. Limit road closures due to rockfall or landslide incidents.	Number of completed unstable slope mitigation projects per year per Region.	Monitor running total of number of unstable slope mitigation projects completed per year needed to address all A and B unstable slopes in each Region within 25 years.
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 2 – Adequate monitoring and road patrols for unstable slopes.	Number of hours Maintenance and Operations (M&O) spends on monitoring unstable slopes and on road patrols for unstable slopes.	Record number of hours M&O district spends on slope stability patrols divided by the number of USMP sites in the District times the number of road miles in the district.
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 3 – Re-open roadways after closure due to unstable slopes as soon as possible. For events of less than 3,000 cubic yards, reopen within 8 hours unless site unsafe without stabilization.	Time of road closures due to slope failures before re-opening.	Evaluate based on the number of road closure hours per year per M&O District.
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 4 – Timely notification of Regional geotechnical personnel.	For events involving more than an eight hour road closure; or a property damage or personal injury accident. Time between incident and notification of Geotechnical personnel.	Evaluate based on number of notifications made within 24 hours of qualifying event per year per M&O District.
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System	Service 5 – Timely response by geotech staff for non-emergency incidents with no injury, property, or road closure in excess of one hours.	For non-emergency instances, a regional geotechnical staff will review the area in person not later than within seven days of the event.	Evaluate based on number of days between each qualifying event and site visit by geotech staff per year per M&O District.

Table 4 - Levels of Service and Performance Measures for USMP, Cont.

Applicable Goals and Policies	Level of Service	Performance Measures	Method of Evaluation
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 6 – Timely response of geotechnical staff within 24 hours or sooner for emergency incidents that result in injury or public property damage or that close the road	For emergency instances, a regional geotechnical staff will review the area in person within 24 hours or sooner after the event.	Evaluate based on number of hours between event and site visit by geotech staff and number of qualifying incidents and response time per year per M&O District
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 7 – Keep USMP database current by frequently reevaluating USMP sites and entering new data into USMP database in order to support goals.	Re-evaluate minimum of 10% of USMP sites each year.	Evaluate by comparing number of USMP site re-evaluations and data updates to number of USMP sites per Region per year.
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 8 - Improve public perception on the DOT efforts and responses to unstable slope conditions that affect the serviceability and safety of roads.	Measure public satisfaction with the USMP results. Show an improvement in public opinion (rating method to be developed) or a 10% reduction is public complaints about unstable slopes per year	Evaluate by comparing results of public surveys or by a measureable reduction in the annual complaints received regarding unstable slopes
Safety, Efficient Management and Operations, System Preservation & Asset Condition, System Mobility & Reliability	Service 9 - Reduce operating cost associated with unstable slopes	Show an annual 5% reduction in unstable slope related cost each biennium.	Evaluate by tracking cost related to unstable slopes such as costs for routine maintenance and for emergency response, Design Section costs for geotech staff for mitigation project design.

SUMMARY AND FUTURE CONSIDERATIONS

Performance Management and Transportation Asset Management use performance measures to monitor asset health and the success of delivery of agency services to customers. Geotechnical Asset Management is still under development and has not moved far beyond the basic steps of inventory and condition surveys. Performance management does not yet include GAM as a significant element of transportation systems. Alaska's Unstable Slope Management Program anticipates the need for development of performance measures to support implementation and deployment of GAM as a part of TAM and PM systems.

Beyond the immediate need for performance measures, GAM development must offer a simple usable form to transmit general information about the health of geotechnical assets. A general index may be derived by combining performance data for a representative group of geotechnical assets. This general index can be documented in its simplest form as a scaled index, and the variability can be tracked and reported periodically. As needed, a more detailed view or grouping of geotechnical assets or individual assets could be reported in supporting "tracking" publications or websites. Figure 6 shows a tiered layout of some potential asset indices and performance measures.

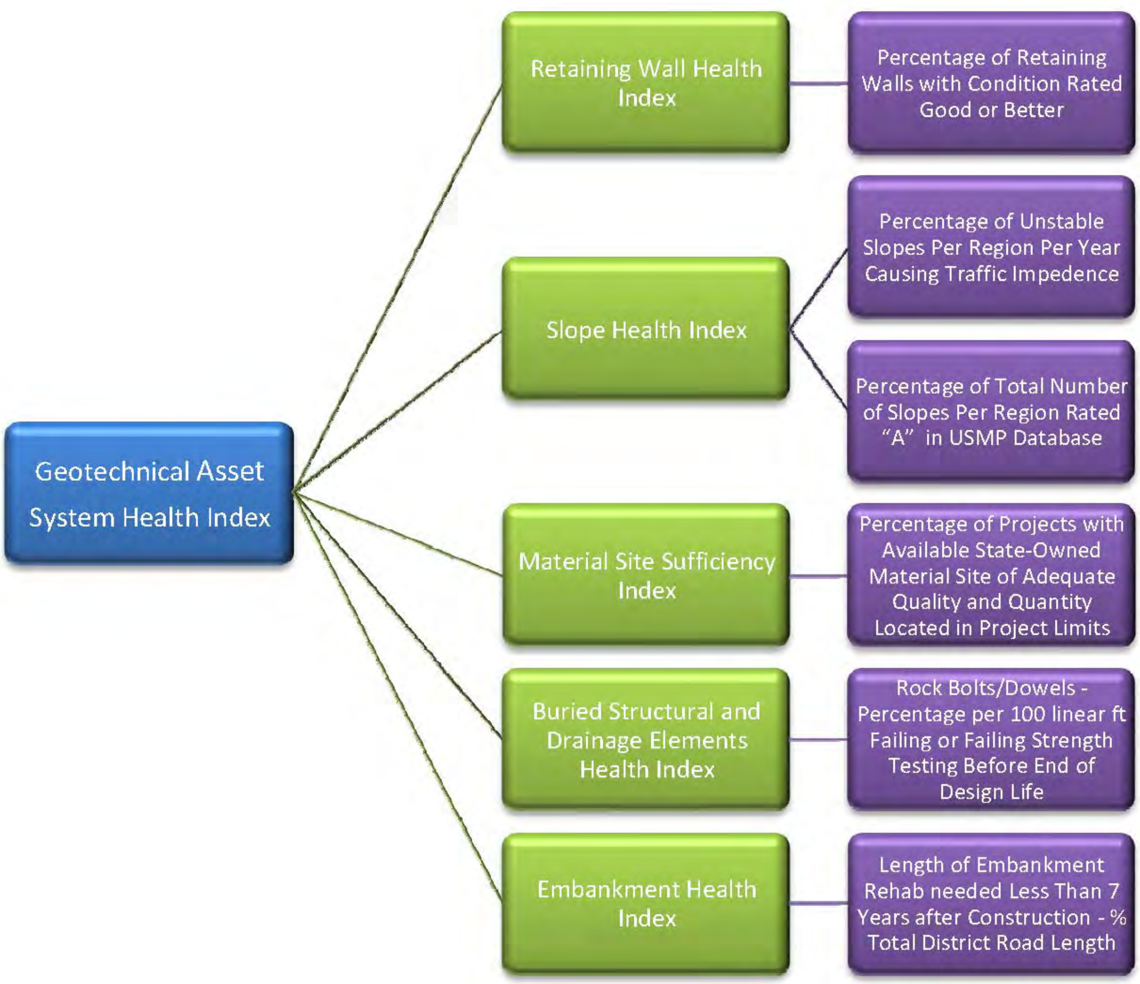


Figure 6 – Future Geotechnical Asset System Health Index

Performance measures directly related to geotechnical assets such as those presented here for unstable slopes provide a basis for devising performance measures for a broader array of geotechnical elements. For PM, these USMP performance measures provide important metrics that when combined with performance information about other geotechnical assets can offer a gauge of the health of an agency's infrastructure system. While beyond the scope of this paper, it will be possible in the future to shape a set of indices that collectively offer a usable picture of the health of geotechnical assets and their related, more conventional transportation assets.

REFERENCES

1. Huang, Scott L. and Darrow, Margaret, *Unstable Slope Management Program – Phase I*", University of Alaska Fairbanks, INE/AUTC RR09.16/G5491, FHWA-AK-RD-09-04, 2009.

2. Pierson, L.A., Davis, S.A., and VanVickle, R., *The Rockfall Hazard Rating System Implementation Manual*, Oregon Department of Transportation, Report No. FHWA-OR-EG-90-01, 1990.

3. Pierson, L.A. and VanVickle, R., *Rockfall Hazard Rating System – Manual*, FHWA-SA-93-057, 1993.).

4. AASHTO, *Transportation Asset Management Guide, Volume I,* 2002 (TAM Guide).

5. Cambridge Systematics, Inc., PB Consult, Inc., and Texas Transportation Institute, *Performance Measures and Targets for Transportation Asset Management*, National Cooperative Highway Research Program Report 551, Transportation Research Board of the National Academies, 2006.

6. Cambridge Systematics, Inc. and High Street Consulting Group, *Transportation Performance Management: Insight from Practitioners,* National Cooperative Highway Research Program Report660, Transportation Research Board of the National Academies, 2010.

7. AASHTO, *Transportation Asset Management Guide – A Focus on Implementation*, 2011 (TAM Guide II).

8. Cambridge Systematics, Inc., *Performance-Based Management: State-of-the-Practice White Paper* (*2009 CEO Leadership Forum*), 2009.

9. Federal Highway Administration, *Beyond the Short Term: Transportation Asset Management for Long-Term Sustainability, Accountability and Performance*, 2010.

10. Von Scheben, Leo, *Alaska Department of Transportation and Public Facilities Strategic Plan*, 2008.

11. Alaska Department of Transportation and Public Facilities, *Let's Get Moving 2030 – Alaska Statewide Long-Range Transportation Policy Plan*, 2008.

12. Sanford Bernhardt, K.L., Loehr, J.E., and Huaco, D., "Asset Management Framework for Geotechnical Infrastructure," *American Society of Civil Engineers Journal of Infrastructure Systems,* Vol. 9, No. 3, September 2003, pp. 107-116.

13. Stanley, David A., *Asset Management in a World of Dirt: Emergence of an Under-developed Sector of Transportation Asset Management*, Technology for Alaska Transportation, Research and Technology Transfer, Vol. 3 No. 2, 2011.

14. Pierson, Lawrence, *Performance Standards*, Personal Communication to David Stanley, 2010.

Identification and Mitigation of an Earth Fissure

Arizona State Route 303L; Glendale Avenue to Peoria Avenue Segment
Phoenix, Arizona

Scott D. Neely, P.E.
Terracon Consultants, Inc.
4685 S. Ash Avenue, Suite H-4
Tempe, AZ 85282
(480)-897-8200
sdneely@terracon.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The author(s) would like to thank the individuals/entities for their contributions in the work described:

Donald Clark, P.E. – Terracon Consultants, Inc.
Nick Priznar – Arizona Department of Transportation
Mike Rucker, P.E. – AMEC Earth and Environmental
Ralph Weeks, R.G. – GeoSouthwest
Mark Wayne, Ph. D., P.E. – Tensar International Corporation

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

The proposed alignment of the 4th segment of Arizona State Route 303L (SR 303L) between Glendale and Peoria Avenues, within the Phoenix metropolitan area is crossed by an existing earth fissure. In addition to the known earth fissure, there are several earth fissures that have been mapped by the Arizona Geological Survey, while other unconfirmed earth fissures have been identified by Terracon Consultants, Inc. during site reconnaissance of the project area. The known earth fissure and other unconfirmed earth fissures cross the mainline, and several ramps and cross roads within the project limits. A methodical approach consisting of numerous steps was agreed upon with the Arizona Department of Transportation to determine which earth fissures could be positively identified and which ones could be removed from further consideration on the unconfirmed earth fissure list.

The investigative information considered in our analyses included:

a. literature review,
    i. geology maps showing subsurface stratigraphy
    ii. earth fissure maps
    iii. lineament analyses of historical aerial photographs
b. historical survey information showing subsidence in the area
c. site reconnaissance,
d. seismic p-wave and ReMi field investigation and analyses,
e. earth fissure trench field investigation,
f. discussions with the ADOT Geotechnical group to decide upon the level of risk the state is willing to accept in design and construction of the highway.

Based on the information obtained from the foregoing efforts, those unconfirmed fissures not showing any substantial evidence for their existence were eliminated from the list of unconfirmed earth fissures. For earth fissures for which positive identification or substantial evidence for their possible existence was obtained, measures to mitigate their potential effects to the roadway were modeled by finite element analyses. Fortunately, none of the earth fissures cross any of the four bridge structures on the project, and therefore, only potential effects to roadway embankments and the west drainage channel were considered for design and construction and determination of mitigation measures.

The results of the finite element analyses indicated that a high modulus material is needed to distribute the strain caused by the potential differential movement that could impact the roadway embankments constructed across the location of the earth fissure. Based on our observations obtained from the earth fissure trench field investigation, the strain occurs over about seven feet as observed by the filled aperture and highly fractured broken zone on either side of the filled aperture. A five foot high zone of material having a Young's modulus of at least 10,000 psi constructed below the embankments will spread 6-inches of potential differential movement occurring over 7 feet beneath the embankment at the fissure, to over 30 feet at the pavement surface. This governed the design of the mitigative measures developed for the project.

INTRODUCTION

The Arizona Department of Transportation (ADOT) has identified an approximate 13 mile section of new freeway (State Route (SR) 303L) along the western portion of the Phoenix basin to be incorporated into the outer-loop system of freeways that encircle the Phoenix metropolitan area. Terracon Consultants, Inc. was on the Michael Baker, Inc. project team which was awarded the design contract for a three mile segment of the new freeway which encompasses the project limits of one-half mile south of Northern Avenue to one-half mile north of Peoria Avenue (see Figure No. 1). The project is approximately 3.0 miles in length and is generally located between Mileposts 109.65 and 112.65 in Maricopa County, Arizona.

Major elements of the project will include:

- Three new lanes in each direction along the mainline of the SR 303L;
- Four new bridge structures; one at each of the three cross roads and one at the traffic-interchange at the proposed Northern Avenue Parkway
- Numerous on- and off-ramps at the bridge structures
- Cross road improvements at:
    - Northern Avenue
    - Olive Avenue
    - Peoria Avenue
- A new retention basin located at the northwest corner of Northern Avenue and SR 303L that will be approximately 1,000 feet by 1,400 feet in plan area, with an approximate depth of 20 feet; and,
- A drainage channel that runs the entire length of the project on the west side of the mainline roadway.

FIGURE NO. 1: SITE LOCATION MAP

The roadway along the mainline will be supported on embankments varying from one foot to 31 feet in height. The entire roadway within this segment of the SR 303L will be raised above existing grades including the proposed ramps. The project will require importation of approximately 1.7 million cubic yards of earth to balance the grading plans.

At the beginning of the project there were seven identified or unconfirmed earth fissures mapped by Arizona Geological Society (AZGS) on the Earth Fissure Map of the Luke Study Area (AZGS, 2008) that either cross proposed roadway alignments or, are located within ¼-mile of the project (see Figure No. 2). Four of the seven earth fissures (Nos. 1, 2, 4, & 5) cross the roadway alignments at 11 locations within the project limits. Earth Fissure Nos. 3 & 6 do not cross any of the roadway alignments on the project. Earth Fissure No. 7 only crosses the west drainage channel.

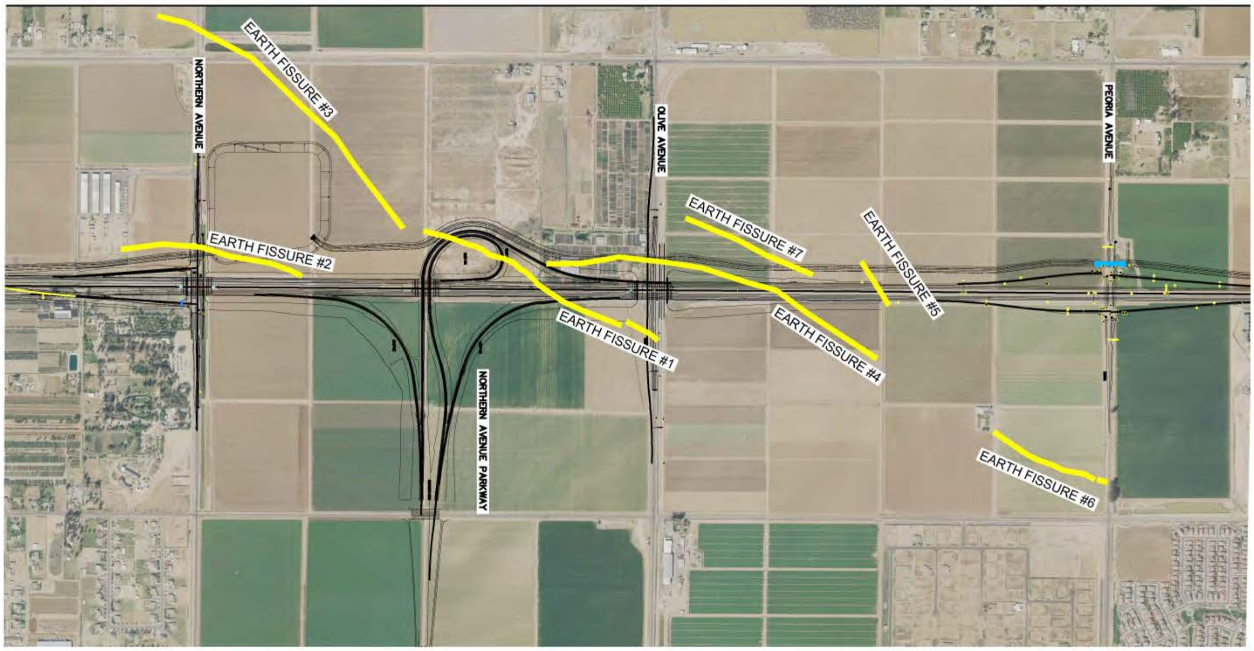


FIGURE NO. 2: EARTH FISSURE MAP

GEOLOGY

The project area is located within the Basin and Range Physiographic Province of the southwestern United States. More specifically, the SR 303L alignment is located within the western portion of the Phoenix basin. The Basin and Range Province is characterized by alternating northwest-southeast trending mountainous terrain of igneous, metamorphic and consolidated sedimentary rocks and broad alluvial valleys or basins, most formed by block faulting and folding. The White Tanks form the mountains and hills west of the current segment of the SR303L, and are aligned North-South, and are composed, in part, of Tertiary age rocks common in the Basin and Range province. The White Tank range also contains Proterozoic metamorphic and igneous rocks closely related to the Arizona Transition Zone between the Basin and Range Province and the Colorado Plateau Province. The western portion of the Phoenix basin is underlain by unconsolidated to semi-consolidated basin-fill alluvial sediments extending down over 12,000 feet below the ground surface (Eberly and Stanley, 1978).

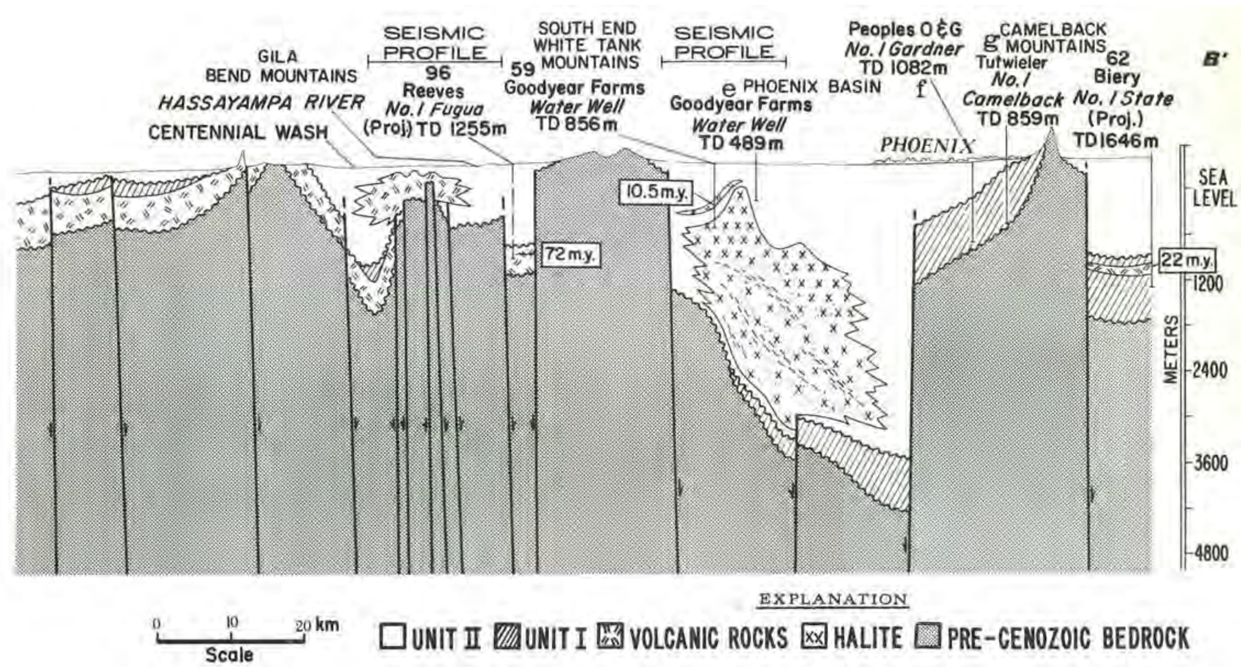


FIGURE NO. 3: GEOOGIC CROSS SECTION THROUGH S.W. ARIZONA INCLUDING THE PHOENIX BASIN

GROUNDWATER

Groundwater was not observed within the upper 110 feet of any boring drilled for the SR 303L project at the time of the field exploration nor when checked immediately upon completion of drilling. Based on information obtained from the Arizona Wells' database website (http://www.sahra.arizona.edu/wells), the depth to groundwater was measured in 2002 to be approximately 380 feet to 410 feet below the ground surface (approximate elevation of 760 feet to 780 feet above mean sea level) at an United States Geological Survey (USGS) monitored well site located less than one mile from the site of the project.

EARTH FISSURES

The AZGS Earth Fissure Map of the Luke Study Area (AZGS, 2008) shows earth fissures present within the project limits categorized as: 1) continuous fissures that are present as some form of observable ground deformation; 2) discontinuous fissures that are observed in the field and surveyed; and 3) "approximate locations of unconfirmed earth fissures" that are based on historic reporting, but were not able to be confirmed in the field by AZGS geologists. The portion of the map encompassing the project is shown in Figure No. 4, and shows four or five echelon fissures crossing the project alignment trending in a general southwest to northeast direction.

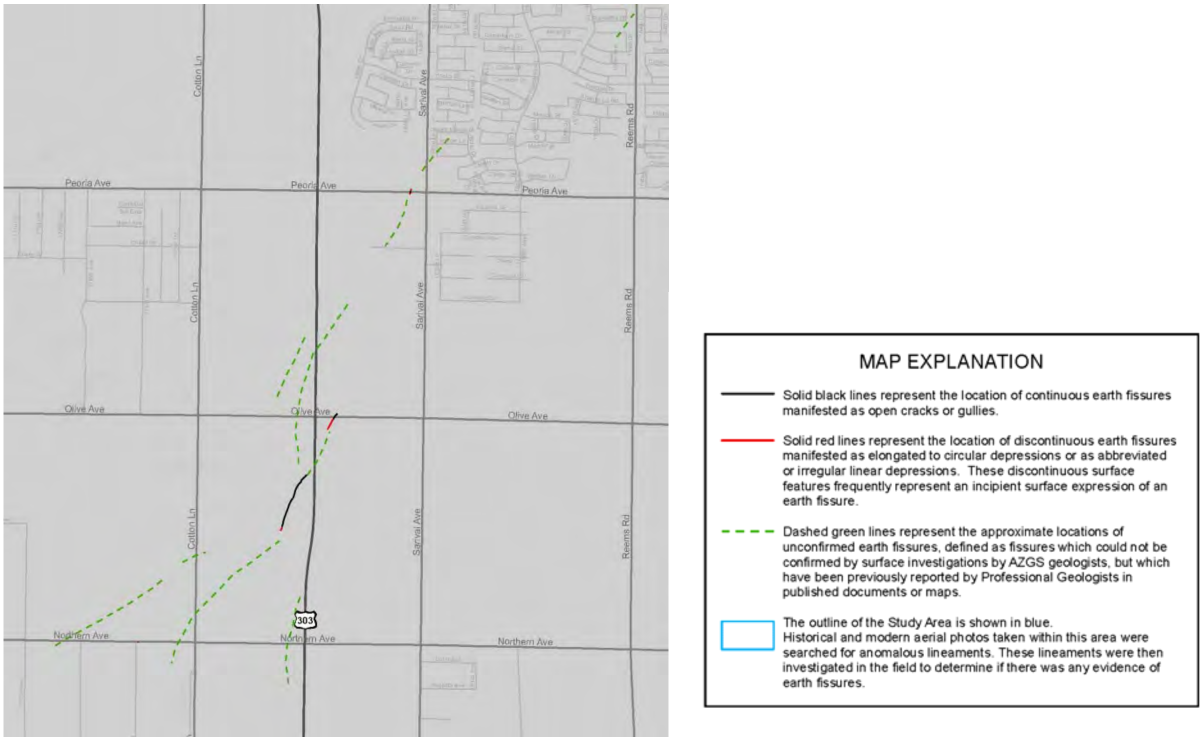


FIGURE NO. 4: AZGS LUKE STUDY AREA MAP OF KNOWN AND UNCONFIRMED EARTH FISSURES

Earth fissures in Arizona have resulted from ground subsidence generally associated with groundwater withdrawal from aquifers in sediment filled basins. The withdrawal of groundwater causes consolidation by the effective stress of the soils increasing from the buoyant unit weight of about 65 pcf, to a moist unit weight of about 130 pcf; an increase of about 100% as groundwater declines in elevation in the area. As an example of this phenomenon, when the groundwater is drawn down 100 feet, the subsurface soils beneath the elevation of groundwater withdrawal are subjected to an increase in pressure of about 100 feet times 65 pcf, or about 6,500 psf of additional in-situ pressure. Since the area being effected is on the order of square miles in plan area, the entire depth of subsurface soils down to bedrock, over 12,000 feet, undergoes a stress increase of 6,500 psf under this scenario. This stress increase causes the underlying materials to compress causing ground subsidence to occur. For the subsurface soils comprised of clay soils, the time for consolidation to take place may be several years or more depending on the length of the drainage path and the permeability of the saturated zone.

Subsidence has occurred in the west valley of the Phoenix Basin as basin sediments have compacted or consolidated from groundwater elevation declines. The decline in groundwater elevation is the result of groundwater pumping for agricultural and municipal uses that have outpaced the rate of aquifer recharge. As the groundwater elevation has decreased significantly, a reduction in volume of the sand and clay sediments through consolidation-type mechanisms has been manifested as subsidence at the ground surface followed by the development of earth fissures. The thought at the present is that there are generally two types of earth fissures. One type of earth fissure is essentially a tension crack caused by differential settlement across a local relatively incompressible subsurface high point. A typical subsurface high in the basin could be a buried mountain with surface geometry similar to what can be seen of the mountains exposed at the surface within the Phoenix Valley (see Figure No. 5).

A second type of earth fissure is caused by shear at the boundary of two significantly different compressible soils and/or materials with different rates of compression (i.e. the boundary between predominantly sand/gravel soils and silt/clay soils). Because one layer compresses significantly more than another, there is a considerable change in the amount of compression. If this change occurs over a short distance, the rate of deformation may be more than the subsurface soil can withstand and it undergoes shear. When this shear plane daylights at the ground surface, it is observed as a change in elevation (see Figure No. 6). This is the type of earth fissure observed on the site at the location of Earth Fissure No. 1 on the SR 303L alignment. These types of earth fissures typically have narrow apertures and do not tend to erode into large chasms.

As observed in our Earth Fissure Trench diagrams (Figure No. 13), there is a shear zone on both sides of the aperture opening in Earth Fissure No. 1. The shear zone extended for about 4 feet on both sides of the aperture opening.

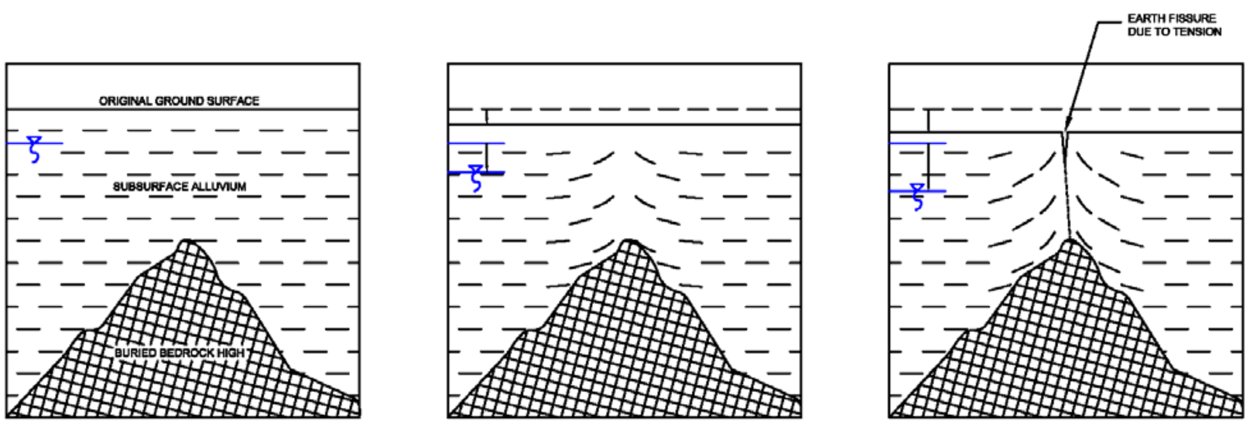


FIGURE NO. 5: SCHEMATIC OF EARTH FISSURE FORMATION DUE TO BURIED BEDROCK HIGH

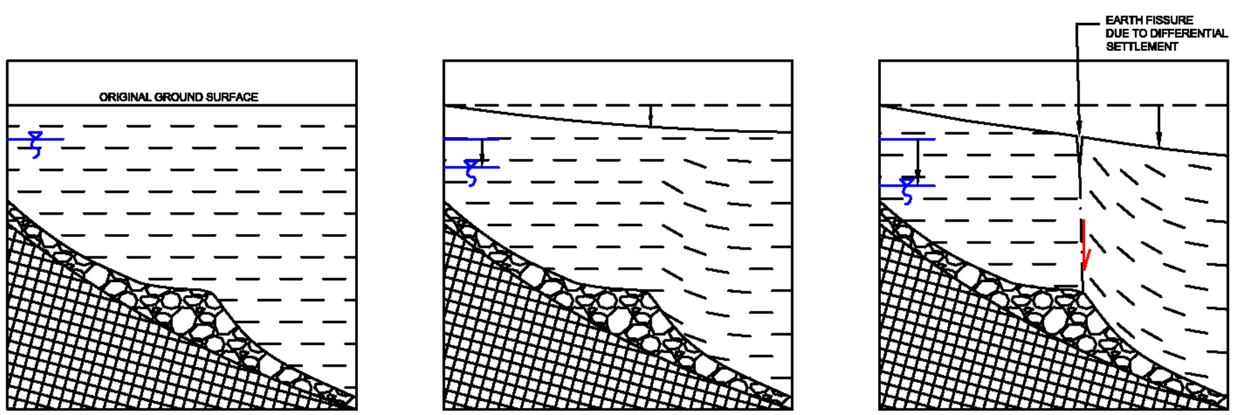


FIGURE NO. 6: SCHEMATIC OF EARTH FISSURE FORMATION DUE TO DIFFERENTIAL SETTLEMENT

PHOTO LINEAMENT ANALYSES

The lineament analyses consisted of reviewing historical aerial photographs for lineations which could indicate the presence of an earth fissure. The second step in the analyses was to determine the probable cause of each lineation found during the photo review. There were several different potential causes of the lineations observed on the aerial photographs, and they were as follows:

- Structural features within a geologic formation such as a fault, unconformity between two lithologies, or an earth fissure;
- Drainage features such as washes and streambeds that have a preferred orientation due to the underlying geological features or slope geometry;
- Animal trails, particularly cattle trails to and from water troughs or feed areas; and
- Man-made lines like roads, dirt paths, irrigation channels and earthen berms.

The lineations may be observed on the photographs as shadows if there is sufficient relief across the feature. Low sun angle photographs are particularly helpful identifying these types of lineations. Lines may also be caused by changes in the vegetative cover, either by a distinct change in the vegetation, or by the appearance of vegetation where there generally is none. Lastly, lines may be caused by a distinct change in the moisture content from one side of a line to the other, such as when an agricultural field is irrigated and the water runs into an earth fissure causing one side to be darker due to the irrigated water and the other side appearing to be dry because the water has run into the open earth fissure.

When trying to determine if a line is potentially caused by an earth fissure, the first objective is to eliminate those lines that have a high probability of being the result of other causes. There are several difficulties with a lineament study when trying to identify unconfirmed earth fissures, particularly when the earth fissures cross agricultural fields that are plowed on a regular basis, and aerial photographs are not taken on a routine schedule.

Aerial photographs dating as far back as 1967 were obtained during our study, and as recent as 2010. The following two aerial photographs (Figure No. 7) are provided to show the process of identifying lineations, and then to illustrate how some of the lineations are removed from consideration as potential earth fissures, due to other causes.

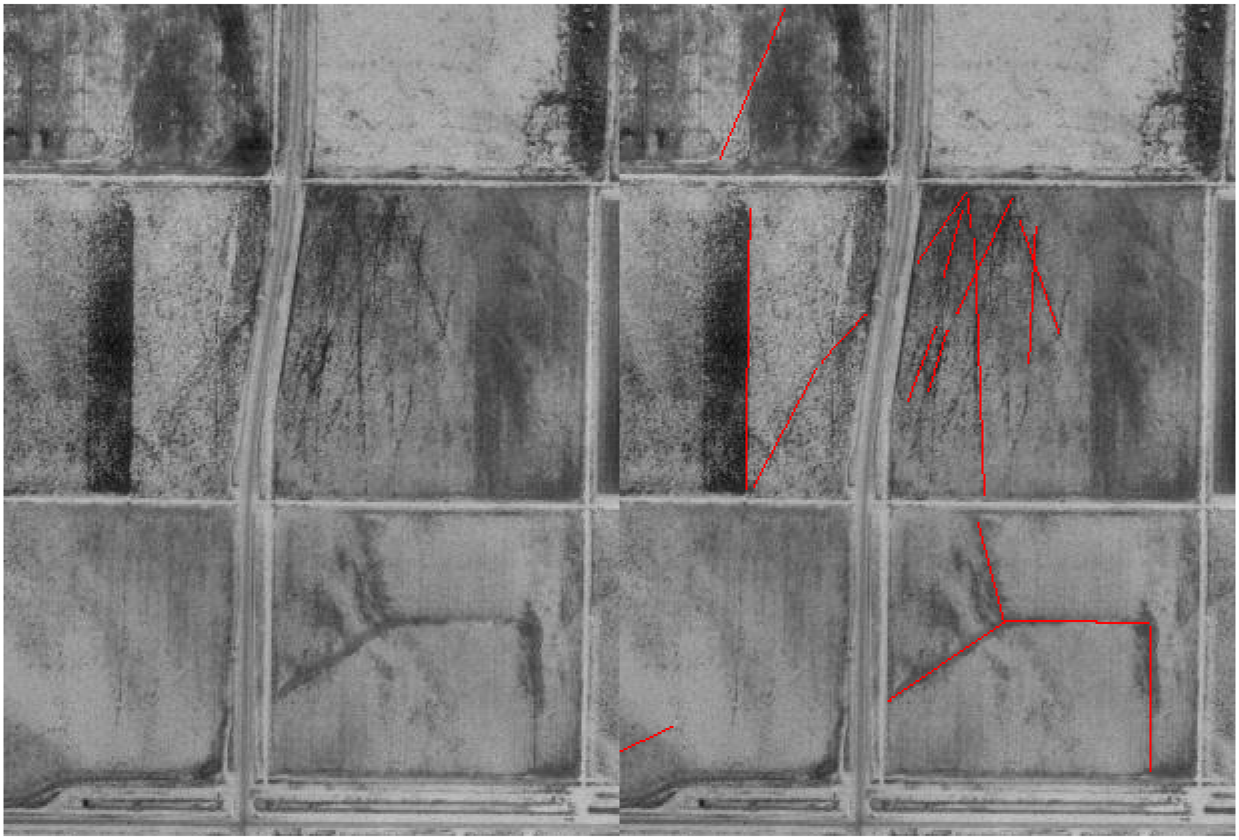


FIGURE NO. 7: AERIAL PHOTOGRAPHS OF AREA IMMEDIATELY NORTH OF NORTHERN AVENUE ALONG THE SR 303L ALIGNMENT. UNMARKED PHOTO ON LEFT, LINEATIONS MARKED IN PHOTO ON RIGHT

The red lines on the right photograph in Figure No. 7 are lineations identified during our study. These lineations were then compared against historical mapped earth fissures, likely drainage direction of local streams/washes, the overall geometry of the lines and site reconnaissance features observed in the field. As can been seen on the following photograph (Figure No. 8) only four lineations remain as location of potential earth fissures after eliminating the other lineations from consideration due to probable cause by other means.

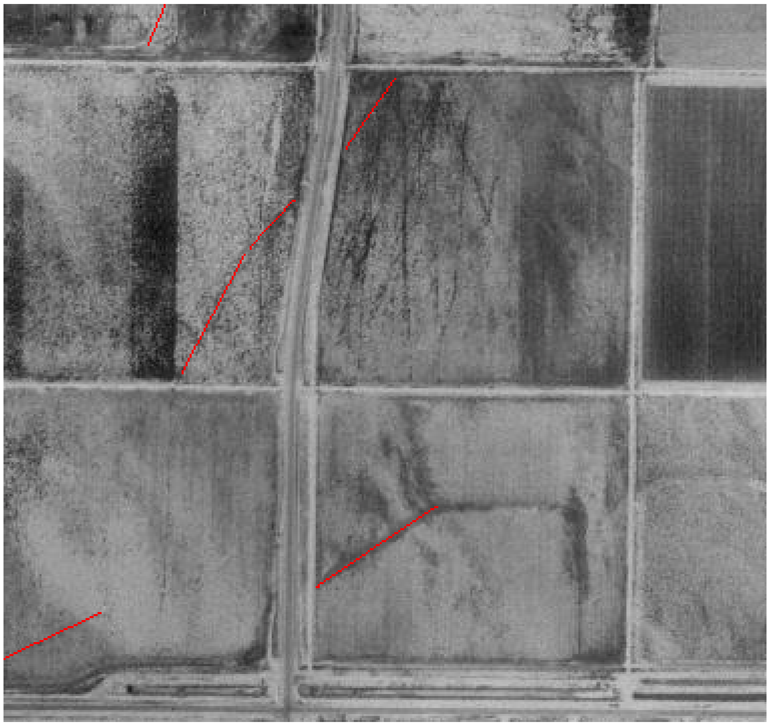


FIGURE NO. 8: AERIAL PHOTOGRAPH WITH UNCONFIRMED EARTH FISSURES MARKED

The following photographs taken in 2008 (Figure No. 9) show the location of Earth Fissure No. 1 which is the fissure known to exist within the project limits.

FIGURE NO. 9: UNMARKED AND MARKED AERIAL PHOTOGRAPHS OF EARTH FISSURE NO. 1

The advantage of reviewing photographs taken over several different years can be seen by comparing the photographs in Figure No. 9 taken in 2008 with those in Figure No. 10 taken in 2009. The continuation of Earth Fissure No. 1 to the east side of the SR 303L cannot be seen in 2008, but is noticeable in 2009 by the slight change in color in the plowed agricultural field just south of Olive Avenue.

FIGURE NO. 10: UNMARKED AND MARKED AERIAL PHOTOGRAPHS ONE YEAR LATER SHOWING COLOR CHANGE EAST OF THE SR 303L OF EARTH FISSURE NO. 1

SURVEY INFORMATION

Shumann (Shumann, 1995) reported the average rate of subsidence in the west valley of the Phoenix Basin was approximately 6 in/yr at the center of the subsidence from 1957 to 1992. Over this period of time from 1957 to 1992 the center of the subsidence has settled over 18 feet. Based on InSAR data on Arizona Department of Water Resources (ADWR) maps, the interpreted rate of west valley ground subsidence at present is approximately 0.4 in/yr. While the present rate of ground subsidence has significantly decreased from the average rate observed between 1957 and 1992, the long-term actual rate of subsidence will vary depending on the land use and future rate of groundwater withdrawal. The significant decrease in the rate of subsidence, in the opinion of Terracon, should correlate to a significantly reduced probability of forming new earth fissures within the project site at the SR 303L.

SITE RECONNAISSANCE

Field reconnaissance of the site was conducted in December 2010 for visual evidence of the presence and/or potential presence of earth fissures and related features. Several identifiers were used as indicators during the mapping of suspect features including:

- Apparent changes in ground surface grade of several feet;

- Distress and vertical changes in surface grade of the asphalt concrete pavement on roadways along known or suspected fissure traces;
- Distress, breaks and deflections of concrete lined irrigation channels;
- Increased concentrations of animal and insect burrows;
- Ground deflation as evidenced by depressions and concentrations of small surface sinks;
- Ground surface collapse under foot traffic; and,
- Alignment of vegetation along suspected lineaments.

Reconnaissance involved site observation for signs of distress at all existing linear features that were located within and immediately adjacent to the project alignment. Additionally, lineaments identified as part of the Photo Lineament Analysis were field checked and evaluated for conducting geophysical survey and possible trench exploration work.

GEOPHYSICAL SURVEYS: SEISMIC P-WAVE AND REMI FIELD INVESTIGATION AND ANALYSES

Refraction seismic surveys were performed at selected locations on the project using a 24-channel seismograph and an array of geophones at 10-foot spacings. The seismic surveys were completed to provide subsurface information to identify the absence or presence of subsurface anomalies consistent with earth fissures at Earth Fissure No. 1 and at the locations of other unconfirmed earth fissures. For this project 18 seismic lines were performed at 13 locations. At five of the 13 locations, two seismic lines performed in series with a slight overlap were conducted to increase the area of investigation and subsequently increase our chances to encounter the suspected earth fissure being investigated.

A sledgehammer was used as an energy source to collect compression wave (p-wave) data. Heavy truck traffic from the existing SR 303L, and other means were used to generate surface wave energy for refraction microtremor (ReMi) data for shallow one-dimensional vertical shear wave (s-wave) profiles. The ReMi was performed to assist the p-wave analyses in determining if there were any subsurface anomalies at each location.

The maximum depth of investigation for the p-wave analyses ranged from about 20 to 40 feet. The depth of investigation for the shear wave (s-wave) analyses was on the order of about 160 to greater than 240 feet. Velocity reversals, where lower-velocity materials underlie higher-velocity materials, would not be detected using the p-wave seismic refraction technique. Significant and relatively large-scale velocity reversals can be detected using shear waves obtained from the ReMi technique.

P-wave attenuation anomalies were not observed in any of the 18 seismic lines performed for this study with the exception of the line (Line 11) located at the northern extent of Earth Fissure No. 1. Even the p-wave traces performed across Earth Fissure No. 1 (Lines 1, 2 and 3) in the barren field near the Northern Avenue Parkway traffic interchange did not show any significant attenuation of signal across the fissure. However, all three lines did show a trough of lower velocity material at the location where the lines crossed the earth fissure. The lower velocities observed in these seismic lines agree well with sheared materials observed at the earth fissure in the earth fissure trench described later in this paper.

Geophysical results from Line 11 have provided both limited observable p-wave attenuation and the interpreted localized zone of lower p-wave velocity consistent with the interpretations at Lines 1, 2 and 3.

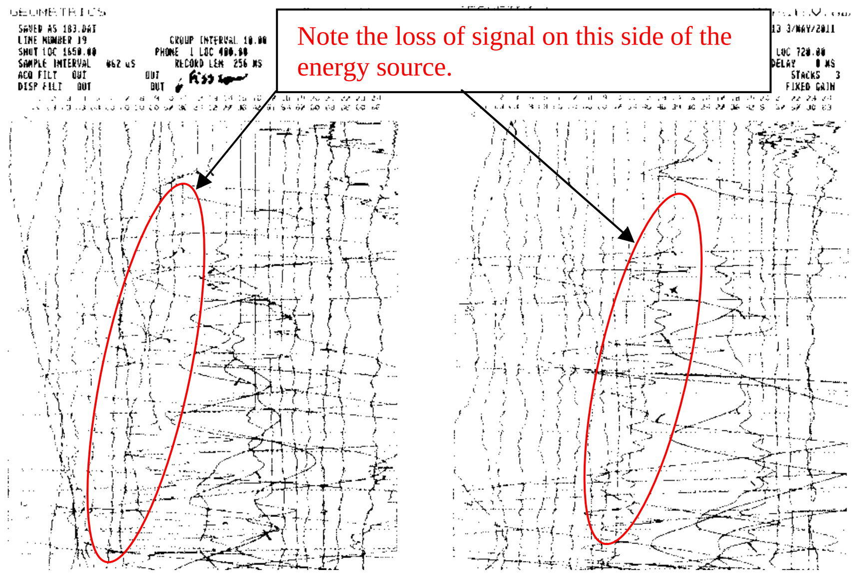


FIGURE NO. 11: AN EXAMPLE OF SIGNAL LOSS ACROSS AN EARTH FISSURE

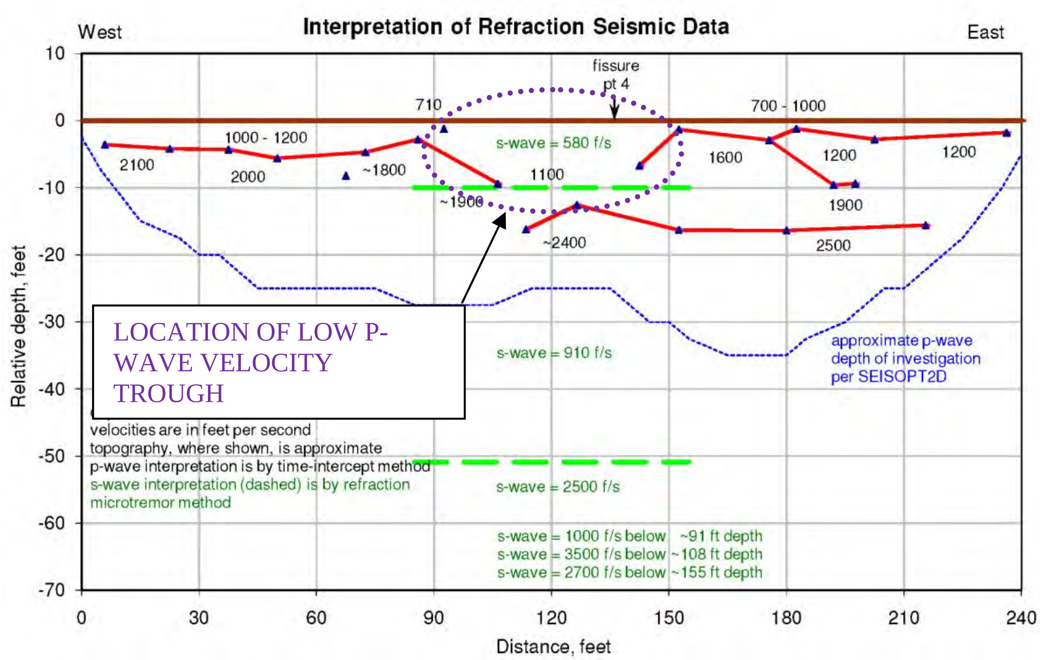


FIGURE NO. 12: AN EXAMPLE OF A LOW P-WAVE VELOCITY TROUGH AT EARTH FISSURE NO. 1

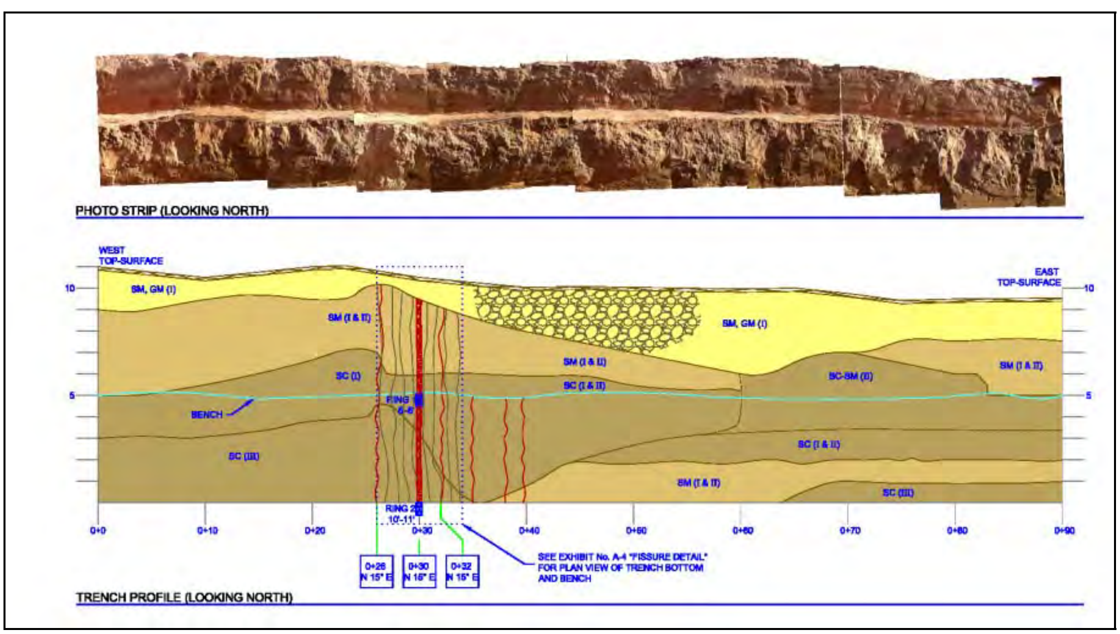
A lack of anomalous signal attenuation across Earth Fissure No. 1 is consistent with the observation of no open aperture at the earth fissure as exposed in the earth fissure trench. The zone of reduced p-wave velocity interpreted in Line 1 is consistent with the shattering / fracturing of the cemented soil structure in the immediate vicinity of the earth fissure.

EARTH FISSURE TRENCH FIELD INVESTIGATION

During the initial field reconnaissance, Terracon observed a number of documented and suspected earth fissure traces within the project area. One of the suspected earth fissures and one of the known earth fissures were selected for further exploration. Trench exploration was used to characterize and define the earth fissures within the subsurface soils at each site. The trench exploration was designed to define the extent of the earth fissures and obtain visual information regarding the type, and the possible mechanism that caused the earth fissure. Subsurface explorations were made along the trends of the known earth fissure (Fissure No. 1), and directly across one suspect unconfirmed earth fissure on the project site.

Each trench was excavated for a distance of 90 to 100 feet using either a CAT 345 B or a CAT 426C excavator equipped with a 48-inch wide smooth bottom bucket. The test trenches were excavated and benched to a width of 6 to 15 feet and a depth of 9 to 11 feet in an attempt to find any clear disturbances or anomalies that may be present or visible through the subgrade soils.

Trench logging required the careful cleaning of excess soil and removing smeared soil marks from the trench walls and floors that were caused by the excavator bucket. The cleaning was required to clearly expose the soil stratigraphy and discontinuities such as cracks or fissure features that might be present. The initial search was performed using a pick, shovel, brush, broom, and an air compressor. Trench sections that displayed evidence of earth fissuring were carefully and completely logged, described and photo-logged by a geologist. A finished trench log for the project is shown in Figure No. 13.



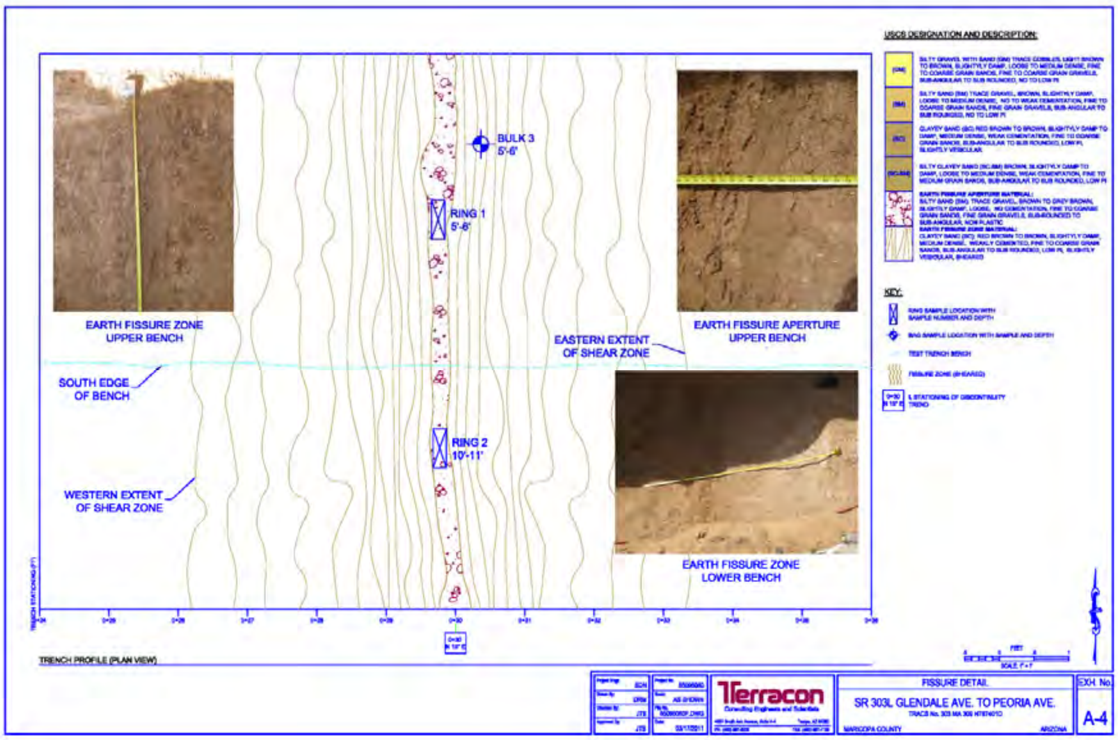


FIGURE NO. 13: DRAWING ON PREVIOUS PAGE IS AN EARTH FISSURE TRENCH LOG DRAWING ON THIS PAGE IS CLOSE-UP OF APERTURE AND SHEAR ZONE

RESULTS OF THE INVESTIGATIVE INFORMATION

The subsurface exploration program along with the field reconnaissance and aerial photograph interpretation defined the extent of Earth Fissure No. 1 located on the project site. Based on all reviews, photo interpretation and site reconnaissance the remaining unconfirmed earth fissures have not been considered for mitigation in the design and construction of the project as a result of the study.

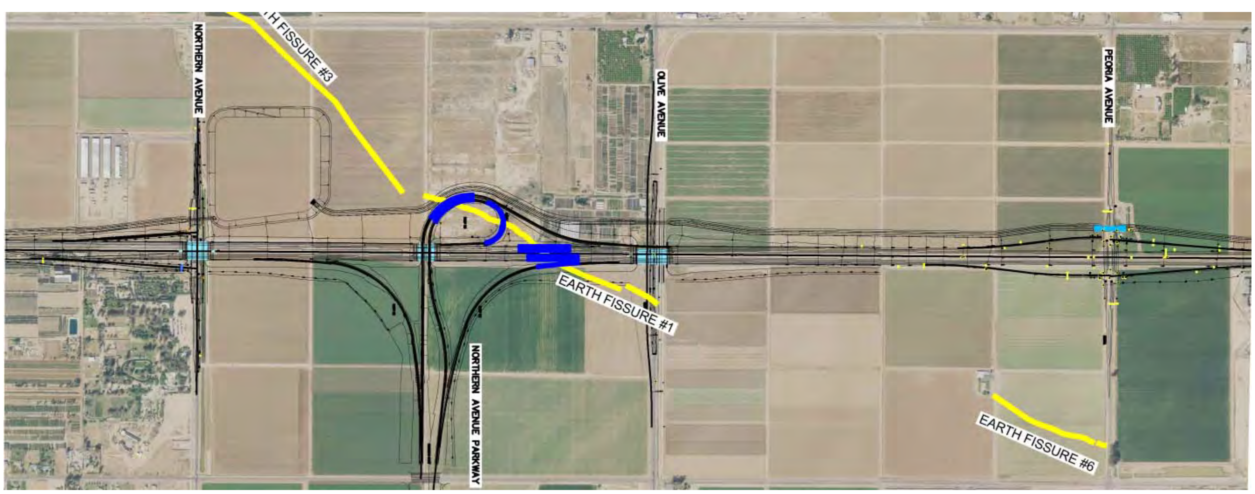


FIGURE NO. 14: PHOTOGRAPH OF APERTURE FILLED EARTH FISSURE OBSERVED IN EARTH FISSURE TRENCH AT FISSURE NO. 1

FINITE ELEMENT ANALYSES

The embankment at STA 52+00 along the Northern Avenue Parkway Ramp WS centerline was analyzed for propagation of the fissure within the embankment. The modeled displacement was 6-inches occurring over a distance of 7 feet below the embankment. The embankment backfill was assumed to be a medium dense granular soil with an elastic modulus (E_s) of 2.0 ksi and Poisson's ratio (ν) of 0.3. For purposes of modeling and earth fissure mitigation, a very stiff material having a relatively high modulus is required to distribute the stresses and strain over a broader area such that the differential movement occurring at the earth fissure beneath the embankment does not cause a safety hazard at the pavement surface. Terracon has designed relatively stiff earth embankments in the past using a geogrid mechanically stabilized aggregate layer (GMSAL). For this project we modeled a five foot thick zone of ADOT Class 2 aggregate base materials reinforced with Tensar Triax TX5 geogrid. This material was modeled having an E_s of 10 ksi (relatively 2 times stronger than unreinforced fill) and with a Poisson's ratio (ν) of 0.35. SIGMA/W was used to conduct the modeling of the embankment and earth fissure mitigation measures. The input and output cross sections from the analyses are shown in Figure No. 15.

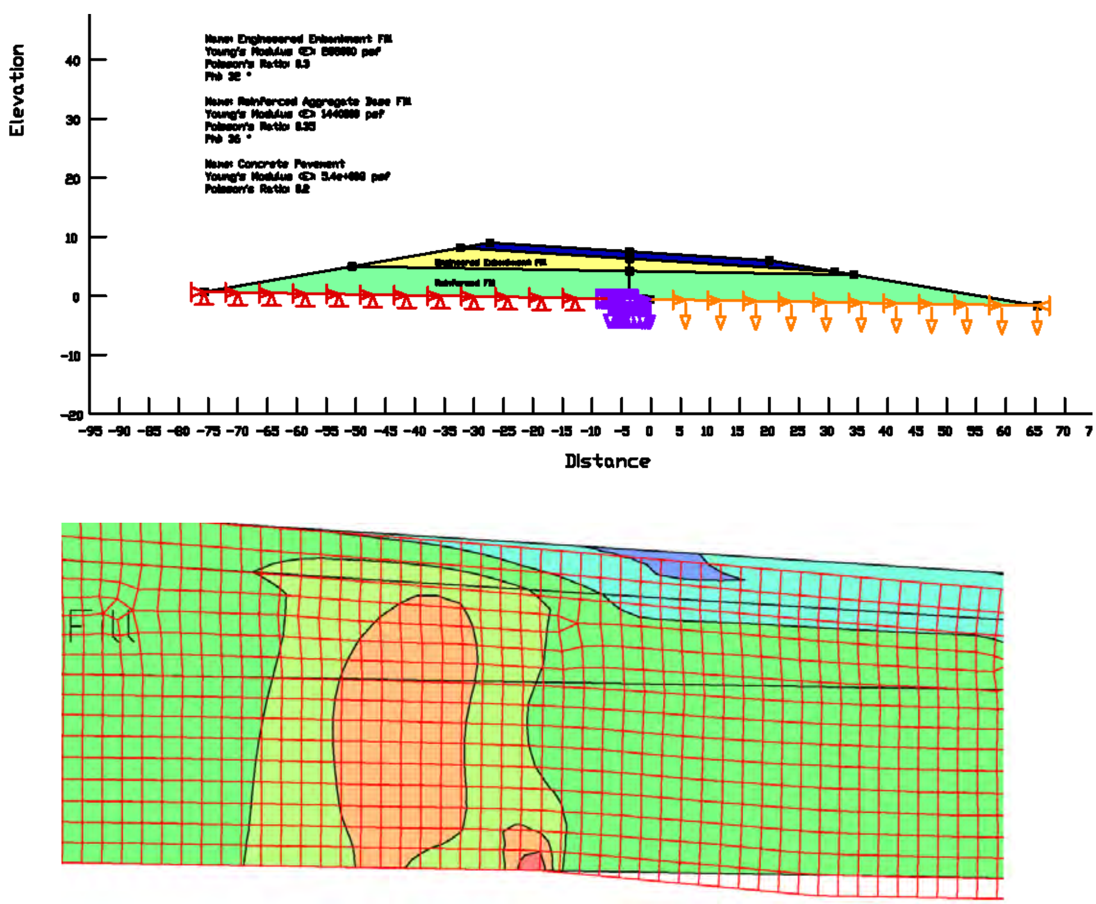


FIGURE NO. 15: TOP FIGURE IS INPUT CROSS SECTION OF ROADWAY AT EARTH FISSURE BOTTOM FIGURE IS CLOSE-UP OF OUTPUT AT OFFSET LOCATION SHOWING DEFORMED MESH AND STRESS CONTOURS WITHIN THE EMBANKMENT

MITIGATIVE MEASURES USED ON THE ROADWAY EMBANKMENT AND THE WEST DRAINAGE CHANNEL

At this time, there is no standard of practice for addressing the risk of existing fissures becoming active during the life of a proposed structure. In our opinion, it is prudent to assume there is an elevated risk that the existing fissure (Earth Fissure No. 1) could become active again for the SR 303L project. There are many factors that have been considered that impact the development of mitigation measures, many political and some geophysical, that are too numerous and outside the intent of this paper. In the writer's opinion, there are very real and substantial concerns that could lead to the future draw-down of groundwater in the area and impact to the new roadway.

For the roadway embankments with more than five feet of fill, we have recommended mitigation consist of a five foot thick GMSAL. The geogrid will be on one foot centers with the bottom layer at the bottom of the ADOT Class 2 aggregate base material and the top layer one foot below the top of the Class 2. The fill is to be compacted to 100 percent of maximum density (tested as per ASTM D698) and at a moisture content within ±2 percent of optimum. The embankment backfill placed above the reinforced Class 2 region is to consist of granular materials compacted to 95 percent of ASTM D698.

At the Northern Parkway ramp locations just west of the mainline, the embankment fill height is less than 5 feet, and therefore, the existing subgrade will be excavated to place the minimum five foot thick GMSAL.

The geogrid reinforcement will extent approximately 300 feet in both directions (ahead and behind) from the location of the earth fissure crossing. The 300 foot distance will be staggered either side of the centerline depending on the skew of the earth fissure to the roadway. The 300 foot length was selected as a conservative approach in consultation with ADOT to deal with variations that could exist in the subsurface profile and delineation of the earth fissures.

Where the west drainage channel will cross Earth Fissure No. 1, we have recommended the following mitigation measures to reduce the chance of forming fissure gulleys:

- A filtration barrier is to be installed in the subsurface to block sediment transport into the fissure. The filtration barrier is to consist of a two-element geosynthetic that will reduce the risk of formation of a fissure gulley by reducing the potential for sediment transport into the fissure. The bottom element of the barrier will be a layer of geogrid installed at the base of the excavation. The upper element of the barrier will be a needle-punched, nonwoven geotextile filter fabric with a minimum grab tensile strength of 120 lb/ft placed over the geogrid.
- The barrier will be placed at a minimum depth of ten (10) feet. The bottom of the excavation will extend along the drainage channel 300 feet either side of the earth fissure. The excavation sides will be benched with steps up to four feet in height, with a net slope no steeper than 1H:1V (horizontal to vertical). The barrier will extend laterally up the excavation side slopes to within five feet of the surface.

The excavations will be backfilled with engineered fill compacted to 95% of ASTM D698.

- In addition to the two-element barrier at the base of the excavation, at least two additional layers of the geogrid will be placed in the backfill. We have recommended the layers be placed at two-foot intervals during filling so the geogrid will be installed at depths of ten feet, eight feet, and six feet below grade. The geogrid at eight and six feet will extend laterally to the limits of the excavation, but not up the side-slopes. A typical cross section of the fissure mitigation excavation is presented in Figure No. 16.

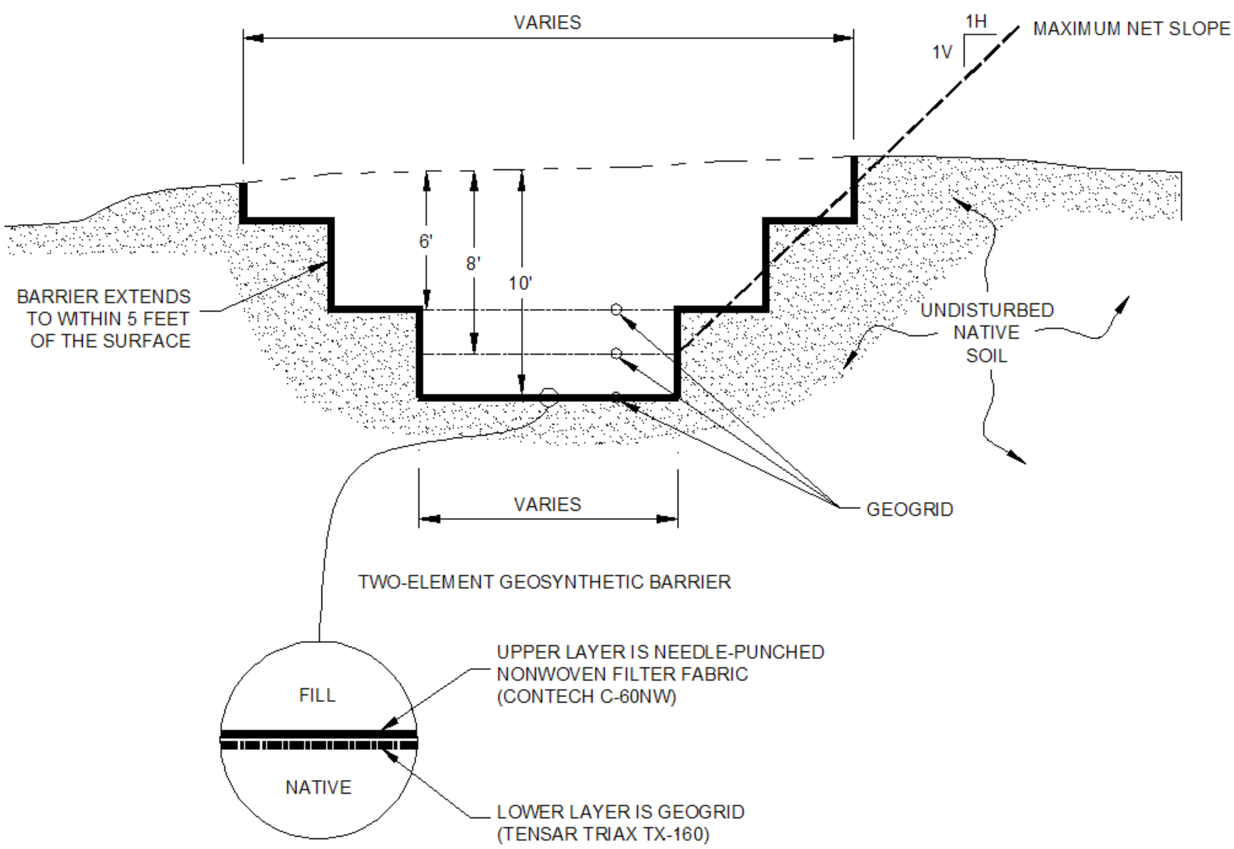


FIGURE NO. 16: MITIGATION EXCAVATION (TYPICAL – NTS)

Grading of the site will be completed to ensure that areas where water could pond do not extend to within fifty feet of the west drainage channel where it crosses the existing earth fissure.

REFERENCES:

1. Schumann, H. H. *Land Subsidence and Earth Fissure Hazards near Luke Air Force Base, Arizona;* K. R. Prince, D. L. Galloway, & S. A. Leake (Eds.), U.S. Geological Survey Subsidence Interest Group Conference, Edwards Air Force Base, Antelope Valley, California, November 18-19, 1992-abstracts and summary (pp. 18-21). Sacramento, CA: U.S. Geological Survey. (Open-File Report No. 94-432). 1995.
2. Rucker, M.L. and Fergason, K.C., *Characterizing Unsaturated Cemented Soil Profiles for Strength, Excavatability and Erodability using Surface Seismic Methods*, American Society of Civil Engineers, Unsaturated Soils??, Vol. 1, 2006.
3. *Earth Fissure Map of the Luke Study Area, Maricopa County, Arizona,* Digital Map Series – Earth Fissure Map 8 (DM-EF-8). Arizona Geological Survey, Tucson, Arizona. 2008
4. Spencer, J.E. and Rauzi, S.L., *Drill Holes in the Luke Salt Body Penetrate Underlying Fault,* Arizona Geological Society, Vol. 35, No.3, 2005.
5. Eberly, L. D., Stanley, T. B. *Cenozoic stratigraphy and geologic history of southwestern Arizona,* Geological Society of America Bulletin, v. 89, p. 921-940, June 1978.

Slope Stability Instrumentation and Web Monitoring on I-5 (Portland, OR)

Naia Suszek
Applied Geomechanics
15579 E. Hinsdale Circle
Centennial, CO 80110
(303)-229-5552
nsuszek@geomechanics.com

Jeff Keller
Applied Geomechanics
15579 E. Hinsdale Circle
Centennial, CO 80110
(303)-693-7833
jeff.keller@geomechanics.com

Stephen Hay
Oregon Department of Transportation
123 NW Flanders St.
Portland, OR 97209
(503)-731-8306
Stephen.HAY@odot.state.or.us

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The author(s) would like to thank the individuals/entities for their contributions in the work described:

Susan Ortiz – Oregon Department of Transportation

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

The I-5 SW Iowa Street Viaduct Bridge Replacement Project is located one mile south of downtown Portland, Oregon. The project is located on Interstate 5 (I-5) and will replace an existing bridge structure and construct six retaining walls to allow for detour traffic. This original segment of I-5 was constructed in the 1950's along an existing railroad grade through steep terrain where soils are predisposed to saturation induced instability. A number of historic landslides have also been mapped in the region; six significant landslides have taken place since the original highway construction in the 1950's and one major slide occurred in 1913 during railroad expansion. The mitigation measures included slope regrading, rock buttresses, horizontal drains, shear keys, and gravity retaining walls.

During project development, a geotechnical exploration program installed nine slope inclinometer tubes in identified slide areas. The inclinometer tubes were monitored with a traversing probe over a two year period. Fully grouted vibrating wire piezometers were also installed with the inclinometer tubes at predetermined depths. Based on historical slide rate data and inclinometer and piezometer readings, the Oregon Department of Transportation (ODOT) chose to implement an Automated Data Acquisition System (ADAS) for the duration of construction. The current ADAS system consists of six in-place inclinometer strings (installed in the original inclinometer tubes), the preexisting grouted-in-place piezometers, and a weather station. Inclinometers at each borehole are spaced 4 to 5 feet apart and span from the ground surface to competent bedrock. The complete ADAS provides near real-time slope stability information in a web based format for remote site monitoring. If a sensor exceeds certain user defined thresholds an e-mail, text message, and web-page notification are immediately sent to appropriate geotechnical staff for further analysis.

This paper will further explore the design elements behind the aforementioned Automated Data Acquisition System and report on system function and performance.

INTRODUCTION

The ultimate goal of the I-5 SW Iowa Street Viaduct Bridge Replacement project is to prepare a section of Interstate 5 (I-5) for bridge replacement. The preparations include replacing the current six lane bridge structure with a seven lane structure and construction of six retaining walls on both sides of the highway. This 0.85 mile section of highway (Figure 1), located one mile south of downtown Portland, Oregon, has experienced seven landslides since the expansion of the railroad in 1913.

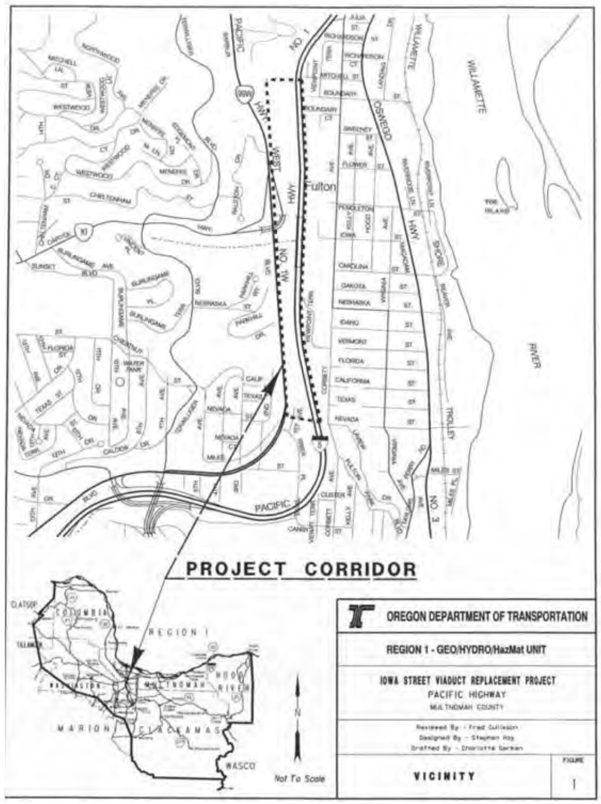


Figure 1 – Project Corridor

Retaining wall construction for the current project requires the removal of some previous landslide mitigations and excavation into historic landslide debris. A valid concern is that this excavation could reactivate a historic landslide or initiate a new slide. Ground movement could also potentially impact I-5 as well as OR99W (the designated I-5 detour route west of I-5) during construction, decreasing safety for the traveling public and construction personnel below slide areas. In order to monitor the slope during construction, an Automated Data Acquisition System (ADAS) has been installed. The ADAS consists of six strings of in-place inclinometers and ten grouted piezometers. The ADAS provides near real-time slope stability data in a web-based format and automatically notifies ODOT geotechnical staff of movement outside user-set thresholds.

PROJECT BACKGROUND

The goal of the I-5 SW Iowa Street Viaduct Bridge Replacement Project is to construct a new bridge on I-5 south of Portland. The final bridge will accommodate seven lanes, while the highway immediately north and south of the bridge will be widened to nine lanes for detouring of traffic during construction. Retaining walls will be constructed on both sides of the highway. Six lanes of traffic must be maintained throughout the project and, therefore, a temporary three lane detour structure will be constructed west of the proposed bridge. Three of the retaining walls will be constructed as tied-back soldier pile walls and the remaining walls will be Mechanically Stabilized Earth (MSE) Walls. The project is scheduled for completion in late 2013.

Geology

The site consists of unconsolidated alluvium overlying basalt (Columbia River Basalt Group). For the most part, the alluvium consists of silt, silty clay, silty sand, and sand. In several areas the alluvium appears as landslide debris. Throughout the area the basalt bedrock can be subdivided into three layers (CRB 1, CRB 2, and CRB 3). CRB 1 is moderately weathered to decomposed and is highly fractured with very close joint spacing; some joints are soil in-filled. CRB 2 is slightly to moderately weathered and highly fractured; the joint spacing varies from very close to moderately close. CRB 3 has similar properties to CRB 2 but contains abundant vesicles (Oregon Department of Transportation- Final Geotechnical Report).

Landslide History

The portion of I-5 in question was originally constructed in the 1950’s along an abandoned railroad grade built in the early 1900’s. Though the construction scope affects a section of highway less than one mile long, it has experienced seven notable landslides since 1900. Some of the identified slide areas were investigated and mitigated during highway construction. These mitigation measures included slope regrading, rock buttresses, horizontal drains, shear keys, and gravity retaining walls.

This first major recorded slide occurred about 1913 (see slide #6, Figure 2). A concrete gravity wall was used to mitigate and retain the slide; this wall was recently demolished as part of the current construction project.

Slides #1 and #4 (Figures 3 and 4) occurred during the original highway construction in 1958. Horizontal drains were installed in an attempt to stabilize the slope, but only yielded limited success. A rock buttress was eventually constructed in 1963. The buttress in front of Slide #4 was recently removed as part of the current construction project. Additional slides were documented by ODOT in 1972 (#5 in Figure 4, cutslope failure from oversaturated soils), 1980 (#2 in Figure 3– fillslope failure resulting from building construction below highway), 1993 (#3 in Figure 3– fillslope failure resulting from long term movement below the highway), and 1996 (#7 in Figure 2– cutslope failure from, oversaturated soils). The slides were relatively localized and mitigation ranged from a shear key drainage to a rock inlay (Oregon Department of Transportation- Final Geotechnical Report).

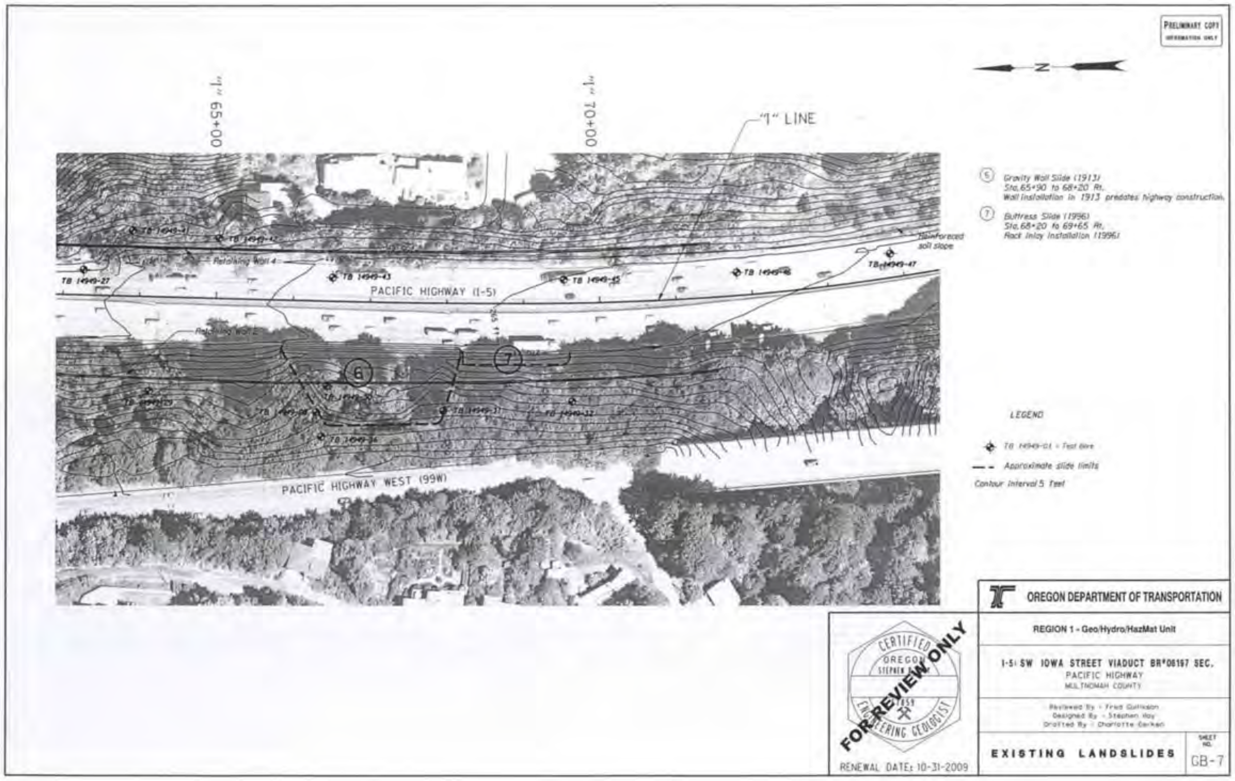


Figure 2 – Historic Landslides #6 and #7

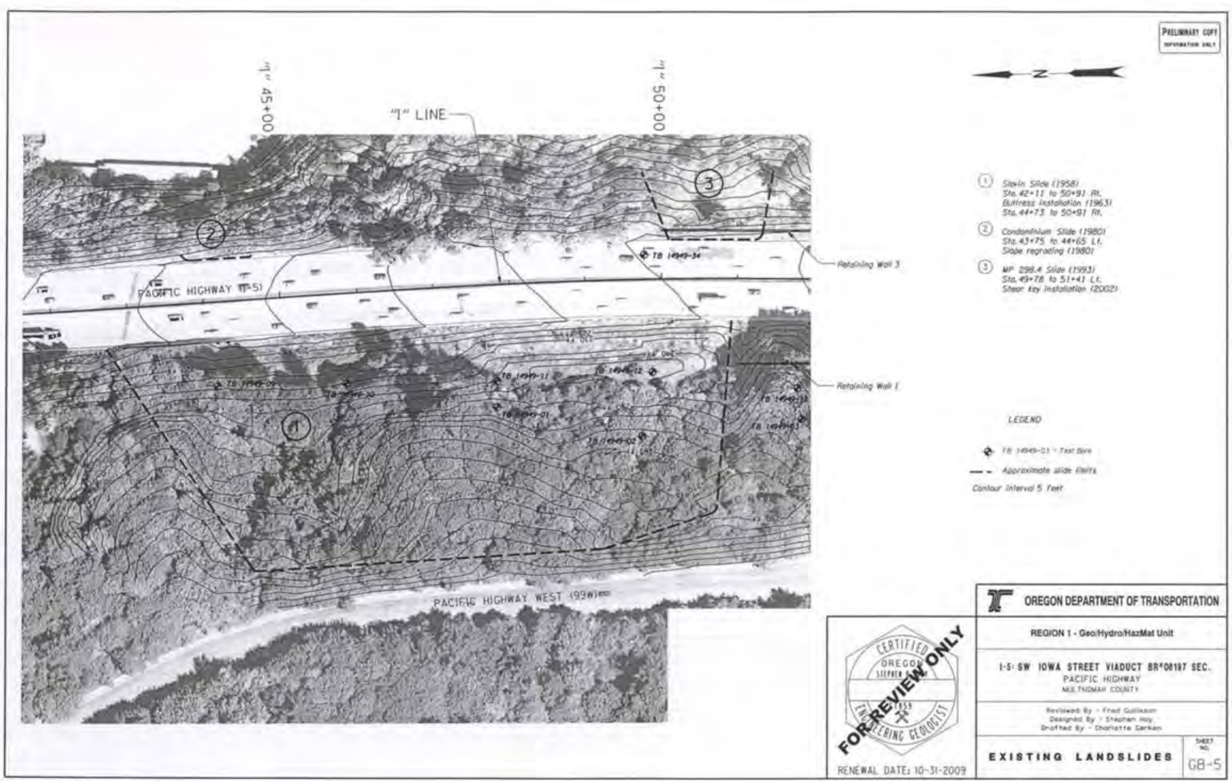


Figure 3 – Historic Landslides #1, #2, and #3

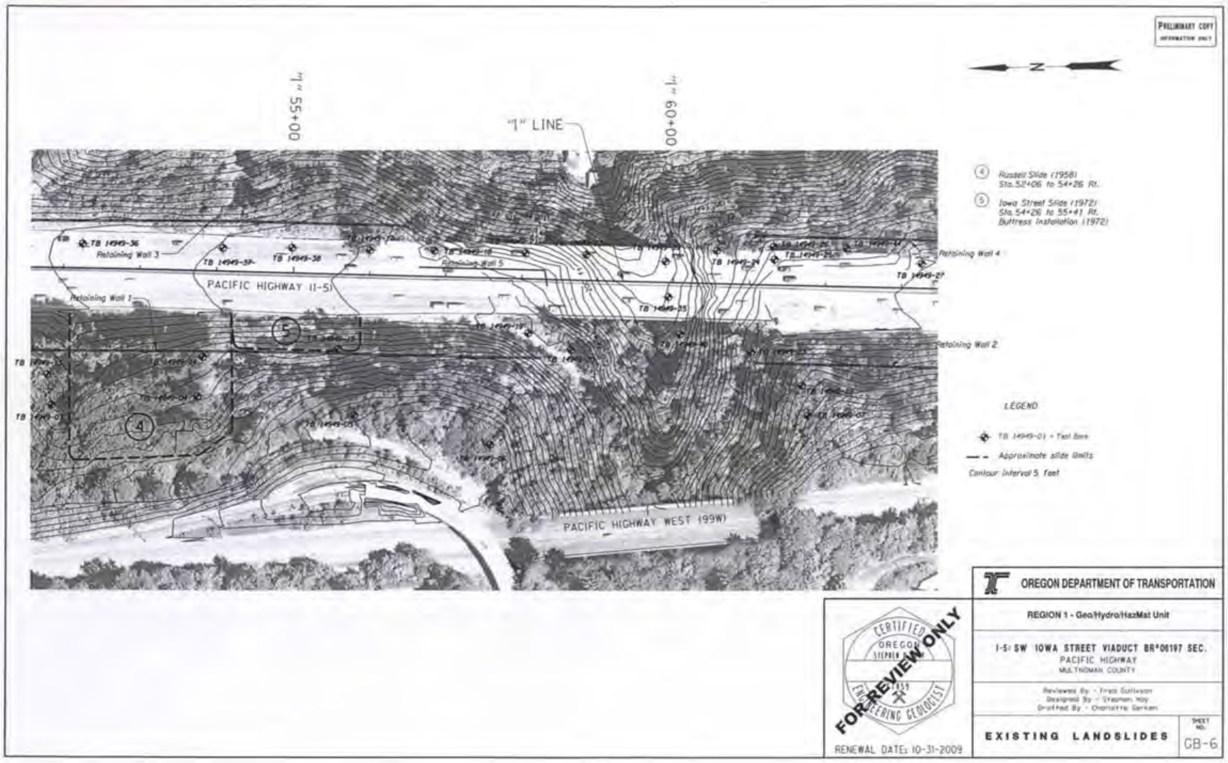


Figure 4 – Historic Landslides #4 and #5

These landslides will be retained with tied-back soldier pile retaining walls.

INSTRUMENTATION

Since the project site exhibits an active and recent landslide history, and because historic landslide mitigation measures are to be removed during the I-5 SW Iowa Street Viaduct Bridge Replacement Project construction, it was determined certain zones would need to be monitored throughout construction process. Due to the nature of the project, ODOT geotechnical staff determined the safest and most convenient monitoring program would be an Automated Data Acquisition System (ADAS). The current ADAS system notifies ODOT if sensors exceed certain user defined thresholds, and ODOT geotechnical staff has developed a procedure to follow for appropriate incident response. ODOT only plans to monitor the site with the ADAS during project construction. Once the retaining walls are completed, the slope should be stabilized. ODOT plans to remove the ADAS equipment at the completion of construction and re-use it on other projects throughout the state.

Sensors

Subsurface in-place inclinometers and piezometers were installed at six different locations. Due to high rainfall levels and the soil susceptibility to saturation and instability, a weather station was also installed.

Between November 2007 and April 2009 a total of 48 boreholes were drilled along the proposed retaining wall and at critical bridge locations. Inclinometer casing with attached vibrating wire peizometers were installed at nine locations. Manual inclinometer readings were taken at three month intervals from April 2009 until August 2010. Piezometer readings were recorded with single channel dataloggers. This manual monitoring took place throughout the project's development. Of the initial 48, six sites were identified by ODOT for ADAS installation. The ADAS upgrade included, in-place inclinometers (IPI), dataloggers for monitoring inclinometers and piezometers, cellular data modems, battery power supplies, and solar panels. Today this hardware, paired with Atlas web-based monitoring software, provides near real-time site data and acquisition via the internet.

In-Place Inclinometers

In-place inclinometers (IPIs) are bi-axial tilt sensors designed to be installed underground for detecting movement of slopes, deep excavations, and embankments. IPIs can be installed individually or in strings, and can be buried directly in the ground or, more commonly, installed in grooved inclinometer casing. Each IPI string provides a lateral displacement profile of the borehole in question. IPI strings are typically installed as permanent monitoring programs; however, as long as the casing remains vertical and does not shear, the sensor string can be removed and installed at a different location.

Because ODOT required a tightly spaced continuous borehole profile, the 906V-H Little Dipper IPI (manufactured by Applied Geomechanics) was selected. IPI sensors were strung together and spaced either 4 or 5 feet apart from the surface to a depth just below rock and as deep as 60 feet.

In a multiple interval installation, Little Dipper IPIs are connected by lengths of fiberglass rods equal to desired monitoring intervals. A Universal Pivot is also attached to the bottom of each Little Dipper to isolate individual sensor tilt measurements for each IPI from the sensors above and below each monitoring interval. The Universal Pivot has fins which slide through inclinometer casing groves and couple the pivot to the walls of the borehole; fins are not required for the Little Dippers (Figure 5).

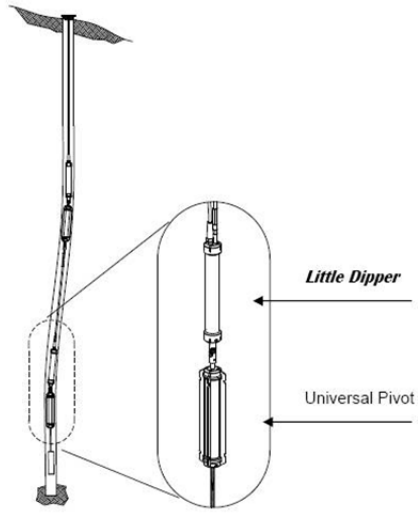


Figure 5 –Little Dipper IPI Sensor String

When the inclinometer string is fully installed, lateral casing movement displaces the Universal Pivots, tilting the sensor. Displacement (d) can be calculated with the following equation:

$$d = L \sin \theta$$

, where L is the monitoring interval and θ is the tilt angle.

In order for the system to accurately monitor tilt, each IPI string must be in tension. The Little Dipper IPI system accomplishes this via several brass weights, which are normally installed at the bottom of the string. The string is suspended by a cap which rests at the top of the casing.

The Little Dipper IPI is a bi-axial sensor, measuring tilt along two orthogonal tilt axes. In this paper, these two axes will be referred to as the x- and y-axis. The sensors were installed with the positive x-axis pointing downslope so that in the presence of a slide each IPI would register an increased tilt in the positive x-axis direction. The y-axis was installed parallel to the embankment slope.

The IPIs at each of the six sites are spaced at four and five foot intervals with the shallowest sensor depths being 4-feet and the deepest being 60-feet. The maximum number of sensors per string is 12, the highest number recommended for the 2.75 inch (70mm) diameter casing. ODOT plans to remove the IPI sensors at the project's completion. As mentioned earlier, the ADAS for the six sites will replace the manual borehole surveying program utilized through August 2010. The remaining three sites will continue to be monitored via a manually operated traversing probe every few months.

Piezometers

Vibrating wire piezometers were installed and grouted in-place when the inclinometer casing was installed in 2007 and 2008. Vibrating wire piezometers were selected because of their durability, stability, and accuracy. Some stations have only one piezometer and some stations have two. Piezometers measure fluid pressures induced by ground water elevations and pore pressures. The specific piezometers installed on the project are designed for low pressure ranges and have a built in thermistor to measure temperature. Historically, borehole piezometers have been installed by surrounding the sensor with sand and sealing the borehole with a bentonite cap. This installation process is difficult, has a high failure rate, and only allows for a single sensor, restricting monitoring to a single elevation per hole.

The ODOT piezometers were installed using the Fully Grouted method, made popular by Mikkelsen in the 1990s (Mikkelsen and Green). This method accommodates easy installation with multiple piezometers per borehole, while allowing other sensors to be installed within the same borehole.

During the ODOT project, piezometers were installed by attaching the sensors to the outside of the inclinometer casing as it was inserted in the ground. Once the casing was fully placed, the outside of the casing was grouted. The grout mix was specially formulated for the piezometers (Mikkelsen and Green). The bottom of the casing was capped prior to placement so that grout would not obstruct the casing's internal grooves.

Until the installation of the ADAS system, the piezometers were monitored with single channel dataloggers. Piezometer data was periodically downloaded by disconnecting the loggers from the piezometers and taking them to an offsite office where the data was uploaded from the dataloggers to a computer. Loggers were returned to the field after each data download.

ADAS Hardware

IPI and piezometer cables are fed into a steel enclosure containing the data logging equipment: datalogger, multiplexor, vibrating wire readout, cellular data modem, charge controller, and battery. The entire system is powered by a 60 watt monocrystalline solar panel. Monocrystalline panels were chosen specifically over their less expensive and more common relatives: polycrystalline panels. Monocrystalline panels charge more efficiently than their polycrystalline counterparts, and because the project site experiences notoriously rainy winters, often enduring weeks without sun, this made them an obvious choice for the ODOT project. The solar panel is regulated by a charge controller that charges a sealed rechargeable 84A/hr battery. Typically ADAS systems of this configuration run off 7 A/hr batteries; a larger battery was selected because of the cloudy Pacific Northwest winters.

The brain of the ADAS is the Campbell Scientific CR1000 datalogger. The CR1000 is programmable and not only collects samples from the IPIs and piezometers, but monitors battery health and controls how the sensors are powered for optimal battery life.

The logger collects samples from the piezometers via a Campbell Scientific AVW200, a vibrating wire spectrum analyzer. The spectral analyzer excites vibrating wire sensors and takes readings. The analyzer also performs frequency analysis on the readings to separate the instrument signal from noise. Without some sort of analyzer, it is difficult to take reliable reading from vibrating wire sensors. The AVW200 is a two-channel device, though it can also accept multiplexor inputs. Since the ODOT project uses only a maximum of two piezometers per ADAS site, multiplexors were not necessary.

Each ADAS site contains 7 to 12 IPIs. Since the datalogger cannot accommodate this many channels, a relay multiplexor is used at each site.

IPI, piezometer, and battery data is collected by and stored on the datalogger at 15 minute intervals. Users interface ADAS data using Atlas web-based monitoring software. Once an hour, data is transferred through a Raven X cellular data modem to the secure Atlas server. A Raven X with a dedicated cell account and IP address is located at each ADAS site. The Raven X has both serial and Ethernet capabilities but only uses Ethernet in this project. The Raven X communicates with the CR1000 through an Ethernet interface which also expands the logger’s memory.

All the ADAS sites are visible from the freeway. The area is also home to a population of transients. There was concern about the equipment, enclosures, solar panels, and sensor cables being vandalized. For this reason the enclosures and solar panels were installed atop 4 inch diameter galvanized poles at least seven feet in the air. Tamperproof bolts were used to mount the solar panels and enclosures; the bolts require a special bit to remove. Steel casings were placed on small concrete pads to protect the casing stuck up from the ground (Figure 6).

Figure 6 –Typical Site

Padlocks were placed on the datalogger enclosures, solar panels, and on casing enclosures. IPI and piezometer cables were fed through an electrical conduit and buried. This conduit helps not only to protect the cables from vandals, but also from the surrounding fauna and forest animals.

Web-Based Atlas Software

Data from the ADAS is collected by Atlas, a customizable web-based monitoring software system. Atlas automatically collects field data, processes readings, generates reports and graphs, and checks for alarm conditions. ODOT personnel can access ADAS data at any time or on any computer through a web browser.

Users can set up to four thresholds for each sensor; when a threshold is exceeded, a message is sent to a user specified distribution group. This message can be distributed via e-mail, pager, or text message. All alarms are cataloged in Atlas. ODOT geotechnical staff can also log into the system and add comments about specific alarms and their causes. All plots included in this paper were generated by Atlas. Thresholds appear on the plots as straight amber and red lines. Initially the thresholds were set at +/- 0.2 inch and +/- 0.5 inch, though thresholds have been increased at some sites to accommodate for changing event conditions.

The main ADAS Atlas page is represented in Figure 7. Each shape represents a different sensor (square-battery and logger temperature, circle-piezometer, triangle-IPI). Each square marks the approximate location of each installation site; sites are located from right to left, respectively as TB-02, TB-03, TB-04, TB-05, TB-07, and TB-08. A plot appears when a user clicks on a specific sensor; the sensor's color changes when a designated threshold is exceeded.

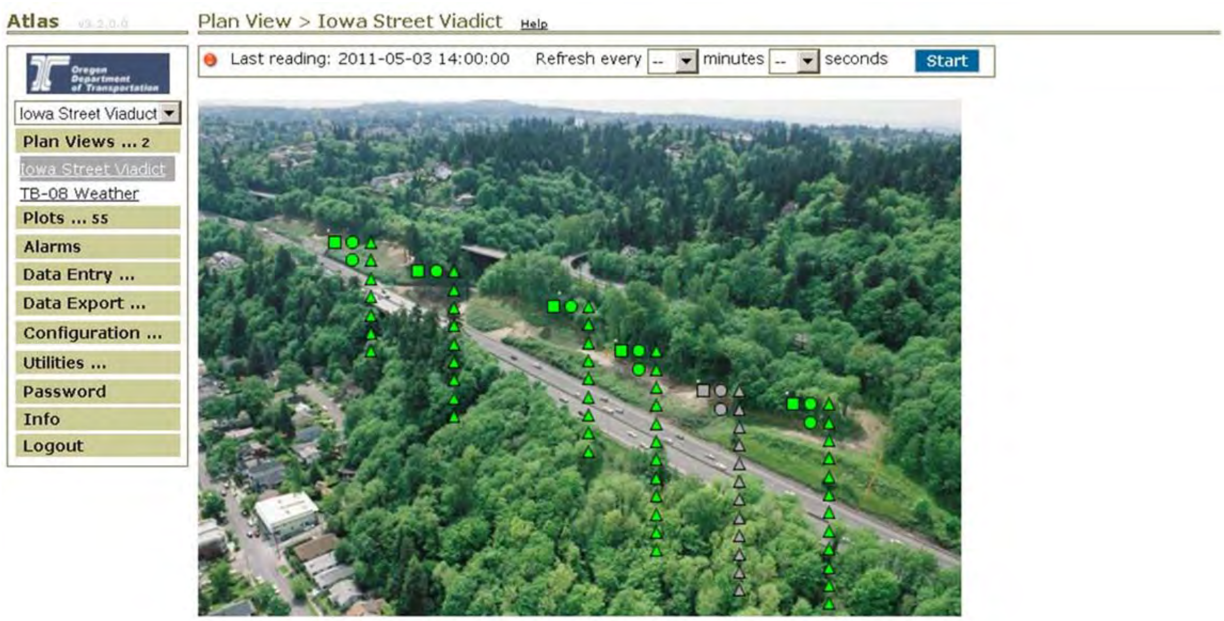


Figure 7 –Atlas Web-Based Monitoring Software

Weather Station

Portland, Oregon experiences high levels of rainfall on a yearly basis. Since excessive rainfall can cause ground instabilities, and because erosion control measures change at specified precipitation rates, a weather station was installed at one of the ADAS sites (TB-08) to track environmental conditions. The current weather station includes a tipping bucket rain gauge, wind vain and anemometer, ambient temperature probe, barometer, and soil moisture sensor. Data is collected and transmitted to Atlas from all the sensors by an on-site datalogger: the same logger that reads the site's IPIs and piezometers. ODOT geotechnical staff and construction workers automatically receive daily and monthly rain reports generated by Atlas.

DATA

At the time this paper is submitted, the ADAS system has been installed and recording data for about seven months. As of this paper's submittal no major ground movement has occurred. This is an example of the data (Figure 8).

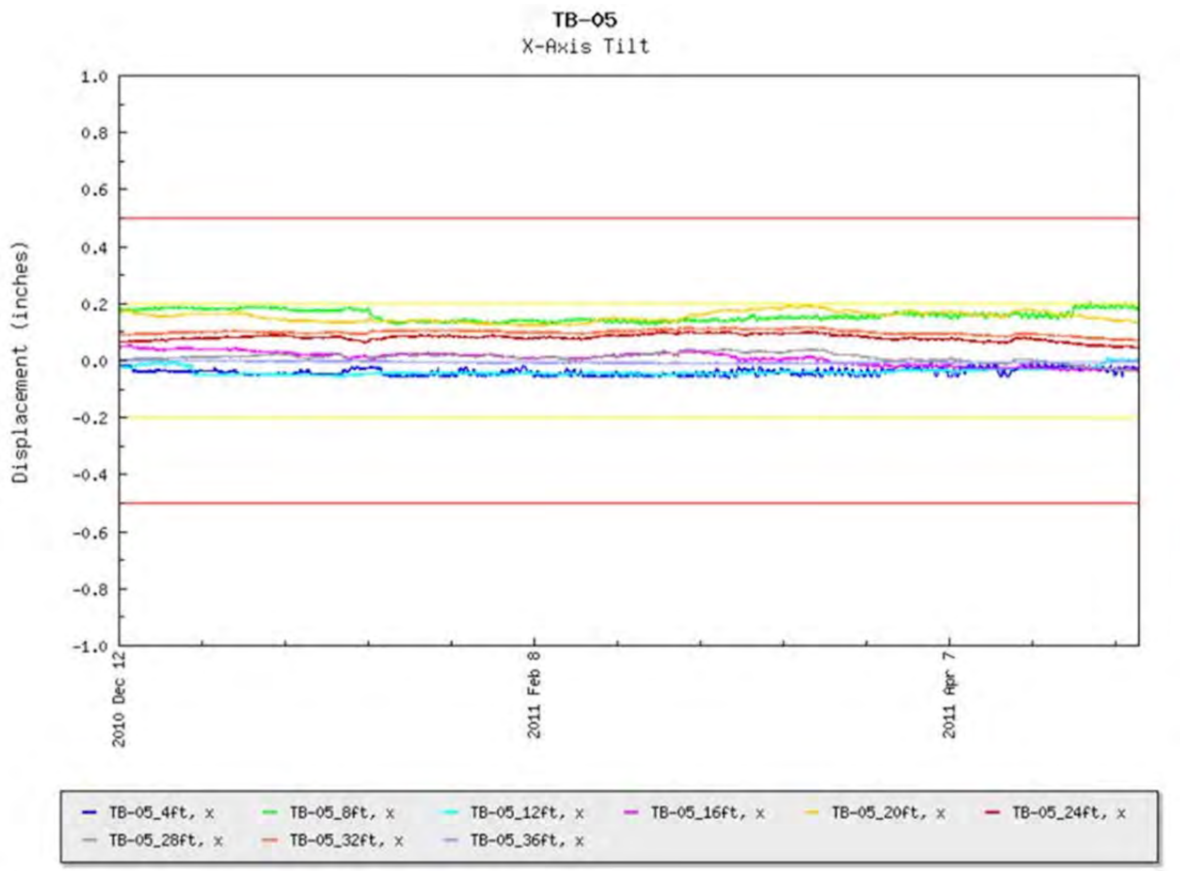


Figure 8 –Site TB-05, X-Axis

Clay Layer Slipping

In March 2010, sites TB-02 (Figure 9) and TB-03 (Figure 10) experienced some movement between 20 and 30 feet depths; the movement was less than 0.2 inches. A thin clay layer at a depth of approximately 30 feet was identified during borehole drilling for TB-03.

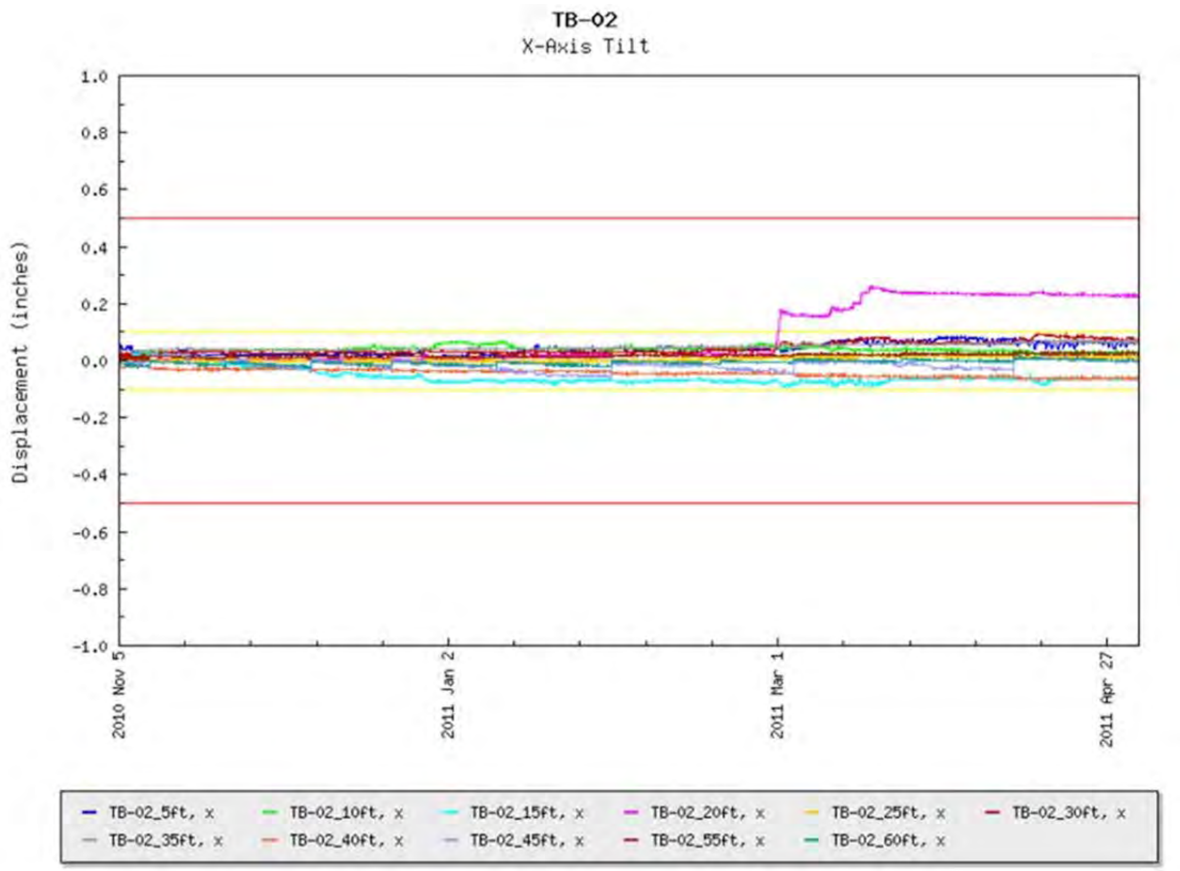


Figure 9 –Site TB-02, X-Axis

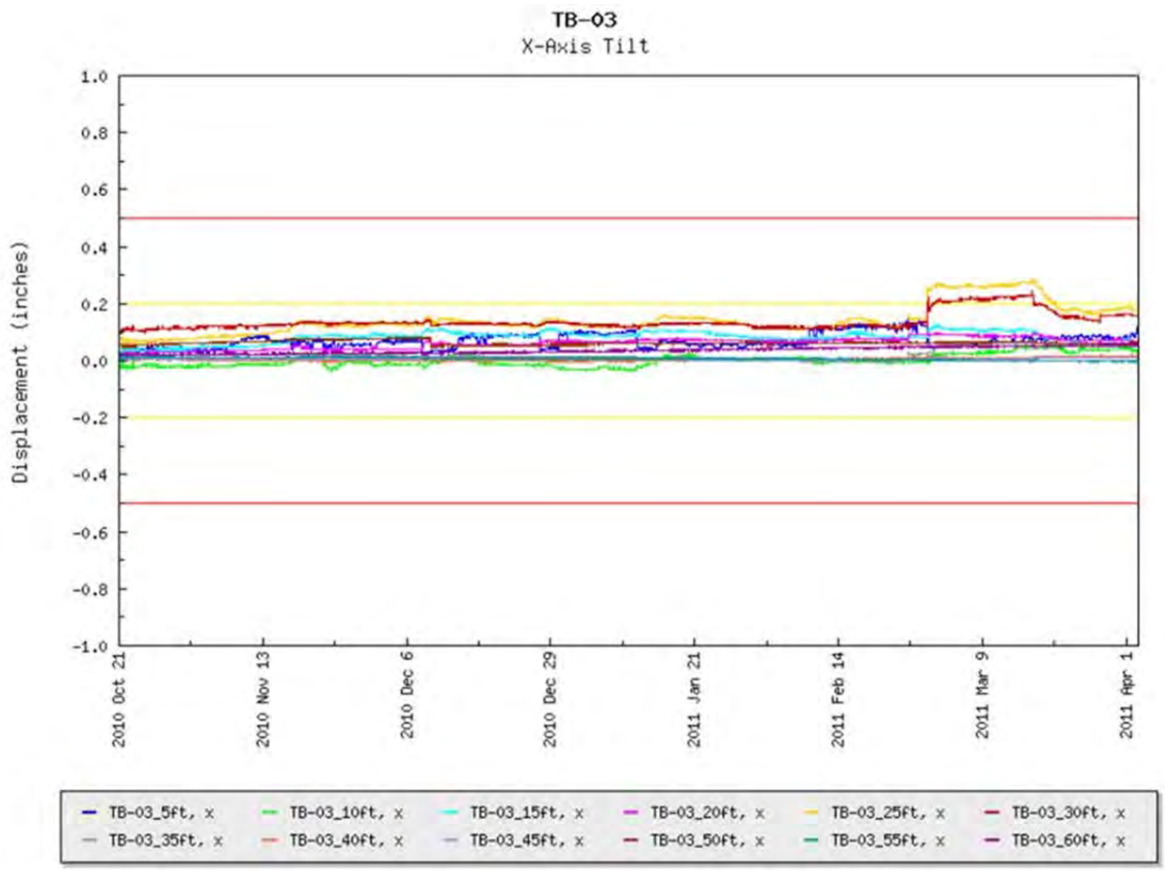


Figure 10 –Site TB-03, X-Axis

Thermal Variation of Shallow Sensors

The shallowest sensors are at 4 feet at sites TB-07 and TB-08; because these IPI's are so close to the surface, they are slightly noisier than other sensors in the system. This is most likely due to surface noise, most notably, thermal variation (Figure 11).

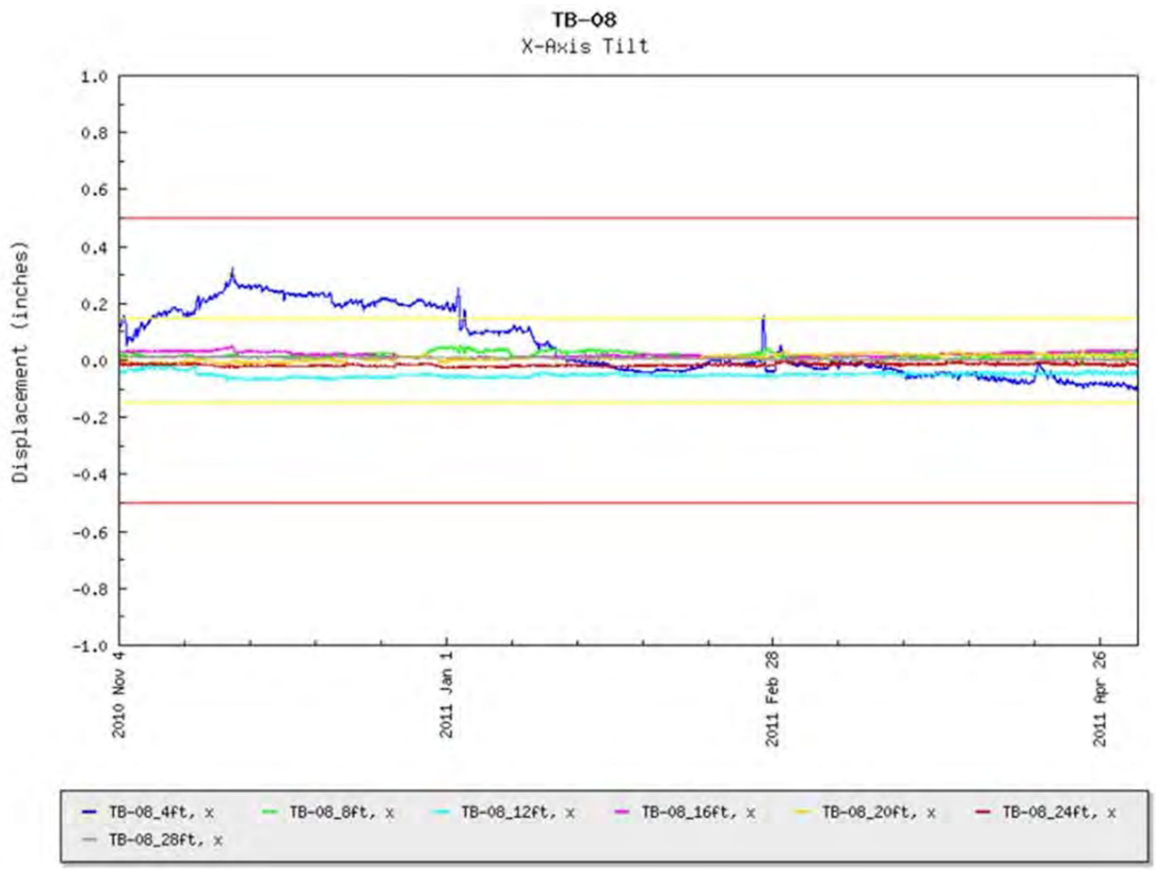


Figure 11 –Site TB-08, X-Axis

Movement Induced by Tie-Back Anchor Installation

Inclinometer casing was installed from November 2007 to April 2008 and considered part of the site exploration and evaluation phase of the project. At that time a tie-back retaining wall type had been identified, but the location of the soldier piles had not yet been finalized. The final design called for a pile spacing of 6 feet 8 inches on center. During the first phase of construction, in September 2010, it was realized by ODOT geotechnical staff that a possibility existed that a tie-back anchor could penetrate the slope inclinometer casing and impact the ADAS sensors at TB-03, -04, -07, and -08.

The first close call was at site TB-07 in November 4, 2010. Fortunately, the anchor drilling and grouting came close enough to the casing to deform it, but it was not punctured. The piezometers, mounted to the outside of the casing also appear to have survived. Another close call was experienced on March 7, 2011 at TB-04. All of the tiltmeters registered some sort of movement (Figure 12).

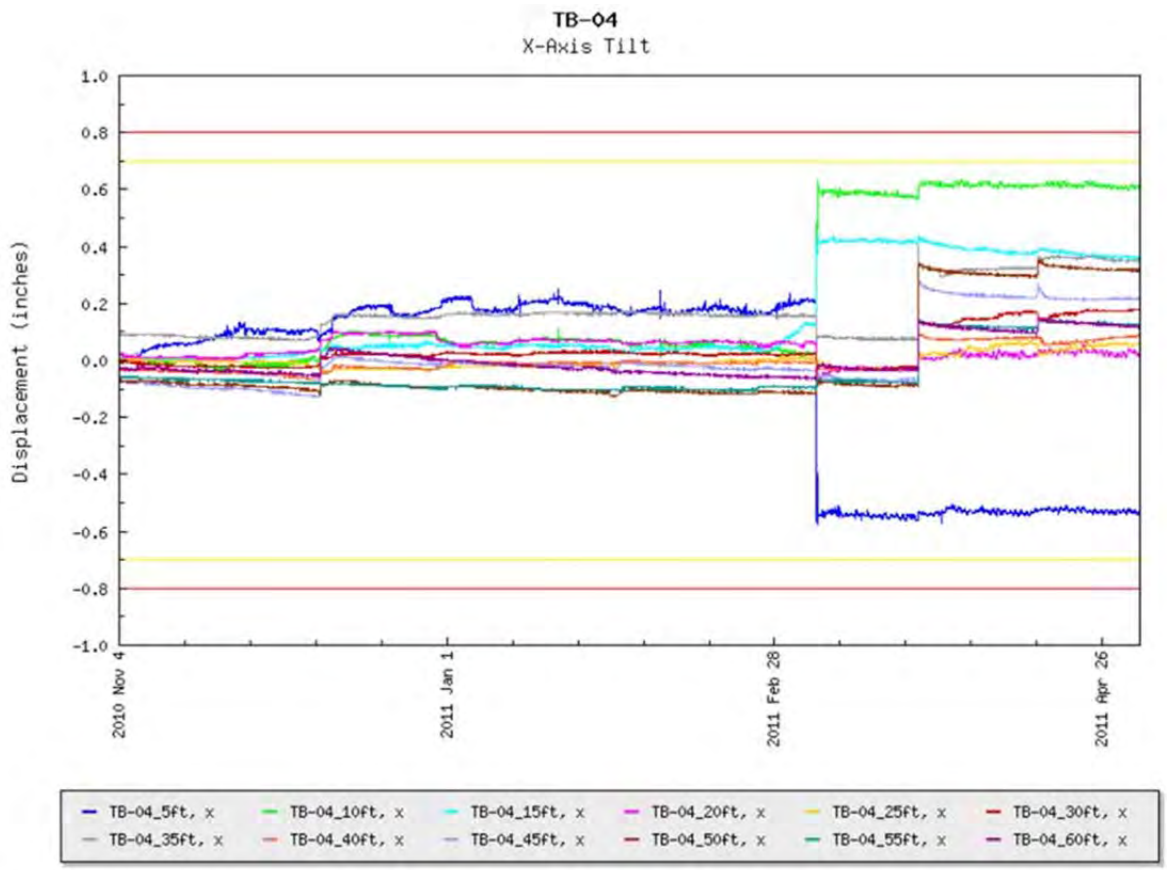


Figure 12 –Site TB-04, X-Axis

Communication to all the tilt sensors and one piezometer were lost at site TB-03 on April 4, 2011. ODOT geotechnical staff visited the site the next day for a visual inspection. Everything inside the steel casing cap was covered in grout, the IPI string was also displaced upward about 11 inches. It appears the inclinometer casing sustained a direct drill hit between 40 and 45 feet (Figure 13). The drilling damaged an IPI cable which caused an electrical short on all of the IPIs in the string. No data was collected during the grouting operation. Though it is unlikely the Little Dipper IPIs survived the pressure grout, further investigation is planned. The deeper piezometer was also fatally damaged. Currently the only functioning sensor at TB-03 is the upper-most piezometer, located at 20 feet. Since the boring has been filled with grout the IPIs cannot be retrieved.

Figure 13 –Site TB-03, Accidental Grouting

Power Problems and Solutions

Because of the typical cloudy Pacific Northwest winter weather, the ADAS power system was designed with a higher capacity battery and larger solar panel in an attempt to accommodate these conditions. Two battery thresholds were set in Atlas to notify the ODOT ADAS geotechnical staff; an Amber Alarm at 12 volts and a Red Alarm at 11 volts. Once the battery drops below 11 volts it can no longer reliably hold a charge and needs to be replaced. The ODOT geotechnical staff started to receive Amber Alarms in October 2010. Since the cellular modem consumes the most power, the data transmission rate was changed from 15 minutes to 60 minutes; the sample rate remained at one sample every 15 minutes. Despite the modification, batteries levels were still dangerously low. The ODOT geotechnical staff developed a swap program which involved purchasing extra batteries and recharging them in the office. They would then swap out a weak battery with a new one when an Amber Alarm went off. If a battery dropped below 11 volts, the battery was retired and a new battery installed.

Because the instrumentation enclosures were installed atop, 7 foot poles they require ladder access as well as special tools to access the enclosure. The 84 A/hr batteries are also heavy and awkward, especially atop a 7 foot ladder in the rain. A new CR1000 program has been developed to extend battery life and minimize, hopefully eliminate, battery swapping. In the current system, the Atlas server collects the data each 60 minutes. In the new program the logger will send the data to the server if a sensor exceeds a certain value; the server will only collect data from the logger once per day. The sample frequency remains at 15 minutes. The new program minimizes the amount of time the cellular modem transmits data, which will increase battery life by limiting cellular modem activation to once a day unless the sensors exceed a defined value. Alarms will still be generated by and controlled through the original Atlas server.

CONCLUSIONS

Landslide mitigation measures have to be removed in order to widen a section of I-5 south of Portland, Oregon; tied-back soldier pile retaining walls will replace historic mitigations. During the construction, six Automated Data Acquisition Systems (ADAS) were installed to monitor an area with historical landslide activity. Each ADAS site includes a string of 7 to 12 in-place inclinometers, 1 or 2 piezometers, a datalogger, multiplexor, vibrating wire interface, cellular modem, charge controller, battery, and solar panel.

These ADAS sites are powerful tools in evaluating slope stability in near real time. Atlas web-based software provides a user friendly interface allowing a variety of users to interact with their data by setting threshold alarms, looking at plots, and generating automated reports. If a sensor exceeds a user defined threshold the ODOT geotechnical staff is notified via e-mail, text message and/or via pager. Care was taken to design a system that is both theft resistant, re-usable, and functions through the overcast Pacific Northwest winters. The biggest challenge thus far is powering ADAS through the Pacific Northwest cloudy winters, a problem hopefully solved by a new program regulating data transmission. Two ADAS sites have confirmed movement in suspected zones. IPI and piezometers were lost during construction at one location, but the logging equipment will be relocated and reused for another project. The current ADAS is proving a valuable hands-off tool in monitoring slopes and keeping the construction crews and traveling public safe.

REFERENCES:

- Conference Proceedings
    1. Mikkelsen and Green. Piezometers in Fully-Grouted Boreholes. *International Symposium on Geomechanics, Oslo, Norway*. September 2003.
- Oregon State Government Reports
    1. Oregon Department of Transportation. *Final Geotechnical Report: I-5 SW Iowa Street Viaduct BR#08197 Sec.* Report DOT-RD-75-196. Oregon Department of Transportation Geo/Hydro/HazMat Unit, 2009.

Transportation Pooled Fund Project of
Highway Geotechnical Applications of Ground-Based Lidar

John Kemeny
Dept. Mining and Geological Engineering
University of Arizona, Tucson, AZ 85721
kemeny@email.arizona.edu
(520) 621-4448

John Combs
Dept. Mining and Geological Engineering
University of Arizona, Tucson, AZ 85721
combs.john@gmail.com
(520) 873-7218

Vanessa Bateman
US Army Corps of Engineers
Nashville, TN
(615) 414-6377
Vanessa.Bateman@tn.gov

Ty Ortiz
CDOT Rockfall Program
Denver, CO 80216
(303) 398-6588
Ty.Ortiz@dot.state.co.us

Christ G. Dimitroplos
Arizona Department of Transportation
Phoenix, AZ 85034
CDimitroplos@azdot.gov
(602) 712-7850

Krystle Pelham
New Hampshire DOT
Concord, NH
(603) 271-3734
KPelham@dot.state.nh.us

Brad Foltz
PA Department of Transportation
Harrisburg PA 17120
(717) 783-6408
www.dot.state.pa.us

Nick Priznar
Arizona Department of Transportation
Phoenix, AZ 85034
(602) 712-8089
NPriznar@azdot.gov

Doug Hadjin
New York State DOT
Albany, NY 12232
(518) 457-4728
dhadjin@dot.state.ny.us

Keith Turner
Colorado School of Mines
Golden, CO
(303) 273-3802
kturner@mines.edu

Mark McClelland
Texas DOT
Now retired

Bill Webster
Caltrans, Geotechnical Design-North
Sacramento, CA
(916) 227-1041
bill_webster@dot.ca.gov

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

This project was funded under FHWA/ADOT contract TPF-5(166).

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

This paper discusses some preliminary results from the Transportation Pooled Fund Study TPF-5(166). Participants in the study in include Arizona, California, Colorado, New Hampshire, New York, Pennsylvania, Tennessee and Texas. As part of the pooled fund project, Lidar scanning is conducted in each state, and the point clouds are analyzed to look at rock mass characterization, rockfall, slope stability and change detection. The purpose of the pooled fund project is to demonstrate geotechnical applications of ground-based LIDAR for highway slopes, and to train state DOTs on the use of point cloud processing software.

Even though the project is not yet complete, a number of important concepts and procedures have been developed, including documented procedures for conducting Lidar scanning for geotechnical analysis, integrating Lidar results into other geotechnical software, analyzing overhangs for rockfall hazard, and analyzing ground movement using Lidar change detection. These topics will form the basis for the final AASHTO report on the Lidar pooled fund project which ends December, 2011.

INTRODUCTION

Terrestrial Lidar (also referred to as Terrestrial Laser Scanning and Ground-Based Lidar) is a new technology for capturing and visualizing three-dimensional data. The output of a Lidar scan is a “point cloud” consisting of millions of points that represent the 3D surface that was scanned. Measurements and calculations can be made from the point cloud itself, or from a triangulated surface produced from the point cloud. Point cloud processing software refers to software specifically designed to process point clouds from Lidar scans. Lidar scanning and point cloud processing software are now routinely being used in a number of engineering and architectural fields (Sparpoint, 2010).

Terrestrial Lidar is ideal for many geotechnical applications, including surface and underground rock mass characterization, surface slope stability, underground ground control, rockfall and displacement monitoring, and change detection. Lidar scanning collects data from a distance, and thus increases the safety associated with data collection in unstable ground conditions. It is also able to collect data from areas where normal access would be difficult or impossible. Lidar data is high resolution and eliminates many of the human bias and low-resolution issues with hand-collected data. Finally, Lidar scanning and point cloud processing is very fast and allows for the characterization of a site in a timely fashion. Details on the use of terrestrial Lidar for geotechnical applications is described in Kemeny and Turner (2008).

A Department of Transportation Pooled Fund Study was created to investigate highway geotechnical applications of ground-based Lidar (Kemeny and Turner, 2009). The pooled fund study focuses on the following issues:

- Transportation agencies are not familiar with the technology and the possible range of uses
- There are a limited number of case studies illustrating the uses of terrestrial Lidar for specific geotechnical applications
- There is a lack of documented and fully qualified procedures for data acquisition to ensure accuracy and “fitness for purpose” of the terrestrial Lidar data.
- Transportation agencies do not have expertise with point cloud processing software.

Eight states are participating in the Lidar pooled fund project: Arizona, California, Colorado, New Hampshire, New York, Pennsylvania, Tennessee, and Texas. Arizona is the lead state on the project. Dr. John Kemeny from the University of Arizona is the PI on this project, and Christ Dimitroplos is the ADOT administrator of this project. Sites were selected in each of the eight states for Lidar scanning. Each state selected one site for scanning, except for Colorado where two sites were chosen. In most cases, the sites selected were rock highway slopes that exhibited rockfall or slope stability problems.

Even though the project is not yet complete, a number of important concepts and procedures have been developed. These include:

- Efficient procedures for scanning highway rock slopes and processing the results for rock mass characterization

- Integrating Lidar results into other geotechnical software, including software for rockfall trajectory analysis and slope stability.
- The introduction of the Overhang Factor (OHF) based on vertical cross sections and 3D meshes through the Lidar point clouds
- Developing procedures for multiple scans of a highway slope and processing the results for change detection

These topics will form the basis for the final AASHTO report on the Lidar pooled fund project, and some results from each of the topics above are discussed in this paper. While the project is currently still in progress, the current work that has been completed has shown that Lidar is an important new technology for highway geotechnical studies.

Section 2 of this paper provides details on the scanning in each of the 8 states. Section 3 provides details on the analysis of the Lidar scans using point cloud processing software. This includes analysis areas of rock mass characterization, rockfall, slope stability and change detection. Conclusions are given in Section 4.

SCANNING SITES AND DETAILS

Figure 1 shows pictures of seven sites in six states where Lidar scanning has been conducted as part of the pooled fund project, and Table 1 provides the location of these sites. At the time of the writing of this paper, scanning has been conducted in Arizona, Colorado, New Hampshire, New York, Pennsylvania and Texas, and scanning in California and Tennessee is planned for June, 2011. All of the sites are highway rock slopes with the exception of one of the Colorado sites, which is an artificial slope composed of Jersey and plastic barriers. Table 1 also lists the geology and current geotechnical issues associated with these sites. All of the sites exhibit some geotechnical issues, and Figure 2 shows some of these issues, including large rockfall events at the Colorado and Arizona sites (Figures 2a and 2b) and a large wedge failure at the Texas site (figure 2c).

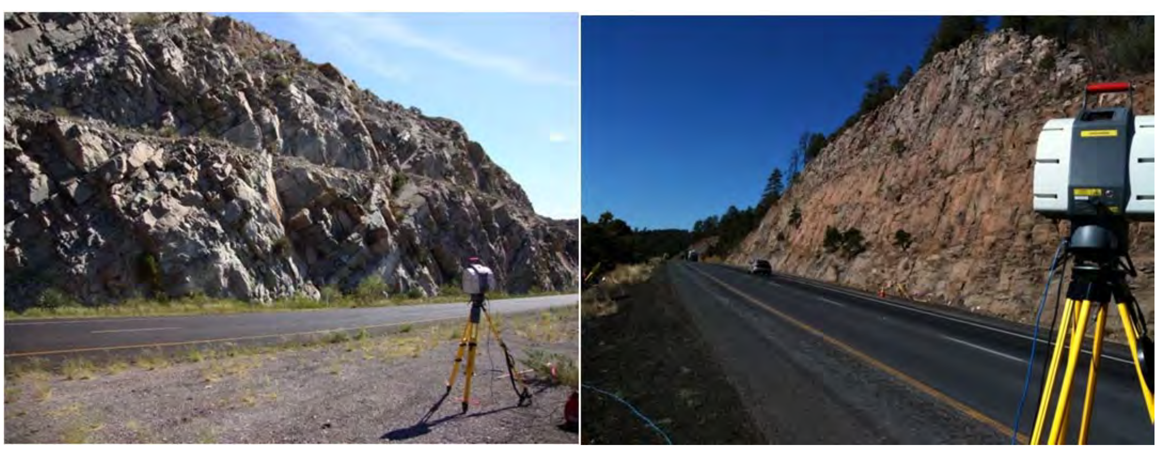


Figure 1. Lidar scanning sites, a) Texas, b) Arizona, c) New Hampshire, d) New York, e) Colorado, f) Colorado, g) Pennsylvania

Table 1. Location of scanning sites along with geology and current geotechnical issues

State	Location	Geology	Geotechnical Issue at Site
AZ	I40 near Flagstaff	Basalt flows and breccia	Rockfall
CO	US 285 near Indian Hills	Ancient landslide deposit	Rockfall
CO	CDOT lot near Empire	Jersey and plastic barriers	Change detection test
NH	I93, near Woodstock	Gneiss and foliated schistose	Slope stability
NY	Route 5 near Schenectady	Finely bedded Limestone	Rockfall
PA	SR 11/15 New Buffalo	strained clastic sedimentary rocks	Rockfall and slope stability
TX	Loop 375 near El Paso	Marble, limestone, rhyolite	Rockfall and slope stability

Figure 2. Examples of geotechnical issues as some of the sites, a) Colorado site showing large rockfall in the highway, b) Arizona site showing large rockfall in the ditch, c) Texas site showing large wedge slide.

Some details on the Lidar scanning that was conducted at each site are presented in Table 2. This includes the slope length and height scanned, the number of scans, the total number of Lidar points captured, the average point spacing, and the type of scanner used.

Table 2. Scanning details

State	Number of Scans	Total Number LIDAR Points	Average Point Spacing	Scan Length (ft)	Ave. Scan Height (feet)	Scanner
AZ	5	16.5 million	2-3 cm	2400	100	Leica Scanstation
CO	4	7.5 million	1.5-3 cm	420	60	Optech ILRIS 3D
CO	7	350,000	2-3 cm	40	15	Optech ILRIS 3D
NH	7	11.0 million	2-3 cm	870	98	Optech ILRIS 3D
NY	7	8.7 million	2-3 cm	730	72	Optech ILRIS 3D
PA	5	3.3 million	3-4 cm	560	128	Cyrax 2500
TX	3	4.3 million	2-4 cm	870	120	Leica Scanstation

As an example, some details of the New York scanning are described and illustrated in Figure 3. As shown in Figure 3a, seven scans were made to capture the 730 foot long by 70 foot high slope. An Optech ILRIS 3D scanner was used along with a Nikon D90 digital camera mounted on top, as shown in Figure 3b. Each scan took about 15 minutes and captured about 1.25 million points, resulting in point spacings ranging from 2 to 3 cm. As described in Kemeny and Turner (2008), a point spacing of 2-3 cm is ideal for geotechnical studies. Two methods for registering the point clouds were utilized. First of all, points were selected for surveying, which included a point under each scanner location and points in the area between the scanner and the rock face. Secondly, the orientation of the scanner was carefully measured, which included measurements of scanner bearing, forward-back tilt, and sideways tilt. In total, about 4 hours were spent at the New York site, which included equipment setup, scanning and miscellaneous data collection, setting markers for surveying, and packing up the equipment. Color point clouds were produced by draping the high-resolution digital images from the camera onto the Lidar point clouds, using software provided by Optech. Figures 3c and 3d show two views of the color point cloud from scan 7. Similar procedures as described above were followed at the other sites listed in Table 2.

Figure 3. Scanning of the New York site, a) location of the seven scans, b) Optech ILRIS 3D scanner with Nikon camera mounted on top, c) and d) two views of the color point cloud from scan 7.

POINT CLOUD PROCESSING

One of the main purposes of the pooled fund project is to demonstrate the different kinds of analyses that can be conducted with point cloud processing software. There are a number of point-cloud processing programs available to process Lidar data. For this pooled fund project, all processing is conducted using the Split FX software (Split Engineering, 2011). This software is designed specifically to extract geotechnical information from point clouds. Each participating state in the pooled fund project will receive a copy of the Split FX software along with training on how to produce the kinds of results shown in this paper.

Rock Mass Characterization

In rock mass characterization, important attributes of the geologic structure are extracted from the point clouds, including discontinuity orientation, spacing, persistence, and roughness. These results are then used in the analysis of slope and underground stability and as input into 3D numerical models. In this paper we show some results from two states on discontinuity orientation. Complete results are available in the individual state reports and the final report of the pooled fund project.

Figure 4 shows results for scan 6 from the New Hampshire site. Figure 4a shows the color point cloud, Figure 4b shows the triangulated mesh, Figure 4c shows the automated delineation of the fractures, and Figure 4d shows the fractures plotted on a lower hemisphere stereonet (larger fractures are plotted with larger circles). Details on the automated fracture delineation are given in Kemeny and Turner (2008). Important structural features that can be seen in the figures includes the green and red colored joints sets that are forming narrow wedge failures, and the large light blue foliation planes where previous plane sliding has occurred.

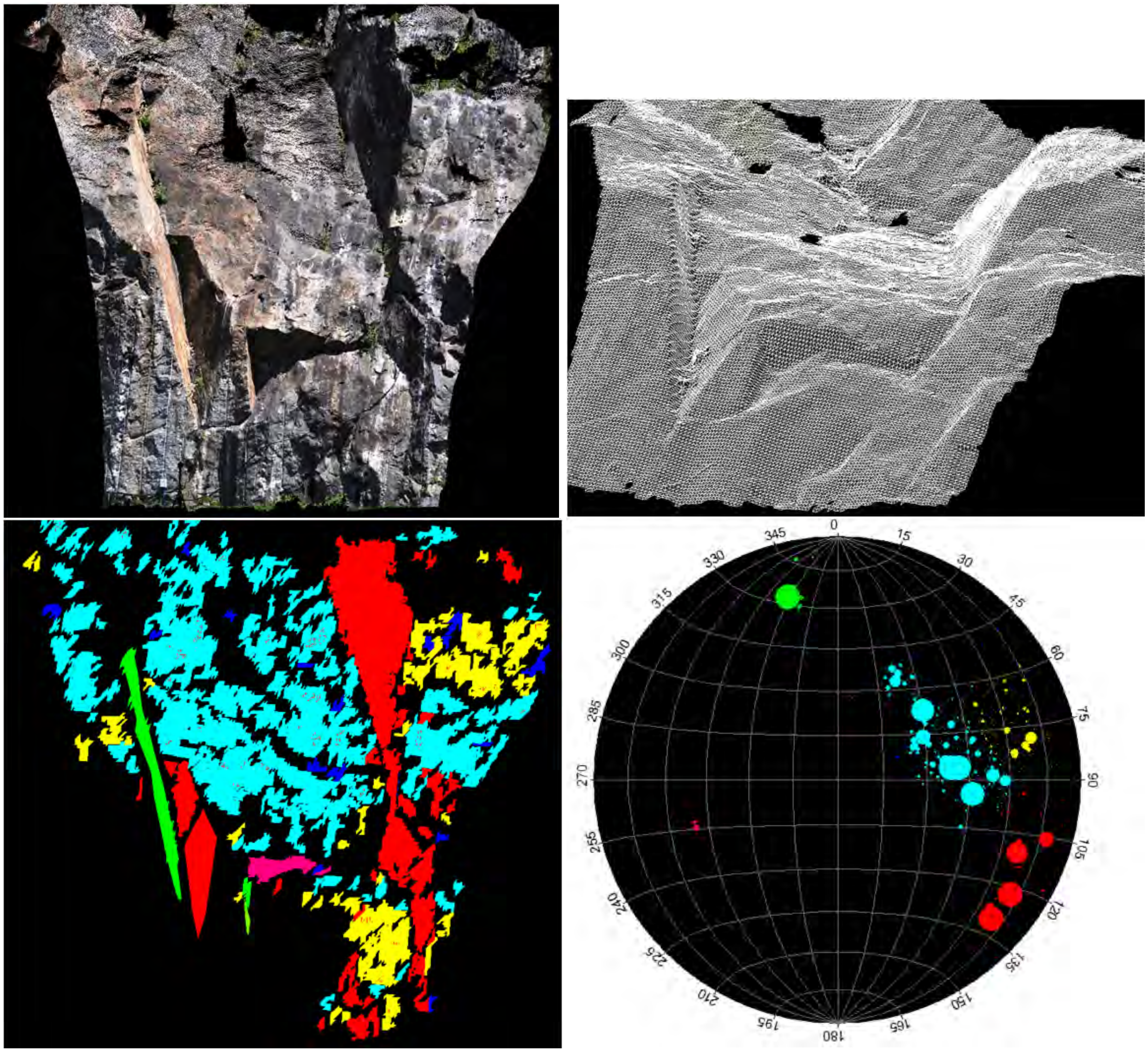


Figure 4. Results from scan 6 in the New Hampshire site, a) color point cloud, b) triangulated mesh, c) delineated fractures, d) lower hemisphere equal area stereonet.

Figure 5 shows results for scan 2 from the Texas site. Figure 5a shows a picture of the scan location, Figure 5b shows the grayscale point cloud, Figure 5c shows the automated delineation of the fractures, and Figure 5d shows the fractures plotted on a lower hemisphere stereonet. A grayscale point cloud was used rather than the color point cloud in this case because the color point cloud had sun shadows obscuring parts of the scan. Important structural features that can be seen in these figures are the persistent red and yellow fractures that form large wedge failures in several sections of the Texas site.

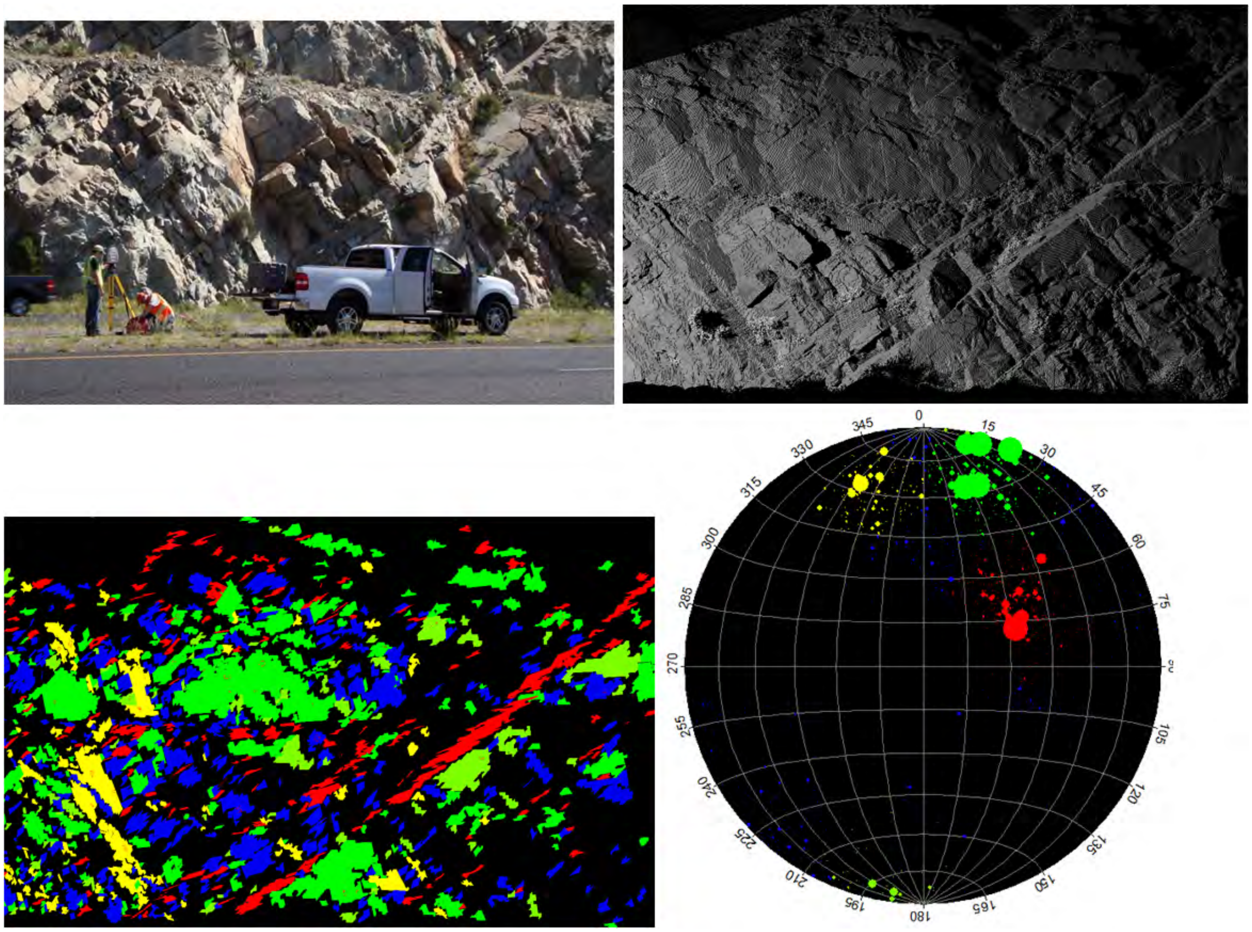
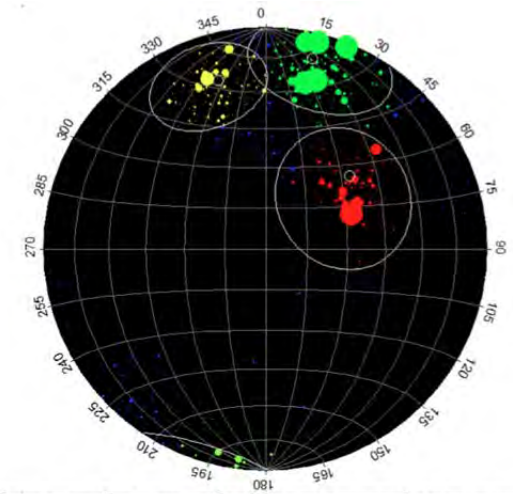


Figure 5. Results from scan 2 in the Texas site, a) picture of the scan location, b) grayscale point cloud, c) delineated fractures, d) lower hemisphere equal area stereonet.

Slope Stability

The rock mass characterization results from the last section can be used to assess slope stability. As an example, slope stability is analyzed for the results shown in Figure 5 (Texas site scan 2). There are many ways that rock slope stability analysis can be conducted. Here a probabilistic slope stability analysis is conducted using the Rocscience Swedge program (Rocscience, 2011). The first step is to calculate the statistical properties for each joint set, which includes the average dip, average dip direction and the Fisher constant (Fisher, 1953). The Fisher constant is a measure of the amount of scatter in the poles, and the lower the Fisher constant the more scatter in the poles. Statistical results for the yellow, green and red sets for scan 2 from the Texas site are shown in Figure 6a. Additional information that is required includes the friction angle and cohesion for the fractures, the orientation of the rock slope, and hydrology information. Since no lab testing was conducted, friction angles of 35 degrees and 0 cohesion were assumed for all discontinuities. Also both wet and dry cases are considered, where in the dry cases the fractures were assumed to be completely dry and in the wet cases the fractures were assumed to be half filled with water.

The Swedge program considers two joint sets at a time and searches for unstable wedges formed from the combination of these sets and the orientation of the free face. To produce a wedge, it randomly picks an orientation from each set and calculates the factor of safety. It does this 10,000 times for each pair of discontinuity sets. If the result of the two picks and the rock face does not result in a removable wedge, it labels the pick as an invalid wedge. The probability of failure is the percentage of trials that results in a failed wedge. There are three discontinuity sets in scan 2 from the Texas site and therefore three combinations that must be considered: red and green, red and yellow, green and yellow. Also both wet and dry cases are considered, giving 6 overall cases. The results of these six cases are shown in Table 3. A probability of failure over about 15% indicates a potentially unstable wedge. Two cases in Table 3 show probabilities of failure over 15%, the wet and dry cases for wedges formed from the red and yellow sets. Figure 6b shows that this correlates very well with the large unstable wedge that occurred along this slope.



Set	Ave. Dip	Ave. Dip Direction	Fisher constant
Yellow	68.1	164.2	46
Green	77.2	193.7	31
Red	41.4	228.9	16

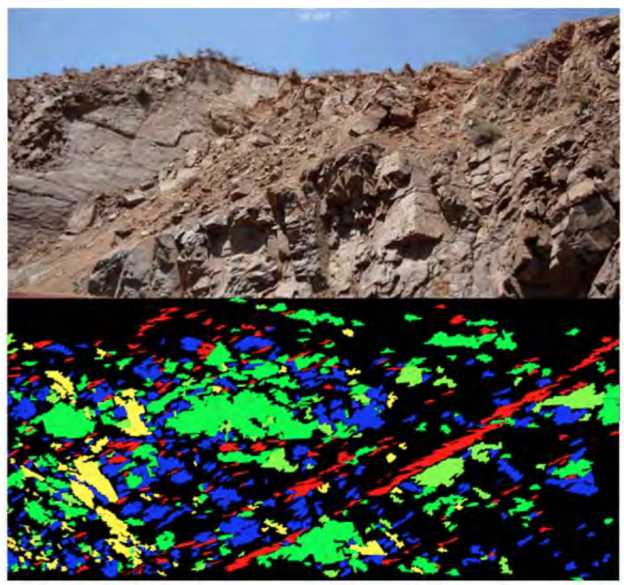


Figure 6. a) Statistical properties for the yellow, green and red sets in the Texas scan 2, b) occurrence of large wedge failure formed by the yellow and red sets.

Table 3. Probability of wedge failure for scan 2 from the Texas site

Wedge	Water	Cases	Valid wedges	Failed wedges	Stable wedges	Prob of Failure
Red and Green	Dry	10000	1987	490	1497	4.90%
Red and Green	Wet	10000	1987	1370	617	13.70%
Red and Yellow	Dry	10000	9308	1561	7747	15.61%
Red and Yellow	Wet	10000	9308	3944	5364	39.44%
Green and Yellow	Dry	10000	1397	396	1001	3.96%
Green and Yellow	Wet	10000	1397	484	913	4.84%

Rockfall

Rockfall is the occurrence of small blocks of rock that become dislodged from a slope and fall or roll to the bottom of the slope. Rockfall occurrence is widespread in highways through rocky terrain and must be considered when designing a highway/ditch/slope system. One indication of a rockfall hazard is the presence of overhangs, as shown in the color point cloud from scan 7 from the New York site in Figures 3c and 3d. Ground based Lidar is an excellent technology for characterizing overhangs, because the dense point cloud provides details on overhangs of all sizes, and because ground-based scanners are most often aimed up at a slope.

As part of the pooled fund project, we have developed a rating system to quickly evaluate a slope for overhangs using either cross sections or 3D triangulated meshes. Segments of a cross section or individual triangles in a 3D mesh are rated as shown in Figure 7. No overhang gets a rating of 0, slight overhang gets a rating of 1, moderate overhang gets a rating of 4 and severe overhang gets a rating of 10. The ratings are summed over a cross section or mesh and normalized by the maximum rating possible (maximum is where every segment or triangle gets a 10). Results for cross sections from scan 4 of the Arizona site and scan 1 of the Pennsylvania site are shown in Figures 8a and 8b, respectively. Based on these results and others from the pooled fund project, OHF ratings greater than about 4% indicate a potential rockfall hazard. Figure 9 shows an example of calculating the overhang rating from a triangulated mesh (taken from the center region of the point cloud in Figure 3c). The triangulated mesh in Figure 9a gives an overhang factor of 5.3 compared with an overhang factor of 6.1 for a vertical cross section through the center shown in Figure 9b.

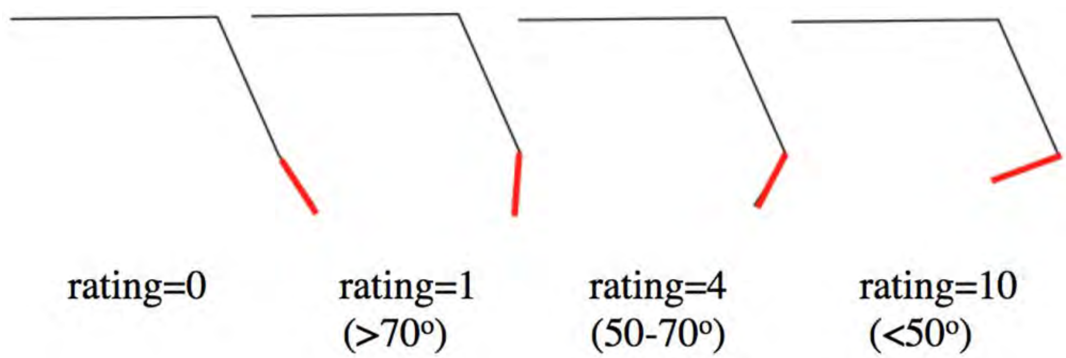
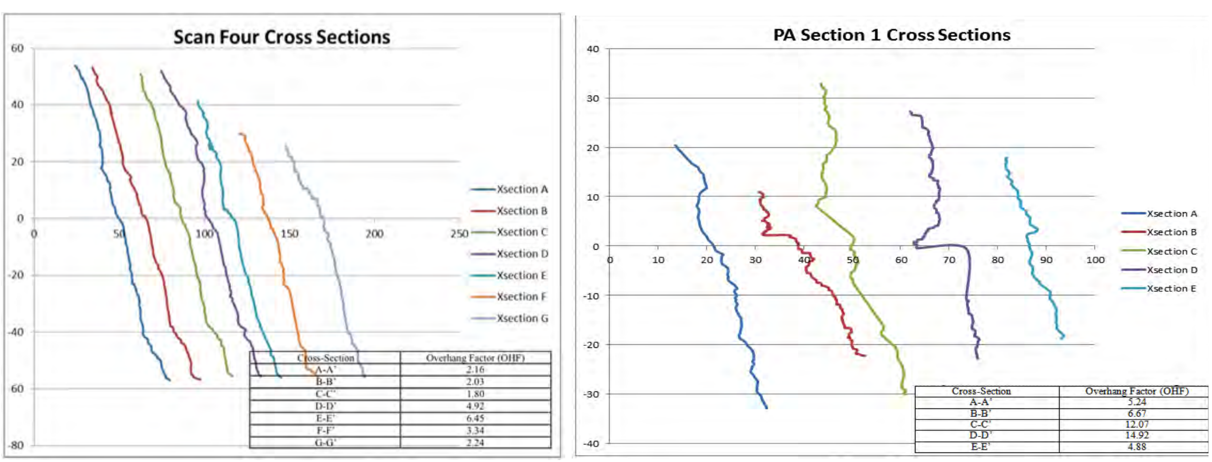


Figure 7. Overhang ratings for no, slight, moderate and severe overhangs.



Cross-Section	Overhang Factor (OHF)
A-A'	2.16
B-B'	2.03
C-C'	1.80
D-D'	4.92
E-E'	6.45
F-F'	3.34
G-G'	2.24

Cross-Section	Overhang Factor (OHF)
A-A'	5.24
B-B'	6.67
C-C'	12.07
D-D'	14.92
E-E'	4.88

Figure 8. Overhang ratings for cross sections from the Arizona and Pennsylvania sites.

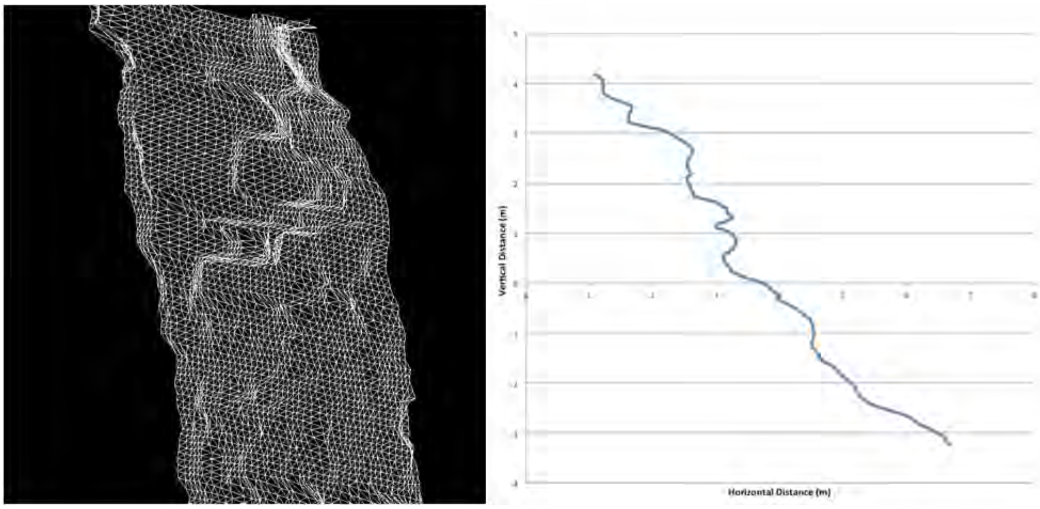


Figure 9. a) triangulated mesh giving OHF of 5.3, b) cross section through the same area giving an OHF of 6.1

Lidar point clouds can also be used to conduct a trajectory analysis of unstable blocks on a slope. Figure 10 shows an example from the New York site where an overhanging block near the top of the slope was analyzed. A cross section from the point cloud was utilized as shown in Figure 10b, along with the Rocscience Rocfall program. Calculated trajectories are shown in Figure 10c and indicate that the rockfall fence at the site is suitable and will prevent the unstable block from landing in the highway.

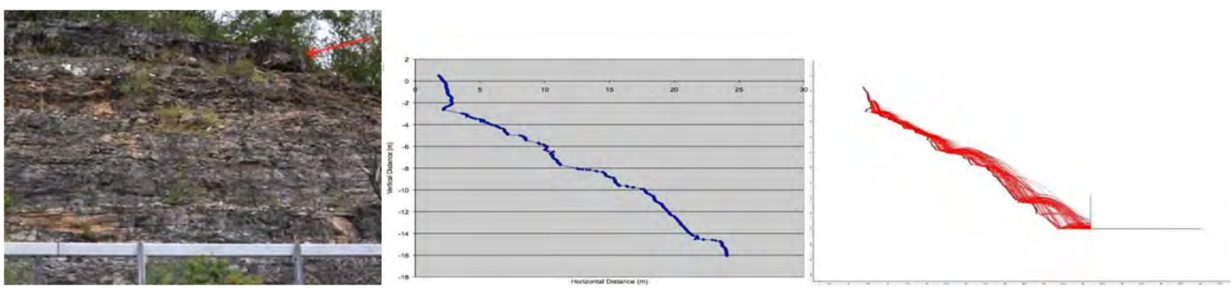


Figure 10. Trajectory analysis of an unstable block at the New York site, a) location of overhanging block, b) cross section, c) calculated trajectories.

Change Detection

LIDAR change detection involves taking the point clouds from “before” and “after” scans and creating a “difference” point cloud. The difference cloud is created by subtracting the after cloud from the before cloud. Before the subtraction takes place the two point clouds must be accurately aligned. This is a two-step process. In the first step markers are manually inserted in the before and after clouds (normally 3 markers are used) to give a crude alignment. This step is not necessary if the scanner is not moved between the before and after scans. In the second step the clouds are accurately aligned using an Iterative Closest Point (ICP) algorithm. Also, before subtraction takes place the clouds are smoothed using a smoothing filter. The subtraction itself is complicated since the points in the two scans will not line up with each other. Thus for a point in one cloud, the equivalent point in the other cloud is determined using interpolation. Once the difference cloud is created, it can be visualized in different ways. Normally change that involves movement away from the scanner (such as a missing rock on the slope) is given one color (red in our case), change that involves movement towards the scanner (such as new rock in a ditch) is given another color (blue in our case) and movement less than the noise level is given a third color (gray in our case). Cross sections through the difference point cloud can also be made to look at detailed movements.

The results from before (September 2009) and after (June 2010) scans taken at the US 285 site in Colorado are shown in Figure 11. Figure 11a and 11b show before and after photos, and Figure 11c shows the difference point cloud of the left and middle parts of the slope. Red indicates missing material and gray indicates movement less than the noise level. Because of the numerous small changes that occurred to the soil matrix in the nine months between the before and after scans, the background noise level is high in this case. Thus only rockfall events with a size greater than 6 inches are shown in Figure 6c. Figure 6c clearly shows several large rockfall events that have occurred, as well as a sign in the lower right that appears to have been bent at some point in the nine-month interval.

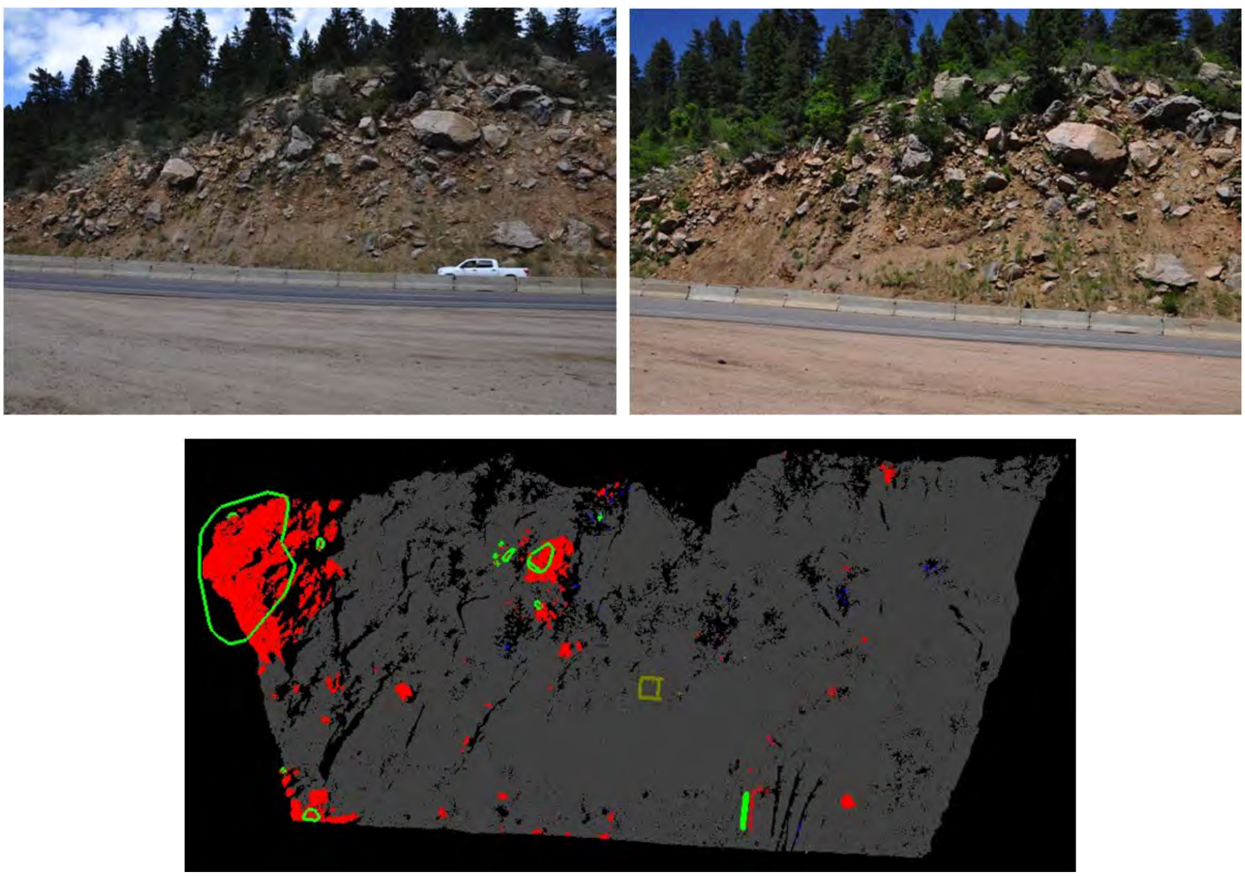


Figure 11. Before (September 2009) and after (June 2010) pictures and difference cloud along Highway 285 in Colorado.

The results from a change detection test conducted near Empire, Colorado is shown in Figure 12. Figure 12 shows the results of “movement 1”, analyzed from scans taken from a distance of 110 meters from the artificial rock wall. The wall is broken up into 12 segments labeled A through L as shown in Figure 12b, and the actual displacements in movement 1 as made by Ty Ortiz are given in Figure 12a. Figure 12c shows the difference point cloud from the Otech ILRIS-3D scans from a distance of 110 meters. Movement toward the scanner is shown in blue, movement away from the scanner is shown in red, and the threshold noise level for this difference cloud is about 1.1 cm (0.43 inches, shown in gray). Comparing Figure 12b and 12c, the blue patches agree with the actual displacements for all regions where a displacement greater than 0.4 inches was made (B2, C1, C2, D1, F2, H1, H2, I1, I2, K, L). Block J is red because it was removed between the scans.

1st Move by hand - Upper Barriers
(measurement taken at base of barrier)

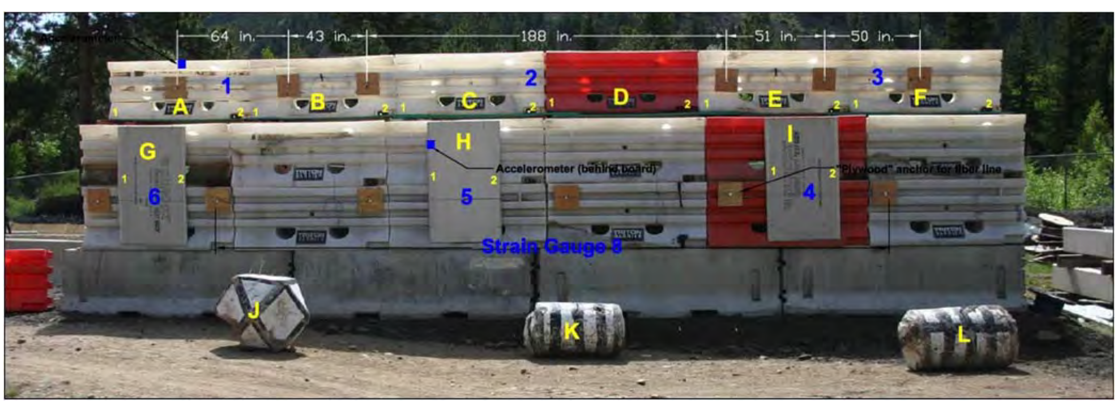
≈ Time - 12:00 to 12:10	Barrier	Movement (inches)	Notes
	A1	0.30	
	A2	0.20	
	B1	0.00	.
	B2	1.70	
	C1	2.50	
	C2	2.50	
	D1	2.00	Barrier D was not intended to be moved. It moved with the barrier C movement
	D2	Not Recorded	No measurable movement from baseline
	E1	Not Recorded	No measurable movement from baseline
	E2	0.15	
	F1	Not Recorded	No measurable movement from baseline
	F2	3.30	Strain Guage #3 lost tension when moving this barrier. At approx 12:15 it was retensioned.

1st Move by hand - Concrete Board

≈ Start Time 12:30	Board	Movement (inches)	Notes
	G1	0.30	Movement of board measured from wall at approximate center. Boards moved with wooden wedges on each side except for board 'I', which we moved with one wedge only.
	G2	0.20	
	H1	0.40	
	H2	0.90	
	I1	0.60	
	I2	2.50	

1st Move by hand - Rocks in front of wall

Start time not recorded	Board	Movement (inches)	Notes
	J	NA	Rock movement is approximate. Rocks moved by pushing/rolling toward scanners. Rock J removed to simulate change detection.
	K	2.00	
	L	29.00	



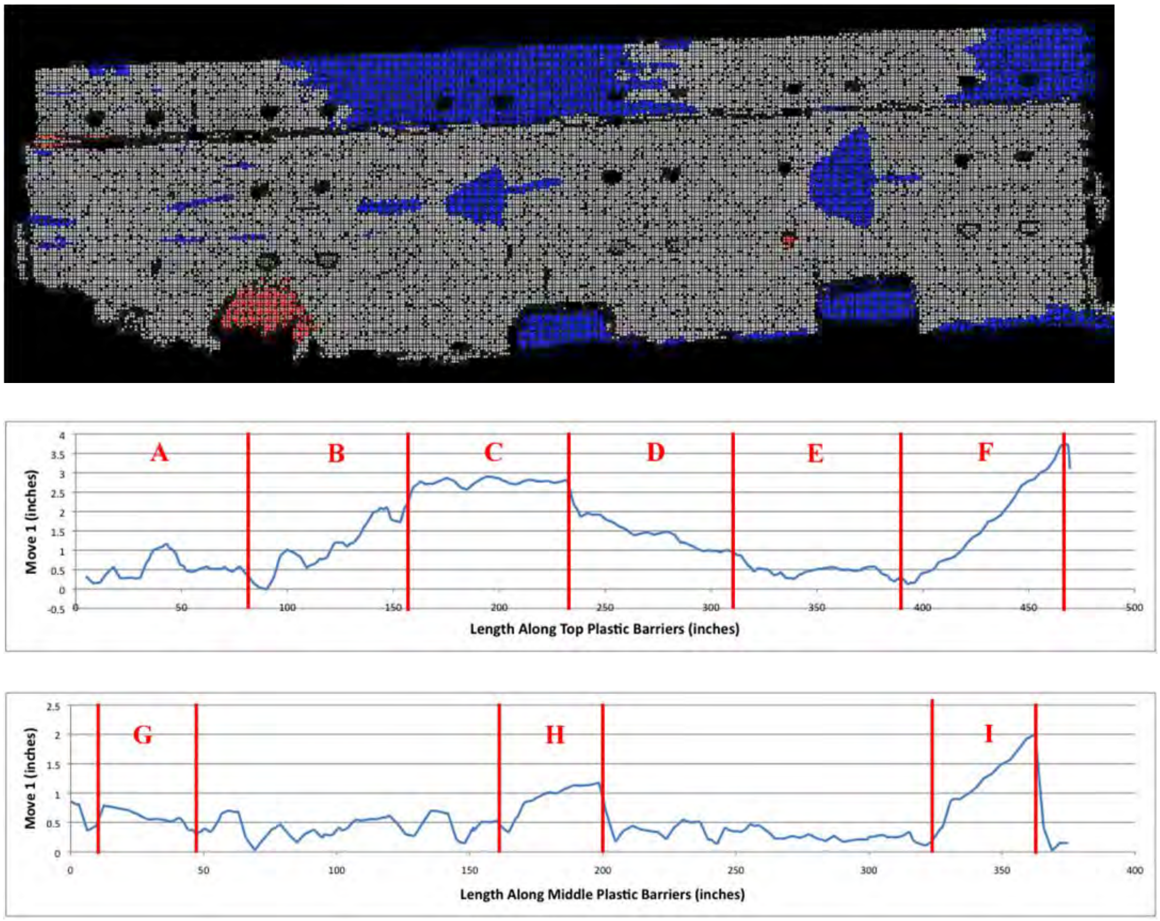


Figure 12. a) actual displacements made to the artificial wall, b) artificial wall with reference numbers, c) difference point cloud using Optech scanner at a distance of 110 meters, d) differences along upper barriers, e) differences along middle barriers.

Figures 12d and 12e show cross sections through the difference point cloud through the top and middle plastic barriers, respectively. They provide additional details and show that the LIDAR scans taken from a distance of 110 meters are capable of detecting change as small as 0.2 inches (see G1 and G2 locations in Figure 12e compared with actual movements given in Figure 12a). Some of the waviness in these cross sections is due to the corrugations in the plastic barriers.

CONCLUSIONS

This paper has discussed some preliminary results from the Transportation Pooled Fund Study TPF-5(166). Participants in the study in include Arizona, California, Colorado, New Hampshire, New York, Pennsylvania, Tennessee and Texas. As part of the pooled fund project, Lidar scanning is conducted in each state, and the point clouds are analyzed to look at rock mass characterization, rockfall, slope stability and change detection. The purpose of the pooled fund project is to demonstrate geotechnical applications of ground-based LIDAR for highway slopes, and to train state DOTs on the use of point cloud processing software.

Even though the project is not yet complete, a number of important concepts and procedures have been developed, including documented procedures for conducting Lidar scanning for geotechnical analysis, integrating Lidar results into other geotechnical software, analyzing overhangs for rockfall hazard, and analyzing ground movement using Lidar change detection. These topics will form the basis for the final AASHTO report on the Lidar pooled fund project which ends December, 2011.

REFERENCES

Kemeny, J. and K. Turner. 2008. Ground-Based LiDAR: Rock Slope Mapping and Assessment, US Dept. Transportation Federal Highways Admin. Central Federal Lands Highway Division, Publication No. FHWACFL/TD-08-006, 2008.

Split Engineering. 2011. www.spliteng.com

Kemeny, J. and K. Turner. 2009. Transportation Pooled Fund TPF-5(166), "Application of Three-Dimensional Laser Scanning for the Identification, Evaluation, and Mangement of Unstable Highway Slopes"

Fisher, R. 1953. Dispersion on a sphere. Proc. Royal Society London, A217, p. 295,305

Rocscience. 2011. RocFall version 4.0 Statistical Analysis of Rock Falls, Swedge version 5.010 3D Surface Wedge Analysis for Slopes, http://www.rocscience.com

Sparpoint. 2010. Spar Point Research LLC investigates and reports on 3D scanning, imaging and position capture technologies, http://sparllc.com/

Advantages of Using Digital Photogrammetry for Rock Cut Slope Design

An Example in the Central Appalachian Mountains

William M. Roman
Gannett Fleming, Inc.
P.O. Box 67100
Harrisburg, PA 17106-7100
(717)-763-7212
wroman@gfnet.com

Michael A. Knight
Gannett Fleming, Inc.
P.O. Box 67100
Harrisburg, PA 17106-7100
(717)-763-7212
mknight@gfnet.com

Robert E. Johnson
Pennsylvania Department of Transportation
District 3-0
715 Jordan Avenue
Montoursville, PA 17754-0218
570-368-8686
roejohnson@state.pa.us

Prepared for the 62nd Highway Geology Symposium, July, 2011

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original authors.

ABSTRACT

Roadway realignment to improve line of sight along a curved stretch of U.S. Highway 15 on Bald Eagle Mountain necessitated removal of an existing rockfall fence and modification of an existing rock cut near South Williamsport, Pennsylvania. Digital photographs of the existing rock cut were taken with sufficient overlap to permit development of a three-dimensional, digital model, which encompassed a 420-foot-long section of the cut through inclined strata of the Juniata and Tuscarora Formations. A digitizing tool was used to measure the orientation, magnitude, and location of 238 rock discontinuities (bedding planes and joints) visible in the model.

A Brunton Geotransit was used to collect additional measurements from behind the rockfall fence and to validate model measurements. Discontinuity measurements were plotted on stereonets to analyze the potential for plane, toppling, and wedge failures along the proposed rock cut. Discontinuity measurements were also used to optimize the azimuth of proposed horizontal drain holes.

Using digital photogrammetry on this project provided several advantages over traditional methods. Safety was greatly enhanced since discontinuities in higher parts of the outcrop were measured without rappelling or free-climbing. A great volume of data was collected in a short period of time, reducing exposure to rockfalls and traffic. Discontinuities were accurately measured over a greater length, and discontinuities having limited surface exposure were measured by digitizing their trace across the outcrop model. Using digital photogrammetry reduced the potential for human error in the collection, recording, and processing of data, and facilitated performance of the analyses required for the rock cut slope design.

INTRODUCTION

To reduce future maintenance along a curved portion of U.S. Highway 15 on Bald Eagle Mountain south of Williamsport, the Pennsylvania Department of Transportation (PennDOT) proposed to remove an existing rockfall fence, modify the existing rock cut, and provide a rockfall catchment zone. Modification of the rock cut provides will also improve sight distance. Rock discontinuity data were needed to perform the rock slope stability analyses required for developing cut slope recommendations. Digital photogrammetry was successfully used to collect the rock discontinuity data. This paper summarizes the site geology, describes the process used to obtain the discontinuity data, and discusses some of the advantages of using digital photogrammetry for rock cut slope design.

PROJECT GEOLOGY

The project involves a curved segment of U.S. Highway 15 situated on the north flank of Bald Eagle Mountain just south of the West Branch of the Susquehanna River (Figure 1). Bald Eagle Mountain is the northernmost ridge within the Appalachian Mountain Section of the Ridge and Valley Province. The Susquehanna River flows through the valley between Bald Eagle Mountain and the Allegheny Front to the north *(1)*. The Appalachian Mountain section features sedimentary strata that have been deformed into open and close plunging folds having narrow

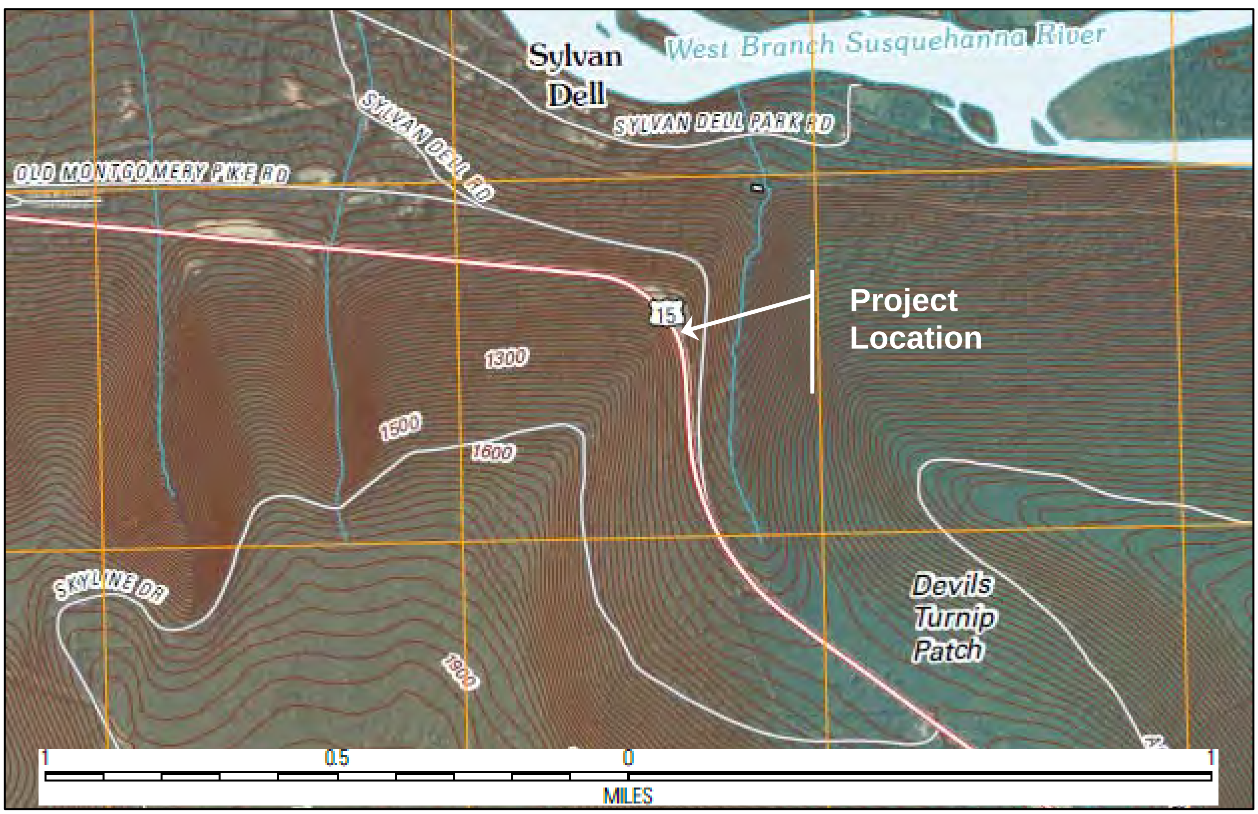


Figure 1 – Project Location Shown on USGS Montoursville South 7.5' Quadrangle Map

hinged planar limbs. The resultant topography consists of long, narrow ridges and broad to narrow valleys that are generally concordant with bedrock geology.

The project lies at the northern end of Wind Gap, which is the first north-south trending gap west of the nose of Bald Eagle Mountain. Wind Gap is more than 700 feet deep, has a bottom elevation of 1,205 feet, and drains to both the north and south. Braun *(2)* has suggested that Wind Gap represents a former segment of the Susquehanna River that was bypassed when one of the river's tributaries south of Bald Eagle Mountain eroded headward through softer rock around the nose of the ridge and captured the river on the north side of Bald Eagle Mountain.

As shown in Figure 2, the project straddles the contact between the Juniata Formation of the Upper Ordovician system to the south and the Tuscarora Formation of the Lower Silurian system to the north *(3)*. According to Faill *(4)*, these rocks formed from sediments deposited in the Appalachian basin from a source area (the Taconic highlands) located to the southeast of the basin. The rocks generally consist of fine- to coarse-grained quartzite interbedded with very fine and fine-grained sandstone, siltstone, and shaly siltstone.

The project is situated on the north limb of the Nittany anticline, the axis of which roughly follows the crest of Bald Eagle Mountain to the south of the project site. Structure

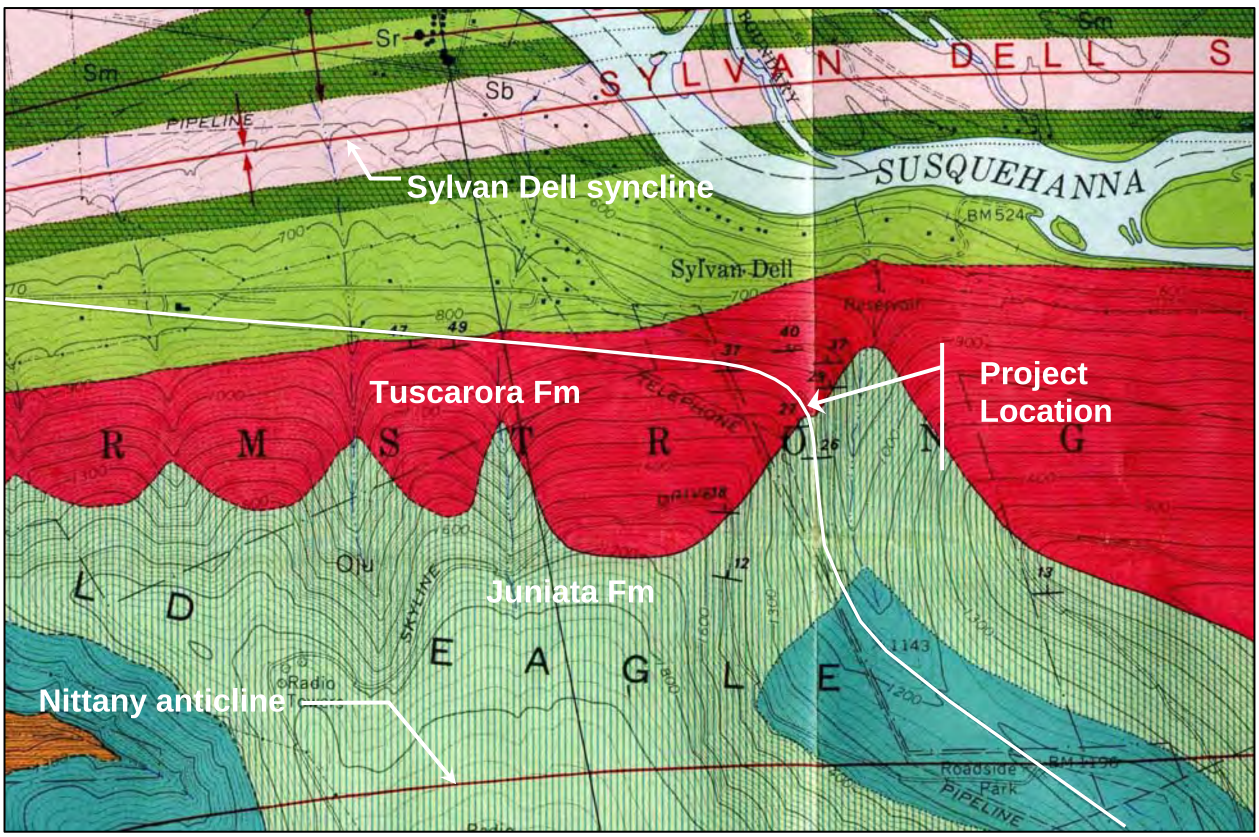


Figure 2 – Project Location Shown on 1:24,000 Scale Geologic Mapping by Faill *(3)*

symbols on the geologic map (Figure 2) indicate bedding dips nearly due north at an angle of 26 to 27 degrees at the project site and at an increasing angle to the north of the site. The axis of the Sylvan Dell syncline lies at the base of Bale Eagle Mountain further to the north.

The project site lies within the limit of the Pre-Illinoian glaciation and just south of the limit of Illinoian glaciation *(5)*. During the Early Pleistocene, an ice dam near the eastern end of Bald Eagle Mountain created Glacial Lake Lesley in the valley between Bald Eagle Mountain and the Allegheny Front to the north. Glacial Lake Lesley had a surface elevation of approximately 1,100 feet and was drained via an outlet at Dix approximately 70 miles southwest of the project site *(2)*. Periglacial weathering is evident in the boulder stream within Wind Gap south of the project site, which is labeled “Devils Turnip Patch” in Figure 1.

EXISTING ROCK CUT

The project encompasses approximately 1,200 linear feet of roadway, from Station 361+00 at its southern end to Station 373+00 at its northern end. Project plans and cross sections indicate the existing cut is a benched cut with a steeper lower slope and shallower upper slope. The steepest slope on the lower part of the cut occurs from about Station 366+50 to Station 368+00, where the cut slope is about 0.4:1 (H:V). The upper slope is cut as steeply 0.8:1 at Station 365. The bench appears to have been truncated by an existing landslide from about Station 367+00 to Station 370+00. The area above the cut is State Forest Land and mantled with colluvium.

DIGITAL PHOTOGRAMMETRY

Photogrammetry is the practice of determining the geometric properties of objects from photographic images. Stereophotogrammetry uses photos taken from different locations to determine an object’s coordinates. The technique takes advantage of the fact that sight rays from an object strike different parts of a camera’s image sensor as the location of the camera changes. Knowing the camera’s focal length, the shift in sight rays can be used to triangulate an object’s location. The advent of digital cameras and high-speed computers combined with the development of sophisticated software has led to the application of digital photogrammetry to rock face characterization *(6)*.

An initial site reconnaissance indicated that the most significant portion of the existing rock cut could be photographed from a grassy area adjacent to the parking lot of the scenic overlook located on the east side of the highway at the apex of the curve (Figure 1). Because of the existing steep slope, challenging access, and high traffic volume, digital photogrammetry was selected as the method of choice for obtaining the bulk of the rock discontinuity data required for the slope stability analyses.

Characterizing a rock face using digital photogrammetry involves the following steps:

1. Calibrate camera/lens
2. Plan photo shoot
3. Place survey control points

4. Take photographs
5. Match common points – solve free network orientation
6. Digitize control points – solve absolute orientation
7. Generate digital terrain model (DTM)
8. Define features and perform analyses

Calibrate Camera/Lens

Camera and lens calibration parameters include focal length, radial lens distortion, principal point offsets, decentring distortion, and pixel scaling factors. Calibration is typically done once for each camera and lens combination, and re-calibration between projects is not necessary. A previously calibrated Canon EOS 5D Mark II digital camera with a 50-mm long lens was used for this project.

Plan Photo Shoot

The photo shoot is planned based on the type of photogrammetry model to be developed. Three types of models are typically used in terrestrial photogrammetry:

1. Simple Convergent Model: Photograph the area or object from two camera locations to create a stereo pair of images.

2. Image Fan Model: Multiple photographs are taken obliquely from two locations, usually with a long focal length lens.

3. Image Strip Model: Photographs taken head on from multiple locations, usually with a wide angle lens.

Site topography restricted the distance from which the rock face could be photographed (i.e., the object distance), so the only feasible model for imaging the rock face was an image strip. This model usually requires 60 percent horizontal overlap of the images; however the amount of overlap was increased due to the curving nature of the alignment. The plan also sought to keep the object distance the same for each camera location. Based on the anticipated object distance and focal length, a plan for shooting the rock face as a strip of nine images was developed (Figure 3).

Place Survey Control Points

Prior to taking the photographs, twenty-two 6-inch-diameter control points were spray-painted and labeled at various locations on the cut and the existing rockfall fence. PennDOT surveyors provided the Northing, Easting, and elevation of each spray-painted control point. Although only tree control points are required to register the model to a real world coordinate system, using more than three control points provides redundancy, which is useful for estimating the accuracy of the model and insuring against bad observations.

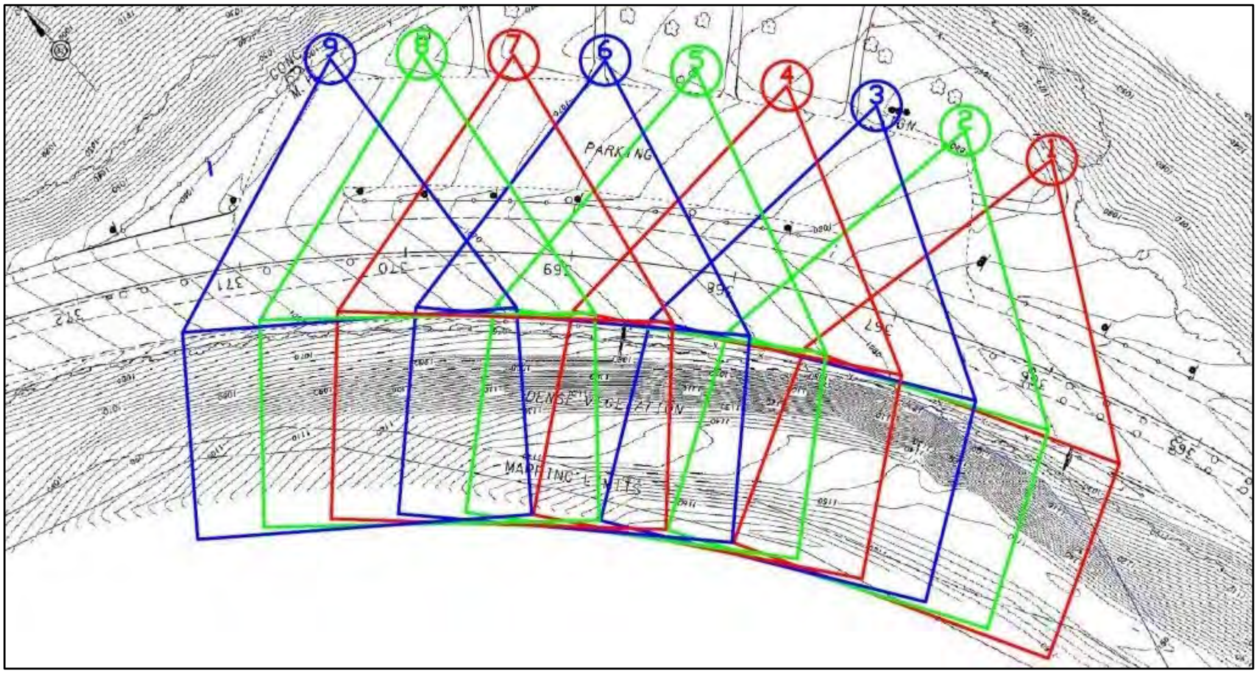


Figure 3 – Plan for Shooting Image Strip Model

Take Photographs

Photographs were taken on the afternoon November 19, 2010. A tripod was used to steady the camera, and a remote switch was used to activate the shutter and minimize movement of the camera (Figure 4).

Match Common Points – Solve Relative Orientation

Photogrammetry software (3DM Analyst Lite Suite) developed by Adam Technology was used to develop the three-dimensional digital model. Common points were identified in pairs of overlapping photos, and a bundle adjustment was performed to determine the relative camera locations.

Digitize Control Points – Solve Absolute Orientation

After the free network orientation was solved, surveyed control points were digitized, and an absolute orientation was solved to tie the model to a real world coordinate system (Northing, Easting, elevation). Figure 5 illustrates the plotted locations of the camera stations (purple icons), the matched points (red dots), and the surveyed control points (teal dots).

Generate DTM

Following the bundle adjustment, a digital terrain model was generated based on the matched points and other common points recognized by the software program (Figure 6). Although the DTM can be edited to add break lines and delete extraneous surfaces resulting from

Figure 4 – Photographing the Rock Face

vegetation, no such editing was performed since the DTM was being used primarily for discontinuity measurements.

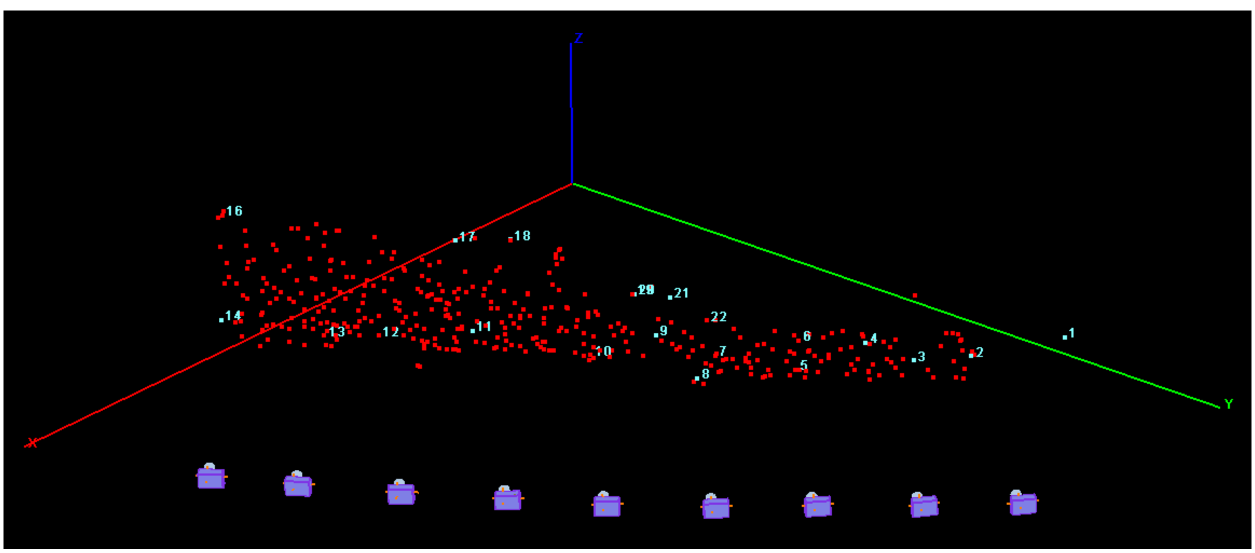


Figure 5 – Locations of Nine Camera Station, Matched Points, and Control Points

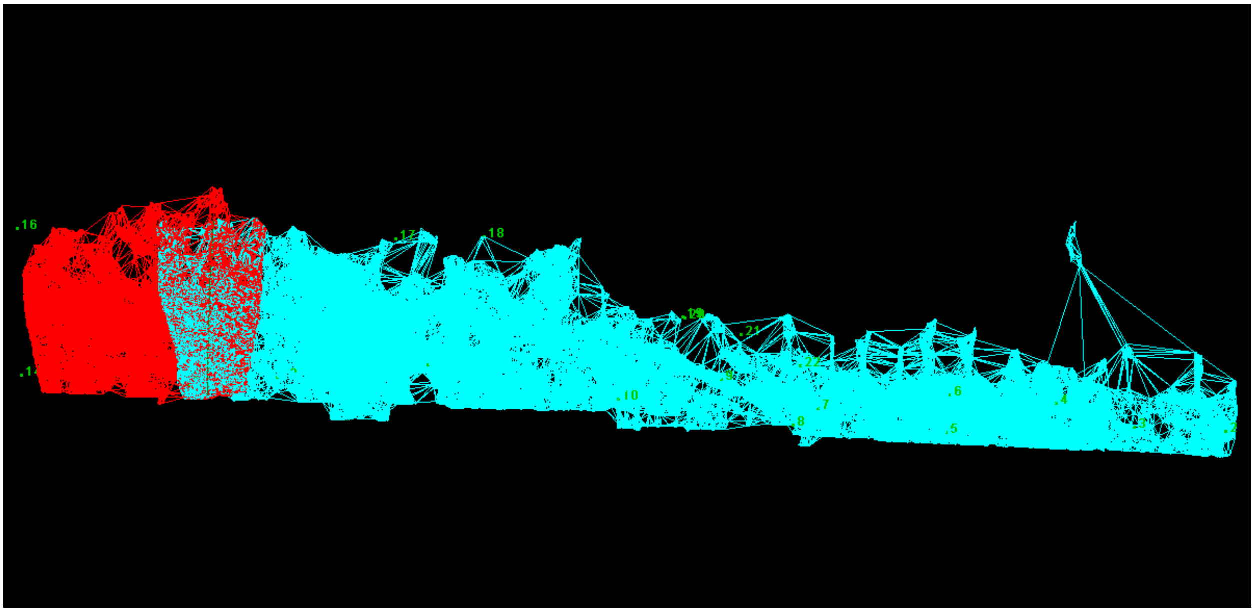


Figure 6 – Digital Terrain Model (DTM) of Approximately 420 feet of Existing Rock Cut

Define Features and Perform Analyses

The feature-picking tool of the software was used to measure 238 of the rock discontinuities (bedding planes and joints) within the 420-foot-long section of roadway encompassed by the digital model. In Figure 7, locations and types of measurements are indicated by the colored disks. Bedding planes are represented by green disks, joints by yellow disks, and the contact between the Tuscarora and Juniata Formations by a red disk. The disks are drawn in accordance with the extent of the points digitized while delineating the feature, so disk size denotes the relative magnitude of the feature. In addition to orientation, the feature-picking tool provides location coordinates of the feature and the magnitude of the feature (Figure 8).

Although the tops of bedding planes were generally not observable in the model due to the upward tilt of the camera during image acquisition, the model permitted delineation of bedding planes based on the change in the location of their traces across the surface of the digital terrain model. Since the camera bearing was generally to the southwest when the photos were taken, joint faces striking north and northwest are more readily apparent in the photographs. The tops of some more steeply dipping beds are evident is some locations in the model. These beds appear to be foreset beds.

An additional reconnaissance of the existing rock slope was performed following development of the model in order to collect additional data and validate the model. An additional 35 rock discontinuities were measured using a Brunton Geotransit with the magnetic declination set to 11.5 degrees west. Measurements were collected from areas north and south of the limits of the digital photogrammetry model and the area behind the existing wall. Some large, conspicuous faces within the model area were also measured for model validation.
The dimensions of 33 rock blocks were measured to provide data for the Colorado Rockfall Simulation Program (CRSP).

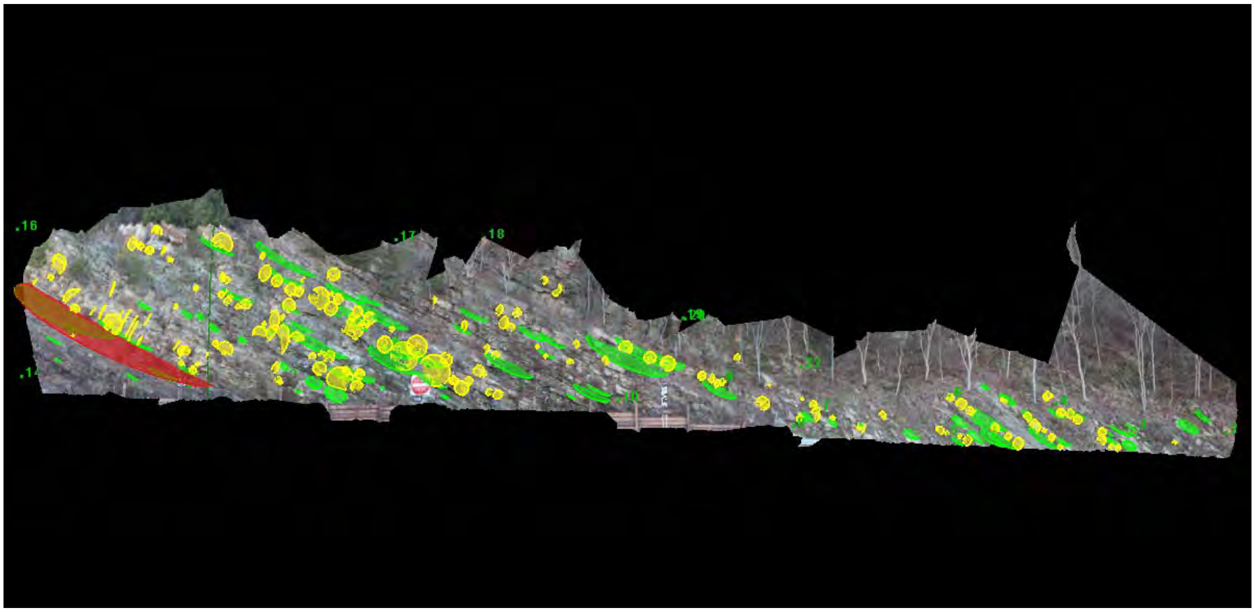


Figure 7 – Digital Model Showing Locations of Measured Bedding Planes (Green Disks); Joints (Yellow Disks), and the Tuscarora/Juniata Contact (Red Disk)

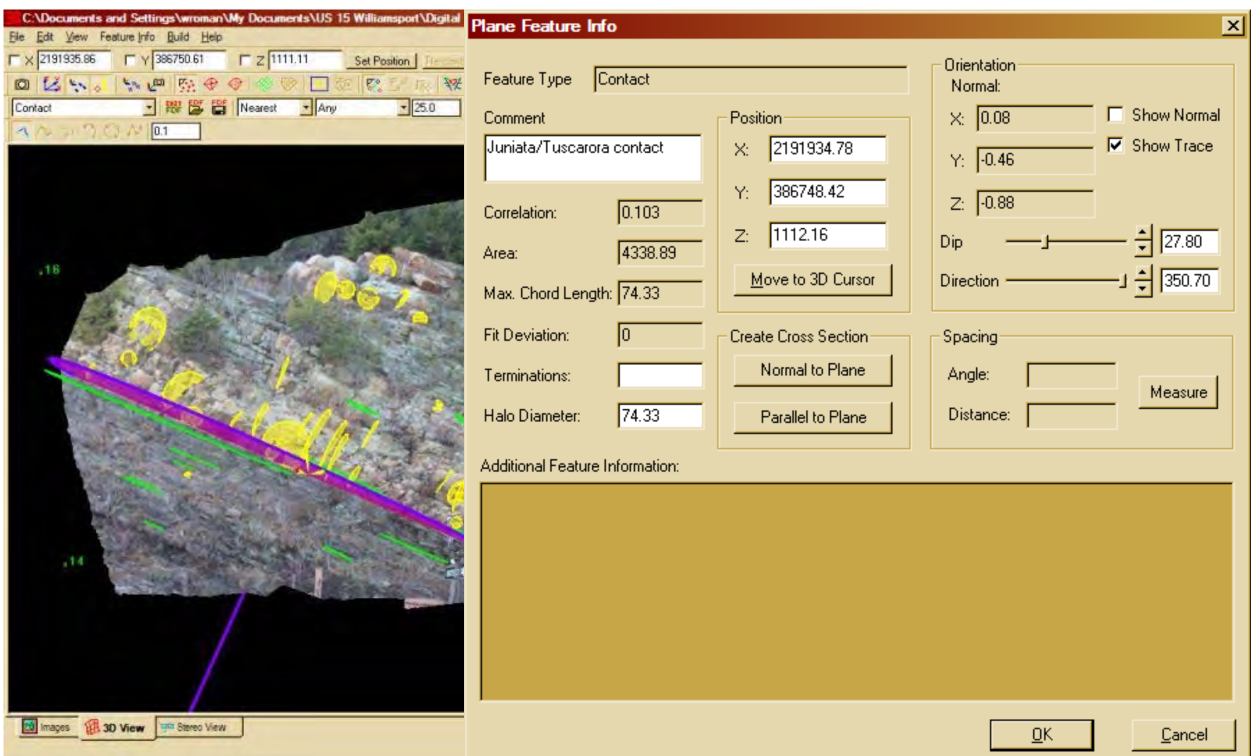


Figure 8 – Dialog Box with Selected Feature Information

ROCK FACE CHARACTERIZATION

Discontinuities of two types were identified—bedding joints (including the contact between the Tuscarora and Juniata Formations) and two sets of steeply dipping joints. The great circles and dip vectors corresponding to the 273 measurements taken on the three joint sets are plotted on stereonets in Figure 9. Joint set 1 consists of bedding joints dipping to the north. Joint set 2 consists of steeply dipping joints striking north-south, and joint set 3 consists of steeply dipping joints striking east-west.

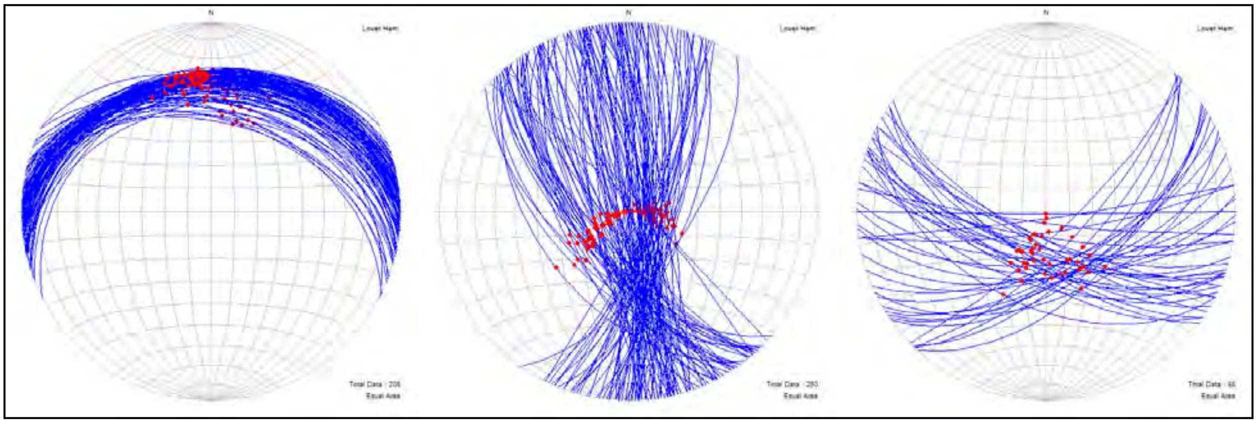


Figure 9 – Great Circles (Blue Lines) and Dip Vectors (Red Dots) of Joint Sets 1, 2, and 3

Joint Set 1 (Bedding)

104 bedding discontinuities were measured—81 from the digital model and 23 by conventional method using the Brunton Geotransit. The manual measurements were generally consistent with the model measurements as indicated by the plot of bedding dip vectors in Figure 10. Bedding dip measurements vary from 24.8 to 51.2 degrees, and measured bedding dip direction varies from N27°W to N27°E. The geometric mean (by Fisher analysis) is a bedding dip of 33 degrees due north. This vector is also the principal component by principal component analysis. The best fit by least squares analysis is 32 degrees due north. The contact between the Juniata and Tuscarora Formations dips 27.8 degrees N9.3°W. About ten of the more steeply dipping vectors to the north-northeast were measured on an unfavorable bed noted during the field reconnaissance.

The variability in bedding may be due in part to the variability within the inferred depositional environment—a coastal alluvial plane with braided channels and possibly a migrating beach strand. Other factors that may contribute to the variability in the bedding measurements include blocks being slightly displaced and the accuracy of the measurements. Bedding dip angle and dip direction do not correlate with roadway stationing.

Joint Set 2

Joint set 2 includes 125 joints having strikes between N50°W and N40°E and dipping steeply to the east and west. The average orientation (by least squares) of joint set 2 dips 88

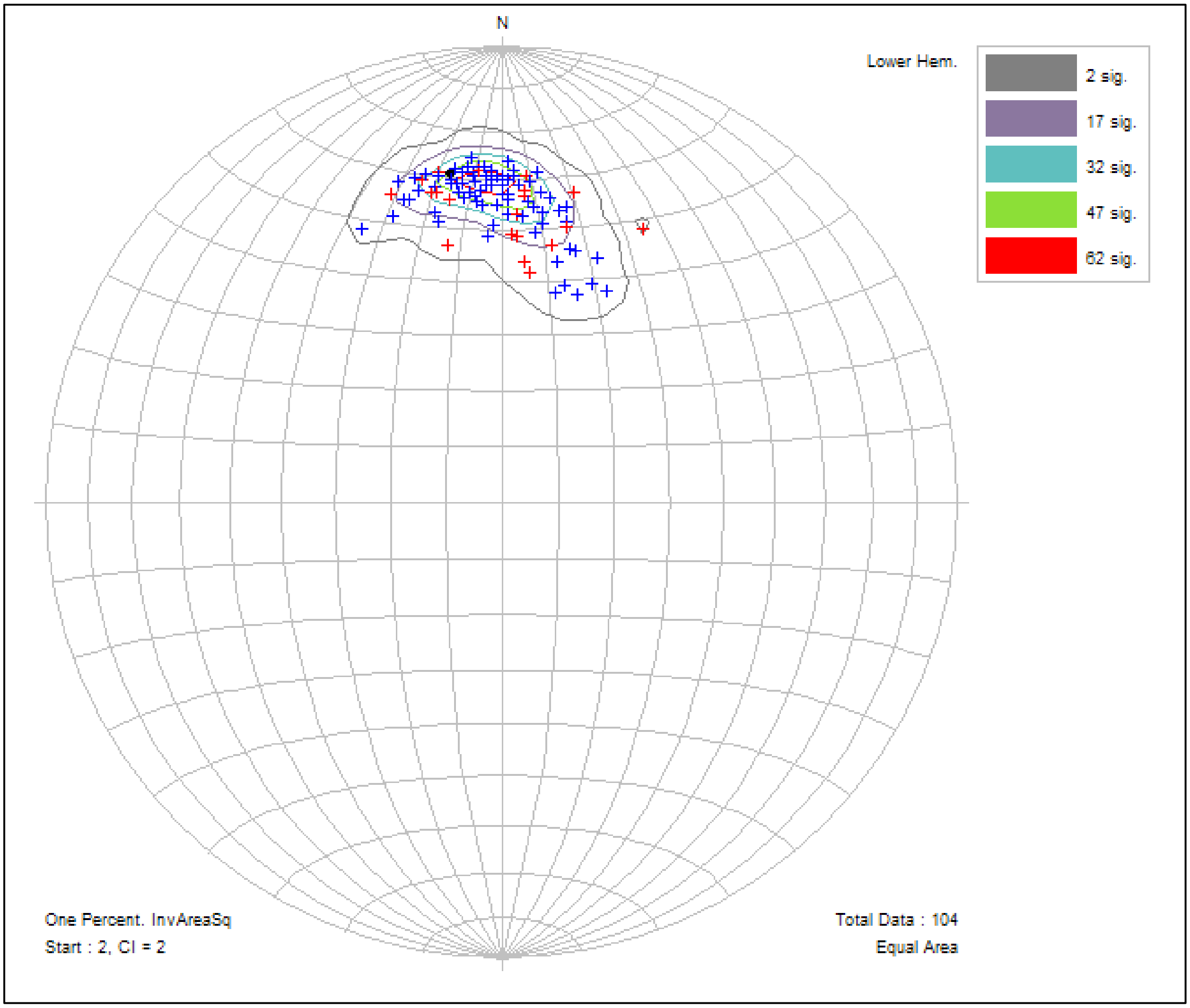


Figure 10 – Bedding Dip Vectors from Digital Photogrammetry Model (Blue Crosses) and Manual Measurements from Outcrop (Red Crosses)

degrees S78°W. These joints likely coincide with the principal stress trajectory that deformed the strata to their present orientation. Joints having these strike orientations may be more developed due to valley stress relief parallel to Wind Gap through which the roadway passes. Joint Set 2 discontinuities are potential lateral release surfaces for plane failures along bedding.

Joint Set 3

Joint set 3 includes 44 joints having strikes ranging from N40°E to S50°E and dipping steeply to the south. The average orientation of joint set 3 dips 67 degrees S2°W. These joints, which are less well developed than the other two sets, tend to strike parallel to bedding strike and are potential up dip release joints for plane failures along bedding.

ADVANTAGES OF USING DIGITAL PHOTOGRAMMETRY

Digital photogrammetry worked quite well on this project. The discontinuity measurements from the digital photogrammetry model were used in stereonet analyses of the kinematic potential for plane, wedge, and toppling failures. The measurements were also used in optimizing the azimuth of horizontal drains proposed to reduce water pressure within the rock mass and improve cut slope stability.

Advantages in using digital photogrammetry included the following:

1. Data were collected safely from challenging to reach areas with no rappelling or free climbing.

2. Exposure to traffic, rockfalls, and inclement weather was significantly reduced.

3. A very robust set of measurements was collected in a very short period of time.

4. There was no need to set up and measure traverses since the software provides coordinates of each measurement.

5. Discontinuities were measured across a greater extent of their length than could be measured by hand.

6. Discontinuities with limited exposure were easily measure by digitizing their trace across the model.

7. Measurements could be evaluated as the discontinuities were being digitized and the disks were created, by rotating the 3D image to assess the closeness of the fit of the disk to the joint surface from various perspectives.

8. The digital photogrammetry software provided Northings, Eastings, and elevations for each measurement, which permitted an analysis of the variability of bedding dip and dip direction along the proposed alignment.

9. Human error in recording the measurements was reduced since the measurements were recorded by the computer.

10. Shallow dipping (<5 degrees) joints can be very challenging to measure with a Brunton compass. Although such joints were not present within this rock mass, when they are present, they can be easily measured using digital photogrammetry.

REFERENCES:

1. Sevon, W. D. *Physiographic Provinces of Pennsylvania, 4th ed.* Map 13. Pennsylvania Geological Survey, 4th ser., 2000, scale 1:2,000,000. (Available online at: [http://www.dcnr.state.pa.us/topogeo/maps/map13.pdf. Accessed Dec. 20, 2010).

2. Braun, Duane, Ciolkosz, Edward, and Drohan, Patrick. *The Pleistocene Record in the Middle Susquehanna River Basin and the Longer Term Evolution of the Susquehanna Basin Landscape*. Field trip guidebook for the Natural Resources Conservation Service Soil Geomorphology Institute at Pennsylvania State University, October 25, 2008.

3. Faill, R. T. Geology and Mineral Resources of the Montoursville South and Muncy Quadrangles and Part of the Hughesville Quadrangle, Lycoming, Northumberland, and Montour Counties, Pennsylvania. Atlas 144ab. Pennsylvania Geological Survey, 4th ser., 1979.

4. Faill, R. T. *Paleozoic*. Chapter 33 in *The Geology of Pennsylvania*. Special Publication 1. Pennsylvania Geological Survey, 4th ser. and Pittsburgh Geological Society, 1999.

5. Sevon, W. D., and Braun, D. D. *Glacial Deposits of Pennsylvania, 2nd ed.* Map 59. Pennsylvania Geological Survey, 4th ser., 1997, scale 1:2,000,000. (Available online at: http://www.dcnr.state.pa.us/topogeo/maps/map59.pdf. Accessed May 8, 2011).

6. Burch, J. S. Using 3DM Analyst Mine Mapping Suite for Rock Face Characterisation. In Laser and Photogrammetric Methods for Rock Face Characterization, Report on a workshop held June 17-18, 2006 in Golden, Colorado in Conjunction with Golden Rocks 2006, The 41st U.S. Rock Mechanics Symposium, Colorado School of Mines, June 17 - 21, 2006, American Rock Mechanics Association, May 2007, pp. 14-32.

Landslide and Rock Slope Movement Evaluation using Surface Seismic Methods – Some Case Studies

Michael L. Rucker, P.E.
AMEC Environmental & Infrastructure, Inc.
1405 West Auto Drive
Tempe, AZ 85284
(480) 940-2320
michael.rucker@amec.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to thank individuals for their contributions in the work described:

Richard Bansberg, P.G. – AMEC Environmental & Infrastructure, Inc.
David E. Peterson, P.G. – AMEC Environmental & Infrastructure, Inc.
Nick Priznar, P.G. – Arizona Department of Transportation
James Wilson, P.E. – Arizona Department of Transportation
Pancho Garza, P.E. – AECOM, Inc.
Wayne Harrison, P.G., C.H.G., C.E.G. – AECOM, Inc.

Disclaimer

Statements and views presented in this paper are strictly those of the author, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systesm – without prior written permission of the HGS. This excludes the original author.

ABSTRACT

In 30 years of using surface seismic methods to support geotechnical investigations, the author has seen several slope failures that occurred during highway construction in areas of low seismic velocities. In general, zones of atypically low seismic velocities indicated the presence of relatively weak geo-materials zones where slope failures later occurred. These situations, where in-situ characterization information was obtained prior to failure, provide an opportunity to learn through experience about slope failure potential. In other situations, initial seismic results indicating potential problem zones have been confirmed through coring during site characterization. Compression wave (p-wave) refraction has been the primary surface seismic method applied, with 12 channel technology with sledgehammer energy source available for geotechnical work since the 1980s. More recently, surface wave technology has become available to assess shear wave (s-wave) profiles. Since 2002, the author has used a combination of both Refraction Microtremor (ReMi) for s-wave and standard seismic refraction for p-wave, using the same field seismograph and geophone array setup, as a standard characterization tool. Strengths and weaknesses of each method are complementary. The inability of seismic refraction to characterize a velocity reversal (lower velocity underlying higher velocity) is overcome by ReMi's ability to make that characterization; this is of special value when a weaker, lower velocity horizon may be an indicator of a slope failure condition. On the other hand, ReMi's inherent constraint as a 1-dimensional vertical s-wave profiling method is supported by seismic refraction's more detailed 2-dimensional (vertical and lateral) p-wave profiling capability that can provide interpretation constraints to ReMi's otherwise non-unique interpretation solutions. Several case studies are presented.

INTRODUCTION

As part of the geotechnical investigation for widening several sections of a rural highway east of Payson, Arizona in the late 1990s, the author completed a number of surface seismic refraction lines to assist in characterizing rock mass excavatability and strength at proposed rock cuts for new highway alignment. The work was straightforward, using a 12-channel seismograph with a sledgehammer energy source. Seismic velocities in the typically highly weathered, fractured granites were frequently less than 5,000 feet per second (f/s), and were commonly in the range of 3,500 to 4,000 f/s. One section included a significant cut in weathered, fractured Pre-Cambrian granites with a major transmission line tower near its crest (Rucker, 2010). The slope had been slightly over-steepened to accommodate offset distance from that tower. Seismic velocities in the cut zone (Rucker, 2000) were 5,000 feet per second (f/s) or less to depths of about 23 to 50 feet before increasing so that blasting was needed. Corresponding rock core indicated low rock quality designation (RQD) rock in that shallow zone and higher RQD rock below that shallow zone (Figure 1). During construction in 2000, initiation of slope failure was observed at the tower, with ground cracking extending between the four tower legs. A combination of blast vibrations and a leaking water line at the top of slope may have contributed to the initiation of movement, which continued over time. A temporary tower was installed farther from the slope crest, and rapid slope re-engineering and modification was completed for a permanent solution (Figure 1). A pre-construction in-situ p-wave velocity of about 5,000 f/s or less correlated to the failure of a rock cut slope during construction.


Figure 1. Recovered core at a rock cut where failure initiated in the slightly over-steepened slope. A seismic velocity of about 5,000 f/s was interpreted in the zone of the shallower low RQD core (above top). The seismic velocity of about 9,000 f/s was interpreted in the zone of the deeper higher RQD core (above bottom). The re-designed and reconstructed slope with the temporary transmission line tower is shown on the right.

The 2000 slope failure in weathered granites was the second case of a large highway rock slope failure during construction at a location where the author had previously obtained refraction seismic information. The first case was in 1996 on a rural highway southeast of Prescott Arizona (Figure 2). A 100+ foot high cut was constructed in slightly to highly weathered Pre-Cambrian schist interbedded with phyllite including zones of highly weathered to decomposed rock. The rock mass in a nearby boring had predominantly high angle foliations and fractures with RQD averaging 66 but ranging from 0 to 100. The mountain slope was generally too rugged for access by a vehicle-mounted drill rig, and seismic refraction was not part of the 1993 design work. The author performed seismic lines across parts of the proposed cut for a contractor during bidding prior to construction. One half of a 300-foot long seismic line was completed across the future failure zone, and a p-wave velocity of 4,800 f/s was interpreted to a depth as great as 52 feet. Below the surface soil and decomposed rock horizon, other p-wave velocities in adjacent seismic lines ranged from 6,500 to 9,500 f/s.

Figure 2. Rock slope failure (left) during construction and new benched cut post-construction.

These examples of rock slope cut failures are discussed in greater detail in Rucker (2010). This paper addresses recent work using a combination of two surface seismic methods to possibly improve characterization of slope failure and landslide conditions.

METHODOLOGY

Great strides in small-scale shallow surface seismic technology, equipment, and software have been made in recent decades. However, successful application of surface seismic methods for geotechnical characterization requires an understanding of and appreciation for the data needs and applications of the engineering geologist / geotechnical engineer as well as proficiency with the geophysical methods.

'Traditional' Seismic Refraction for Geotechnical Engineering

Standard shallow compression wave (p-wave) seismic refraction using at least a 12-channel seismograph with sledgehammer energy source has been an available tool for decades; the author began using such equipment for geotechnical investigations in 1981. Simple depth-velocity interpretations based on intercept time method (ITM) were calculated first using calculators and later spreadsheets. Software to automate parts of the interpretation process gradually became available. Interpretation results are commonly 2-dimensional profiles of the subsurface with up to perhaps 3 layers of increasingly higher velocity (more competent) geo-materials. A common subsurface profile could include a soil horizon, a highly weathered and fractured rock horizon and more competent bedrock horizon.

Figure 3. Example field setups showing seismic equipment at a possible slide impacting transmission line tower foundation (left) and road widening on a rock cut and fill slope in Sedona, Arizona (right).

By obtaining lateral profile as well as vertical depth information, seismic refraction results provide valuable geotechnical information for rock cut investigation that is complementary to vertical borings. Hand carrying the seismic equipment allows seismic refraction information to be collected in rugged terrain such as proposed rock cut locations where drill rig access is limited or not feasible. Because seismic velocity is strongly related to geo-material strength and modulus, it has become the preferred method to assess rock excavatability, primarily through the publication of rippability charts for specific sizes of equipment (Cat, 1984; 1993). Such information is typically needed for rock cut characterization. However, seismic refraction has limitations. The groundwater table profoundly impacts p-wave seismic velocity results; refraction can be used to detect the water table in soil environments, but may not be able to distinguish between water table and relatively soft rock, or provide a useful p-wave velocity to characterize material strength below the water table. A critical limitation is the inability to detect a lower velocity horizon underlying a higher velocity horizon (velocity reversal). The physics of refraction require that horizon velocities increase with depth to be effectively interpreted. That assumption is violated in a velocity reversal condition. In a rock slope failure or landslide setting, such a low velocity horizon might be a critical failure zone, or might provide information of the potential or geometry of lower strength material zones. The previously described cases of rock slope failures in ground with in-situ seismic velocities of 5,000 f/s or less are examples of seismic velocity identifying zones of lower strength rock mass in rock cuts.

Refraction Microtremor (ReMi) Surface Wave Method

Surface Rayleigh waves propagate at slightly slower velocities than shear waves, and thus can be used to estimate shear wave (s-wave) velocities. Seismic surface (Rayleigh) wave methods, including ReMi (Louie, 2001; Optim, 2004), utilize the physics of surface wave dispersion to characterize the subsurface as a vertical 1-dimensional s-wave profile. Overlapping or adjacent profiles can be performed to obtain 2-dimensional subsurface profiles. The interpretation process leads to non-unique solutions, so that results tend to be less precise than seismic refraction. However, the non-unique solutions can be constrained when other subsurface information is available. The nature of surface wave methods brings several valuable attributes to surface seismic characterization. For a given geophone array length, the ReMi depth of investigation tends to be greater than for seismic refraction. Surface wave methods can characterize the subsurface profile below velocity reversals and below the water table where refraction cannot or is severely limited. ReMi can also be used at noisy environments such as traffic where ambient ground vibrations interfere with or overwhelm seismic refraction signals.

Seismic Characterization using Combined Seismic Refraction and ReMi

Since ReMi became available in 2002 the author has used a combination of seismic refraction and ReMi as a standard characterization tool in geotechnical investigations, including rock cut investigations. The same geophone array setups can be used to perform both methods using low frequency (4.5 Hz) geophones at typically 10-foot spacing, a 12- or 24-channel seismograph, and sledgehammer (p-wave) and ambient noise or person jumping (s-wave) energy sources. Higher frequency seismic refraction data and low frequency ReMi data require different seismograph data collection settings. Rucker (2006) describes how the strengths and weaknesses of the two surface seismic methods are complementary. More complete and robust characterization can be obtained by combining seismic refraction and ReMi interpretations at the same location than with either method alone.

An example combined seismic refraction and ReMi interpretation at rock slope with recent slope failures is presented in Figure 4. Strengths and limitation of each method are apparent in this interpretation. The p-wave seismic refraction interpretation provides a 2-dimensional interpretation of 2 to 3 horizons of subsurface materials. The surficial soil horizon ranges in depth from about 1 to 5 feet across the 240-foot long profile. An underlying highly weathered to decomposed rock horizon has a range of p-wave velocities (numbers in black) of 2,300 to 3,100 f/s. The variability in p-wave velocity indicates variability in the geo-material mass strength across the rock cut. A deeper, higher p-wave velocity near the west end of the profile indicates a zone of more competent rock; that rock can be seen in the accompanying photo. The interpreted p-wave refraction depth of investigation is only about 20 feet across the profile. Only the upper portion of the rock cut is characterized by the p-wave profile. The ReMi profile is a 1-dimensional characterization of the rock cut slope. A thin low-velocity horizon and a higher s-wave velocity zone deeper in the subsurface is interpreted in the ReMi results at depths greater than the interpreted p-wave depth of investigation. This combined seismic interpretation will be discussed in greater detail shortly.

CASE STUDY – CUT SLOPE FAILURE IN WEATHERED FRACTURED GRANITES

A few miles east of the 2000 rock failure shown in Figure 1, construction is progressing (in 2011) on an adjacent section of the rural highway east of Payson, Arizona. The original geotechnical investigation, with some seismic lines, was completed in 2000. Seismic p-wave velocities were less than 5,000 f/s at several cut locations; at design slopes of 1.25 to 1 (horizontal to vertical), these cuts have performed well during construction. One cut (Figures 4 through 6) was designed and constructed at 0.5 to 1; progressive slope failures in the spring of 2011 have been occurring there. Seismic refraction Line LGV-16, completed in 1998, crossed this cut over an aplite dike. P-wave velocities in the upper portion of the cut at LGV-16 were about 4,500 f/s; this section of the cut has not failed. P-wave velocities in the highly weathered granites at Line LGV-16 and another nearby seismic line ranged from about 3,400 to 4,900 f/s.

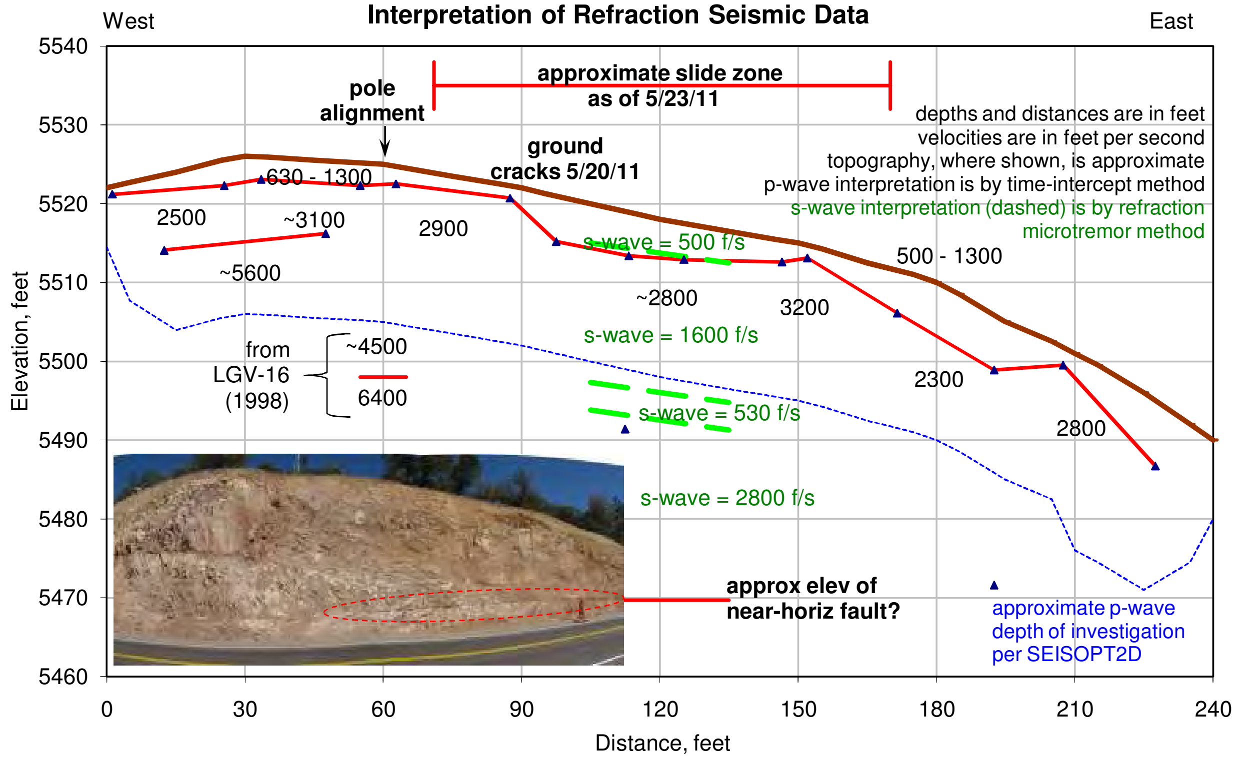


Figure 4. Seismic refraction and refraction microtremor interpretation of Line 1 at the top of a rock cut under construction in weathered fractured granite and aplite. Results from this line are incorporated into the subsurface cross-section profile presented in Figure 6.

Figure 5. Progressive spring 2011 slope failures at 0.5 to 1 slope in weathered fractured granite and aplite dike cut.

Deeper, higher velocity rock at LGV-16, with a p-wave velocity of about 6,400 f/s, was interpreted at an interface that was dipping slightly and becoming shallower downslope. Core from geotechnical boring B-16 (completed in 1999), perhaps about 30 feet downslope from the current top of cut and approximately centered at LGV-16, had RQD of zero to 10 throughout the aplite dike to the total depth of 50 feet. Core pieces were highly weathered and moderately soft to moderately hard at depth shallower than 14 feet; this corresponded to the p-wave velocity of 4,500 f/s. Below a depth of 14 feet, the core pieces were moderately weathered with occasional highly weathered zones and hard to very hard; this corresponded to the p-wave velocity of 6,400 f/s. At that time, seismic results were used primarily to characterize major soil, weathered rock, bedrock subsurface interfaces and assess excavation conditions. ReMi technology was not yet available.

Current Cut Slope Profile Evaluation

In addition to observation and measurement of relative movement across a nearly horizontal thin clay-filled fault in the lower portion of the cut, further evaluation at the problem cut using surface seismic was performed on May 20, 2011. Three 240-foot long 24-geophone seismic lines roughly parallel to the rock cut crest were completed above the cut. Line 1 (Figure 4) was about 30 feet upslope of the original top of cut, Line 2 was about 50 feet upslope from Line 1, and Line 3 was about 50 feet upslope from Line 2. Line 1 was close to visible ground cracks about 10 to 40 feet west of its' center. The westernmost portions of the seismic lines were over the weathered, fractured aplite dike, and the rest of the lines were over weathered, fractured granite. Approximate topography and elevations are shown in the seismic interpretations; a conceptual cross-section through the cut and seismic line centers summarizing seismic results is presented as Figure 6.

Current slope failures were observed in the highly weathered granites present at the center portion of Line 1. Weathered granites were assumed to continue upslope through the eastern portion of Lines 2 and 3. Below the surficial soils, typical p-wave velocities in the highly weathered granites at Line 1 ranged from 2,300 to 3,200 f/s, at Line 2 ranged from 3,700 to 4,700 f/s, and at Line 3 ranged from 3,300 to 5,700 f/s. Interpreted representative s-wave velocities in these areas ranged from about 1,600 to 1,700 f/s. Higher p-wave velocities were interpreted at typical depths below 30 feet at Line 2 and below 15 to 20 feet at Line 3. A range of s-wave velocities and depths were interpreted in the lower portions the seismic lines; these interpretations will be discussed shortly.

The aplite dike was interpreted as a zone of relatively higher seismic velocities typically in the western portion of the lines, especially in Line 1. Typical p-wave velocities in the shallow portion of the dike ranged from about 2,500 to 3,100 f/s at Line 1 nearest the cut to about 3,500 to 4,200 f/s at Line 2, to 4,200 to 5,800 f/s at Line 3 farthest from the cut. In the deeper portion of the dike, p-wave velocities ranged from about 5,500 to 5,600 f/s at Lines 1 and 2 up to 9,200 f/s at Line 3, to the interpreted p-wave depths of investigation. ReMi s-wave velocities were assumed to be controlled by the lower velocity weathered granites in the center to eastern portions of the lines, and not typically applicable to the aplite dike zone.

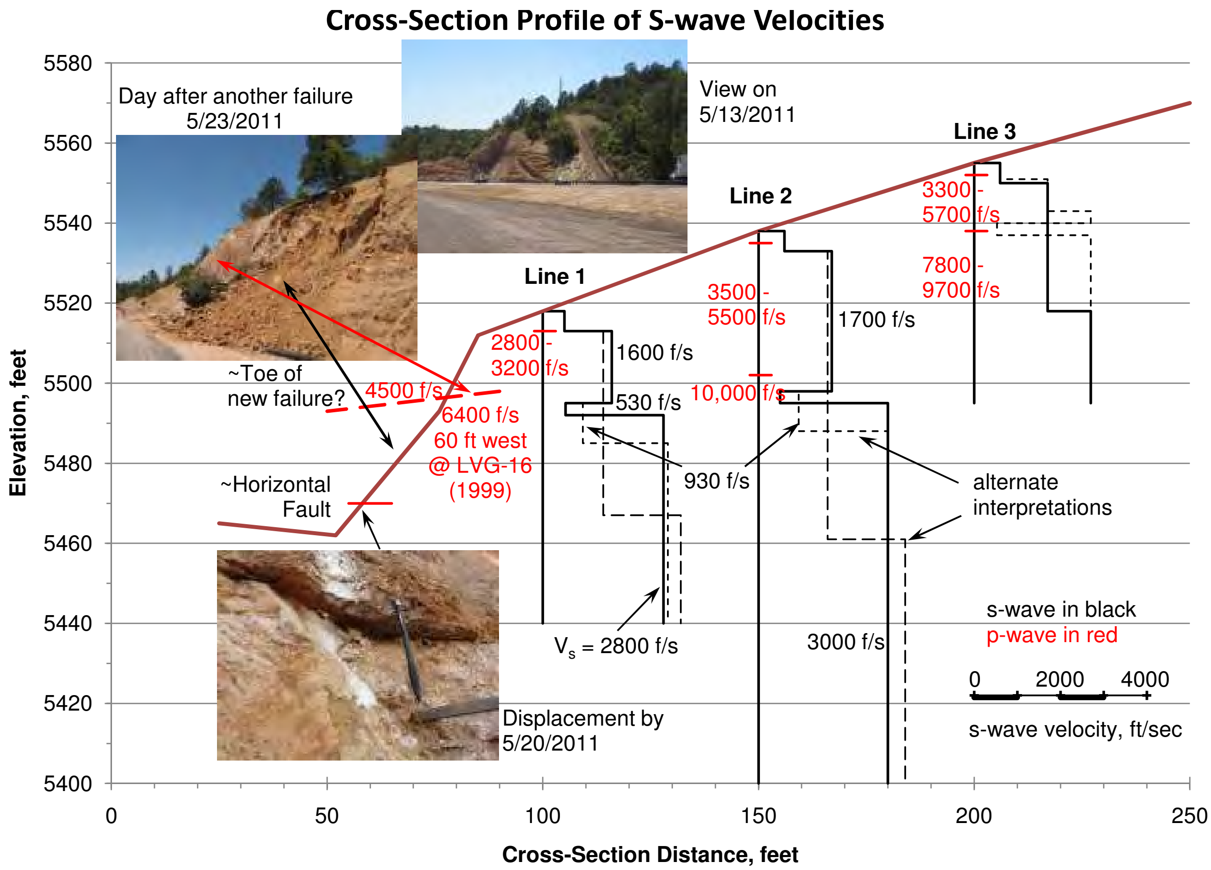


Figure 6. Cross-section perpendicular to cut summarizing interpretations at seismic lines. ReMi interpretations are non-unique; several alternative s-wave velocity profiles are shown.

S-wave Velocity Profiles

Surface wave methods such as ReMi used to derive vertical s-wave profiles result in interpretations that are non-unique solutions. A range of depths and velocities, and various combinations of horizons, can result in similar solutions that provide good matches to dispersion curves derived from processed results of field measurements during the interpretation process. However, the ability to identify a lower velocity horizon underlying a higher velocity horizon (velocity reversal) and superior depth of investigation complements seismic refraction and helps overcome p-wave seismic refraction constraints and limitations. P-wave results provide shallow interface depths and estimates of shallow s-wave velocities to partially constrain the s-wave interpretations.

A series of alternative s-wave profile interpretations for Lines 1 through 3 are presented in Figure 6. These interpretations were based on s-wave velocity-frequency dispersion curves developed from the ReMi data at the seismic lines; dispersion curves used for interpreting the ReMi s-wave profile at Lines 2 and 3 are presented in Figure 7. ReMi results were derived from measurements at the central 18 geophones; data from the 3 geophones at each line end, where significant topographic changes were present or subsurface changes had been interpreted in the p-wave results, were not used. Shallow constraints on s-wave interpretations were provided by the respective p-wave interpretations. Across a horizon with variable seismic velocities, the

ReMi interpretation tends to be controlled by the lower velocity portion of that horizon. The upper two ReMi s-wave horizons (Figure 6) with s-wave representative s-wave velocities of about 500 to 600 f/s and 1,600 to 1,700 f/s, reflected corresponding p-wave velocities of about 1,000 to 1,200 f/s and 3,200 to 3,400 f/s, respectively.

Below the upper two horizons, the ReMi results became less constrained. Interpreted high velocity p-wave interfaces were present at Lines 2 and 3, and in the historic Line LGV-16 near to Line 1. To be consistent with these p-wave interfaces, corresponding higher velocity horizons were needed in the s-wave profile. Adding only a deep high s-wave velocity horizon, the elevation for the top of that horizon was below 5470 feet in Lines 1 and 2. Corresponding top of high p-wave velocity horizons at Line 2 and LGV-16 near Line 1 were above elevation 5495 feet. To more closely match the s-wave high velocity horizons to the corresponding p-wave horizons, a thin low velocity horizon needed to be inserted into the s-wave profile. Two possible alternative interpretations (Figure 6), with ‘weaker’ horizon thicknesses of 10 and 3 feet, had resulting s-wave velocities of 920 f/s and 530 f/s, respectively. The thinnest (3-foot) horizon was nearest in depth to the p-wave interpretations. Similar results were obtained in comparing s-wave and p-wave profiles at Line 3.

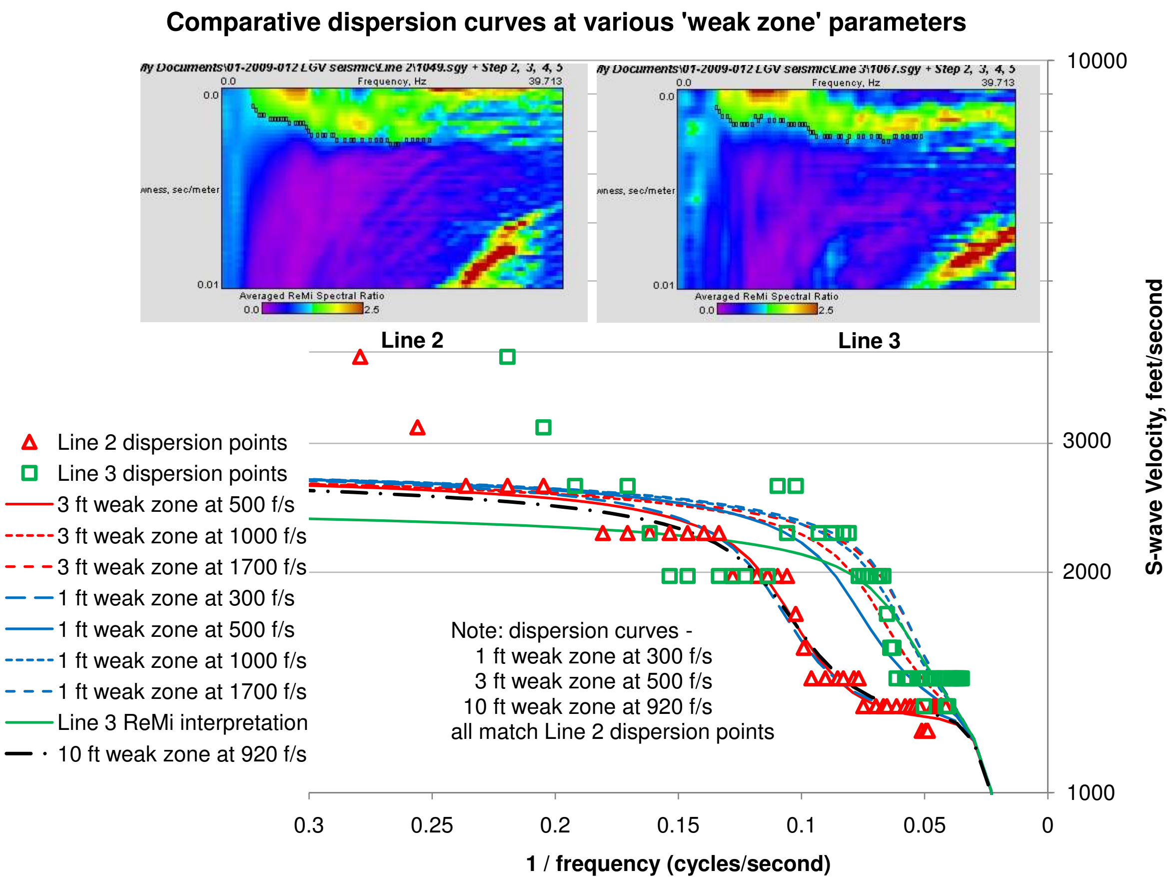


Figure 7. Example dispersion points from Lines 2 and 3 ReMi results and example dispersion curves for s-wave profile interpretation. Resulting s-wave depth-velocity interpretations are presented in Figure 6.

To illustrate the impact of various subsurface profile combinations on dispersion curves and non-uniqueness of possible ReMi interpretations, dispersion curves for several thicknesses and s-wave velocities of thin ‘weak’ horizons are presented in Figure 7. Dispersion curves were developed at selected ‘weak’ horizon thicknesses at several s-wave velocities. Very similar dispersion curves effectively matching the Line 2 dispersion points were developed using s-wave velocities of 300, 500 and 920 f/s at respective ‘weak’ horizon thicknesses of 1, 3 and 10 feet. Without other constraints, a range of interpretations where both the ‘weak’ horizon thickness and s-wave velocity decrease, as represented in Figure 7, could provide equally reasonable results.

Observable geologic features shown in Figure 6 could be consistent with either extreme of the s-wave interpretation options. S-wave profiles with no low velocity ‘weak’ horizon resulted in a top of high s-wave velocity horizon consistent with the near horizontal fault observed near the toe of the rock cut. However, an interpreted high velocity p-wave horizon had to be ignored. Constraining the deeper s-wave interpretation with p-wave interpretation results assumed that the influence of the thin horizon on the p-wave results was not significant. Yet, the toe of the most recent observed local failure in the slope appeared to be at about the elevation of the interpreted thin low velocity ‘weak’ horizon well above the near horizontal fault.

GEO-MATERIAL PARAMETERS ESTIMATION FROM SEISMIC VELOCITY

Two relevant parameters for slope stability analysis, unit weight and apparent cohesion (from unconfined compressive strength), can be estimated from seismic velocity as shown in Figure 8 (Rucker, 2008). Using representative s-wave velocities of 1,600 and 3,000 f/s for the upper and lower portion of the cut slope, approximate unit weights of 121 and 132 pounds per cubic foot (pcf) are estimated for the upper and lower portions of the cut.

A minimum unconfined compressive strength for a geo-material mass can be estimated from seismic velocity. The geo-material mass is assumed to be unfractured (rock quality designation RQD of 100) so that fracturing does not impact seismic velocity. Seismic velocity is then related to the modulus of the unfractured geo-material mass, and modulus is then related to unconfined compressive strength (UCS) as described by Rucker (2008). Figure 9 plots the resulting UCS-seismic velocity relationship. Since field seismic velocity is reduced by in situ discontinuities that separate drilling core sample pieces, results of core sample testing plot above the UCS-seismic velocity trend; estimating UCS from seismic velocity is conservative. Using representative s-wave velocities of 1,600, 3,000 and 500 f/s for the upper, lower and thin ‘weak horizon’ portions of the cut slope, approximate UCS of 60, 400 and 2 pounds per square inch (psi) are estimated for the upper, lower and thin ‘weak horizon’ portions of the cut.

Assuming an undrained shear strength condition, apparent cohesion can be estimated as one-half of unconfined compressive strength. Apparent cohesion may only be relevant where the geo-material mass can be anticipated to behave as a soil mass; a rock mass might be anticipated to fail along discrete fracture or other discontinuity planes rather than as a material mass. For representative s-wave velocities of 1,600 and 500 f/s for the upper highly weathered granite and thin ‘weak horizon’ decomposed granite portions of the cut slope, approximate apparent cohesion of 4300 and 140 pounds per square foot (psf) are estimated for the upper and thin ‘weak horizon’ portions of the cut.

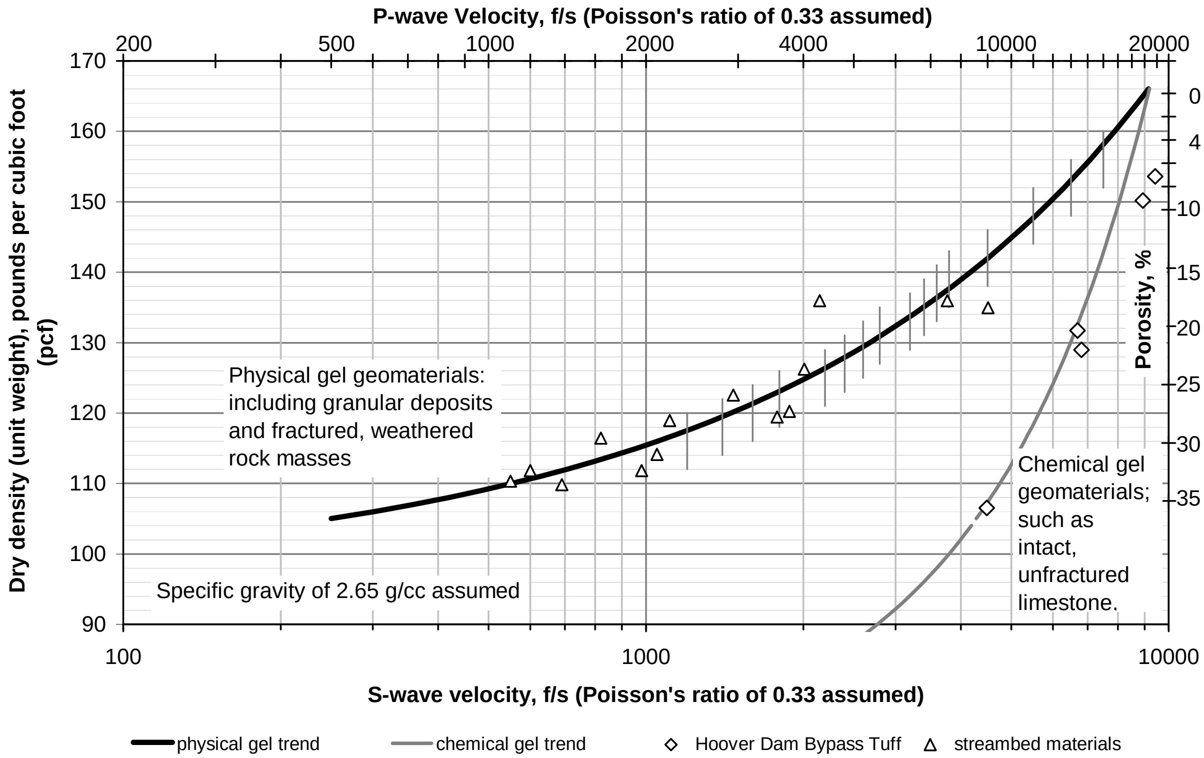


Figure 8. Example relationships between seismic velocities at a Poisson's ratio of 0.33 and unit weights or dry densities and porosities are shown at a particle specific gravity of 2.65 g/cc (Rucker, 2008). The physical gel trend should be used for weathered, fractured granite.

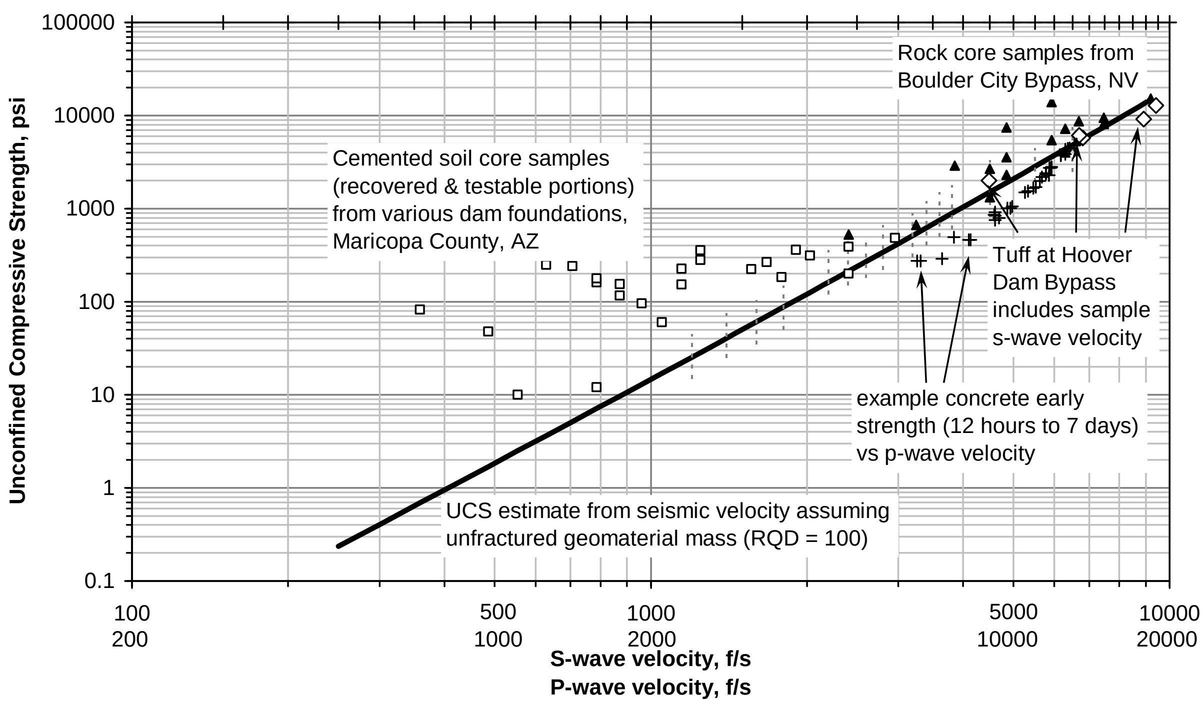


Figure 9. A relationship between seismic velocity and minimum unconfined compressive strength is developed through relationships of low strain to high strain modulus derived from seismic velocities and static UCS testing (Rucker, 2008).

Where a geo-material mass tends to be behave in a rock-like manner rather than a soil-like manner, the concept of apparent cohesion is no longer valid; a rock mechanics approach to geo-material parameters becomes appropriate. Barton (2007) summarizes studies of p-wave velocity compared to UCS and RQD. Rock mass discontinuities significantly reduce seismic velocity. As summarized by Barton, RQD has been related to intact laboratory and field seismic velocity through a concept of velocity ratio (Deere and others, 1967), where

$$RQD = 100 * V_{\mathbf{field}}^{\mathbf{2}} / V_{\mathbf{intact}}^{\mathbf{2}}$$

RQD is estimated as a percentage using field and laboratory p-wave velocities. RQD may be roughly approximated from interpreted p-wave velocities measured above the groundwater table.

CASE STUDY – LANDSLIDE IMPACTING A RURAL HIGHWAY

A major landslide (Diaz and others, 2008) closed a rural highway between Phoenix and Payson, Arizona in the spring of 2008. The highway was reopened after temporary repairs, and an extensive investigation, design and mitigation, including buttress fill and multiple lines of shear piles, has been completed. Monitoring of the mitigated slide zone by inclinometers and survey is continuing.

Figure 10. Landslide on SR87 east of Sunflower, AZ in March 2008 (left) and ReMi line (right) performed in May 2011. Landslide photo courtesy of AzGS (2011).

The author completed a ReMi line on the mitigated slide on May 20, 2011. A 230-foot long 24-geophone array was deployed parallel to and about 75 feet west of the highway along an edge of the buttress fill at elevation 4210 feet. Surface wave energy source location was a concern at this site; the ReMi method assumes that surface wave energy used in the interpretation is propagating in a direction parallel to sub-parallel to the geophone array. Surface wave energy that propagates perpendicular to sub-perpendicular to the geophone array cannot be used for ReMi interpretations, and may be of sufficient magnitude to mask or overwhelm appropriate ambient surface wave energy. The geophone array was parallel to the highway, and the primary ambient ground noise was highway traffic. Highway traffic that drove past the geophone array during data collection generated the largest signal while closest to the array.

Unfortunately, that was also when an individual vehicle ambient energy source was located perpendicular to the geophone array. Undesired vehicle noise was a significant concern during data collection.

ReMi data was acquired in a manner that utilized two kinds of surface wave energy. Higher frequency ReMi data was acquired using a person jumping at the center of the geophone array to provide properly oriented higher frequency ambient surface wave energy. Data sets were collected at 1 millisecond sampling rates for 12 seconds when vehicles were primarily beyond the ends of array. Lower frequency ReMi was acquired by waiting for heavy trucks to pass by the array before triggering data collection. Data sets were collected at 2 millisecond sampling rates for 24 seconds. Seismic refraction p-wave data was collected at the geophone array center to provide shallow subsurface profile constraints for the s-wave interpretation.

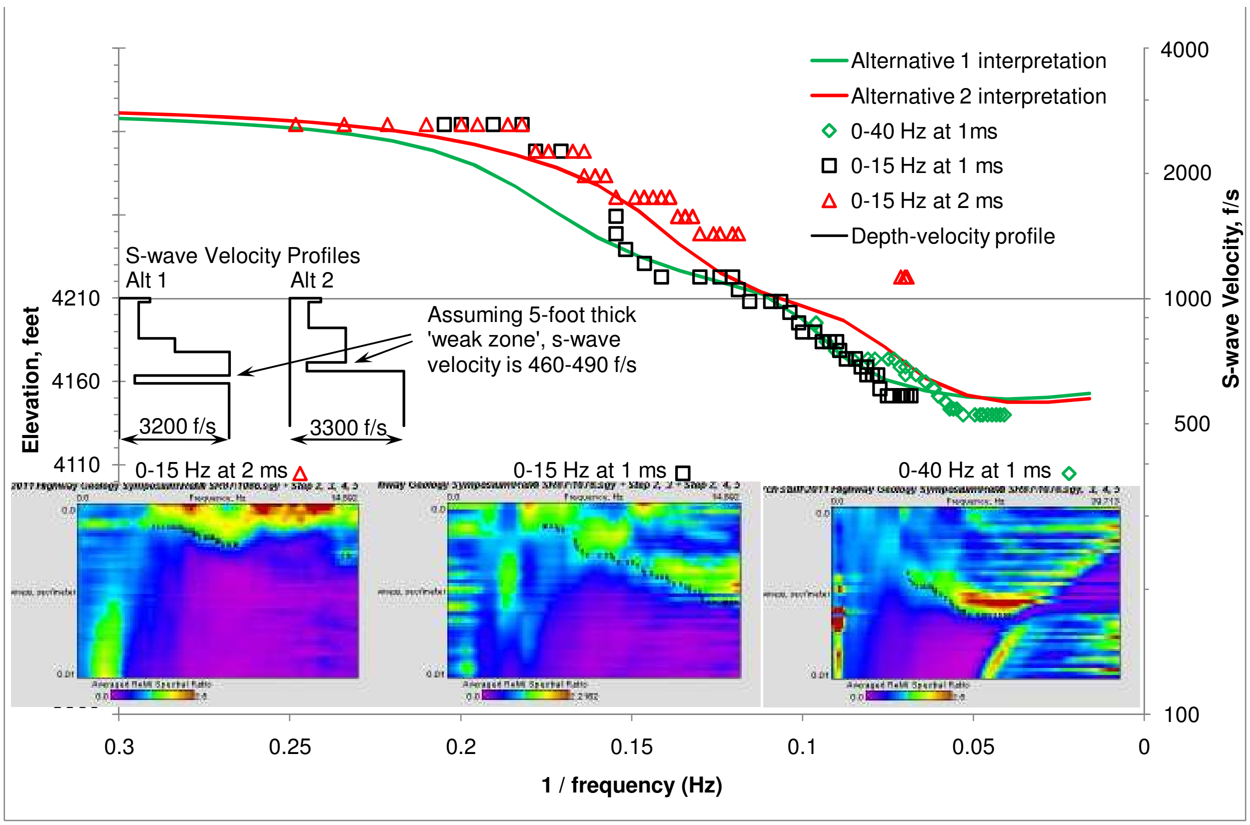


Figure 11. Results and interpretations for the ReMi line completed at landslide in Figure 10.

Processed results from the ReMi data collection effort and interpretations are presented in Figure 11. The 1 millisecond data was processed in frequency bands of 0 to 40 Hz for shallow results and 0 to 15 Hz for intermediate subsurface depth results. The 2 millisecond data was processed at 0 to 15 Hz for deep subsurface results. Two alternative non-unique s-wave profile interpretations utilizng all three processed data sets are presented in Figure 11. Assuming the presence of a 5-foot thick 'weak zone' in the subsurface profile, such a zone was interpreted to be present in a depth range of about 40 to 50 feet below that ground surface. Interpreted s-wave velocities in that 'weak zone' ranged from about 460 to 490 f/s. Nearby rows of shear piles installed as part of the mitigation extend to depths of 65 to 73 feet.

CONCLUSION

Being able to characterize geo-material mass strength through seismic velocity, surface seismic methods may be an effective tool to identify or evaluate potential landslide or slope failure conditions. As with all geotechnical tools, surface seismic methods have advantages and limitations. Surface seismic results are part of a larger body of knowledge, including remote sensing, geologic mapping, subsurface exploratory borings and test pits, and ground measurements and monitoring, needed to understand difficult site conditions. As with all tools, increasing experience and gained knowledge in applying these methods will enhance their value and contribution to solving problems of landslides and slope failures.

REFERENCES:

Arizona Geological Survey (AzGS), 2011, http://www.azgs.gov/hazard%Resources%202008/Hwy87%20Landslide%20Mar08/image5_b.jpg, accessed May 29.

Barton, N., 2007, *Rock Quality, Seismic Velocity, Attenuation and Anisotropy*, Taylor & Francis/Balkema, London, UK, pp. 25-27.

Caterpillar Tractor Company, (Cat), 1984, *Caterpillar Performance Handbook*, Edition 15, Peoria, Illinois, October.

Caterpillar Tractor Company, (Cat), 1993, *Caterpillar Performance Handbook*, Edition 24, Peoria, Illinois, October.

Deere, D.U., Hendron, A.J., Patton, F.D. and Cording, E.J., 1967, Design of surface and near surface construction, In: *Rock, Failure and Breakage of Rock*, Fairhurst, C. (ed.): 237-302. New York, Society of Mining Engineers of AIME.

Diaz, M., Gootee, B.F. and Youberg, A., 2008, *Reconnaissance Report on the Easter Weekend Landslide of 21 March 2008, SR-87, Gila County, Arizona*, Arizona Geological Survey Open-File Report OFR 08-04, November.

Louie, J.L., 2001, Faster, better: Shear-wave velocity to 100 meters depth from refraction microtremor arrays: *Bulletin of the Seismological Society of America*, **91**, 347-364.

Optim, 2004, SeisOpt@ReMi™ Version 3.0, Optim LLC, UNR-MS 174, 1664 N. Virginia St, Reno NV, 89557-0141 USA

Rucker, M.L., 2000, Earthwork factors in weathered granites by geophysics, in Nazarian, S. and Diehl, J. (eds), *Use of Geophysical Methods in Construction, Geotechnical Special Publication No. 108*: ASCE, Reston, Virginia, 201-204.

Rucker, M.L. 2006, Integrating Seismic Refraction and Surface Wave Data Collection and Interpretation for Geotechnical Site Characterization, *FHWAGeophysics2006*, St. Louis, Mo., December 4-7. http://www.optimsoftware.com/white_papers/images/FHWA2006RuckerP-waveS-wave.pdf

Rucker, M.L., 2008, Estimating in-situ geo-material mass density, modulus and unconfined compressive strength from field seismic velocity measurements, *Highway Geophysics-Nondestructive Evaluation Conference*, Charlotte, NC, Dec. 1-4.

Rucker, M.L., 2010, Shall We Use ReMi to Characterize Landslides? – Some Case Studies, Environmental & Engineering Geophysical Society, *FastTIMES* Vol. 15 (2), pp. 33-42.

HORIZONTAL DIRECTIONAL DRILLING FOR EXPLORATION OF LOUISVILLE – SOUTHERN INDIANA RIVER BRIDGES EAST END NORTH AND SOUTH BOUND TUNNELS

CRAIG S. LEE, P.E.
Senior Engineer
S&ME Inc.
422 Codell Drive
Lexington, Kentucky 40509
859.293.5518
clee@smeinc.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The author would like to thank the individuals/entities for their contributions in the work described:

Mr. William Leake, PE, PLS (S&ME)
Mr. Dexter Newman, PE (S&ME)
Mr. Nate Peterson, PG (S&ME)
Mr. David Durman (S&ME)
Mr. Jerry Baber, EI (S&ME)
Mr. Justin Beckham, EI (S&ME)
Mr. York Little, EI (S&ME)
Mr. George Small (Tech Directional)
Mr. Daryl Greer, PE (KYTC)
Mr. Bart Asher, PE (KYTC)
Mr. Gary Valentine, PE (KYTC)
Mr. Andy Barber, PE (KYTC)

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

The Lexington office of S&ME Inc. is performing horizontal directional core drilling, conventional vertical drilling, and surface and borehole geophysics to support the Geotechnical Baseline Report for the Louisville-Southern Indiana Ohio River Bridges East End Tunnel project. This innovative approach to obtaining the geotechnical data for the tunnel design has saved taxpayers millions of dollars compared to the cost of an exploratory tunnel. In addition to costs, the S&ME approach has been far less disruptive to the adjacent residential developments. This project represents the longest known use of this technology for a highway tunnel in the Unites States.

HORIZONTAL DIRECTIONAL DRILLING FOR EXPLORATION OF LOUISVILLE – SOUTHERN INDIANA RIVER BRIDGES EAST END NORTH AND SOUTH BOUND TUNNELS

CRAIG S. LEE, P.E.
Senior Engineer
S&ME INC.
422 Codell Drive
Lexington, Kentucky 40509
859.293.5518
clee@smeinc.com

PROJECT DESCRIPTION

The Louisville-Southern Indiana Ohio River Bridges Project is a "priority" national transportation project which addresses long-term, cross-river transportation needs in Louisville, Kentucky and Southern Indiana. It is one of the largest transportation projects in the country and will result in safer travel, less congestion and improved access to destinations in the region. According to the project web site (kyinbridges.com) the estimated cost for the project is $4.083 billion, based on projected year-of expenditure dollars, which takes into account inflation. This cost reflects updated estimates prepared in 2010 in the **Financial Plan Update**. Ongoing cost-savings efforts now involve the exploration of potential options that could reduce the overall cost by more than $500 million. The target date for construction to begin is August, 2012.

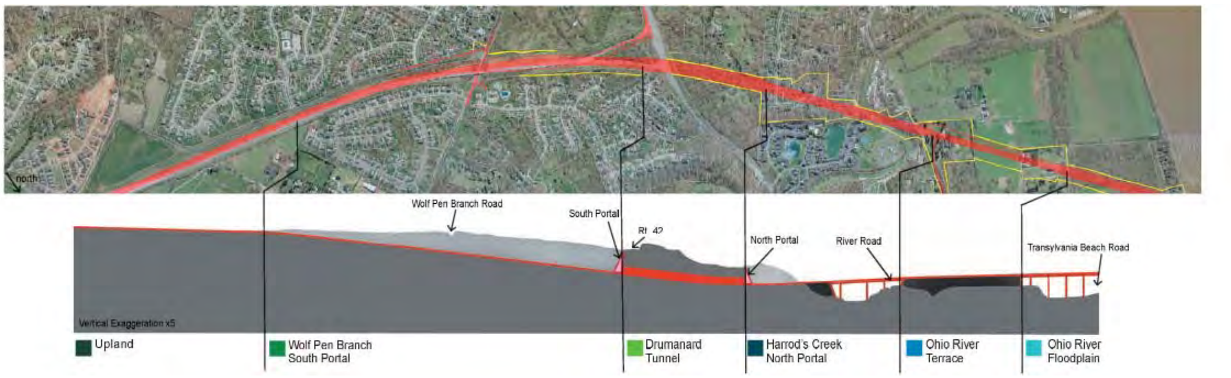
The overall project consists of six segments:

1. Kennedy Interchange
2. New Downtown Bridge
3. Downtown Indiana Approach
4. East End River Bridge
5. Kentucky East End Approach
6. Indiana East End Approach

The tunnel project is part of the Kentucky East End Approach segment.

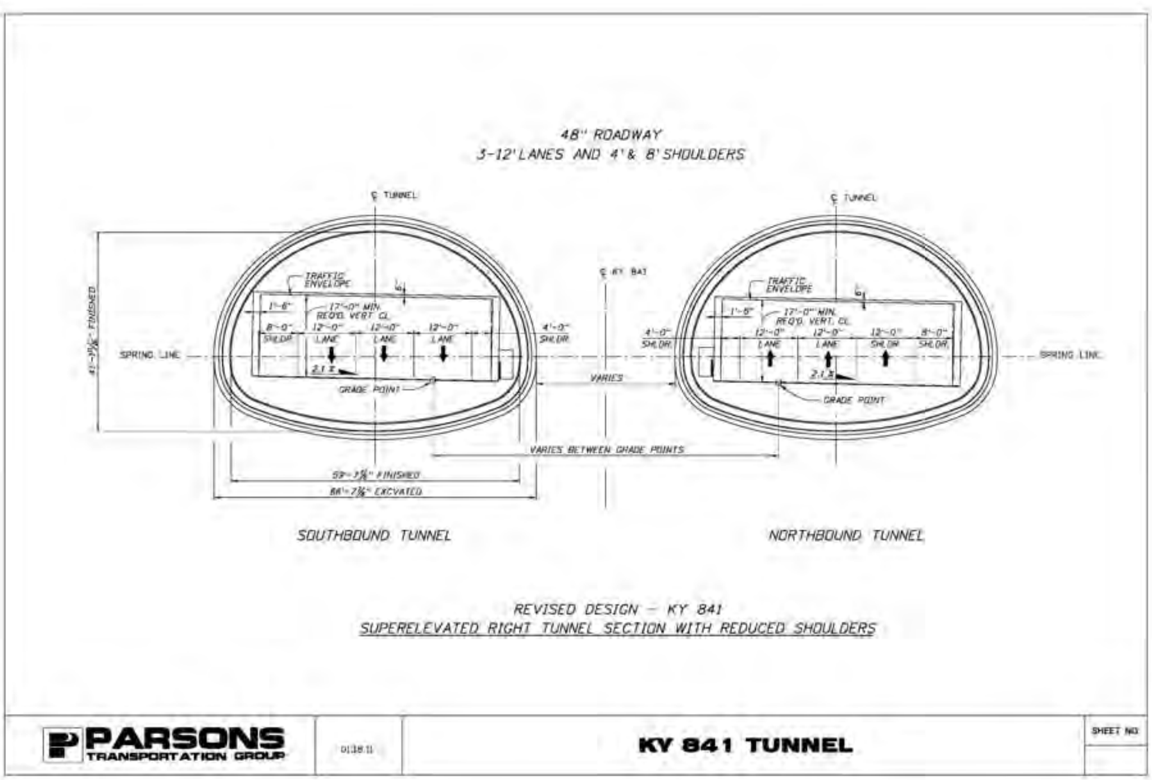
BACKGROUND OF TUNNEL

The original design of the I-265 extension proposed a conventional open cut roadway through the hillside that includes the Drumanard Estate. The Drumanard Estate was recently placed in the National Registry of Historic Places and must be preserved. This forced the alignment underground into twin tunnels, a northbound and a southbound tunnel. The conceptual profile is shown in the graphic below taken from the Louisville Bridges web site.



Graphic from kyinbridges.com

As of this date, the tunnels have an inside finished width of approximately 60 feet with an inside finished height of approximately 41 feet. The typical section is shown below along with a computerized rendering of the tunnel also from the Louisville Bridges website.



Graphic from kyinbridges.com

Originally, a 12 foot diameter exploratory tunnel was designed for the southbound lane. The original tunnel designer requested the exploratory tunnel to evaluate the ground conditions and enable the tunnel designer to develop the ground support conditions. After the cost of the exploratory tunnel significantly exceeded the engineering estimate the exploratory tunnel was scrapped by KYTC.

The Lexington office of S&ME became involved in the exploratory tunnel project through each of the design build teams for the exploratory tunnel. The exploratory tunnel design included a significant instrumentation and horizontal rock coring program as part of the exploratory tunnel. At various point along the exploratory tunnel, the contractor was required to advance a horizontal rock core boring into the adjacent tunnel to explore and test the conditions in the adjacent tunnel. The specifications for the rock cores into the adjacent tunnel required an accuracy of 0.5 percent with the instrumentation installed within one foot of the location shown on the drawings. We identified and teamed with Devico to provide the directional rock coring system that could achieve the specified accuracy of the drill hole location.

When the exploratory tunnel was scrapped, S&ME personnel proposed that KYTC consider using horizontal directional coring to explore the ground conditions along the entire tunnel alignment in lieu of the exploratory tunnel. Over four years, S&ME met with representatives of KYTC, FHWA, Devico, and several underground drilling contractors to advance the reasonableness of this technology to obtain much needed information both for the design of the tunnel and for the Geotechnical Baseline Report. The horizontal directional drilling information was deemed critical for use by the Design Build contractors in managing risks in the bidding process. In the Spring of 2010, S&ME was selected by KYTC to perform the horizontal directional coring and in December 2010 we obtained a contract and notice to proceed.

Our scope is to perform horizontal directional core drilling along the crown of both tunnels and along the pillar between the two tunnels. Each boring is approximately 2400 feet long. The tunnel themselves are only about 2000 feet long, but we need to set our drill rig back from the tunnel face 400 feet to provide room to steer the coreholes from the surface to the tunnel crown. Our preference is to keep the steering to 4 degrees or less. This reduces the friction on the drill tools. In addition to the drilling program, S&ME is performing a comprehensive testing program on the recovered rock core. In the field, Nate Peterson, PG of S&ME logs and photo documents the core. He also produces daily field reports with photographs and a field report for each day. These reports are reviewed by the tunnel consultants. Samples of the recovered core are included at the end of this paper.

S&ME selects representative samples of the recovered core and performs point load testing in the field. In addition, representative samples of the recovered core are sent to our Knoxville rock mechanics laboratory for additional testing including:

- Unconfined Compressive Strength Testing, Method C
- Unconfined Compressive Strength Testing, Method D
- Direct Shear
- Brazilian Split Tensile
- Petrographic Analysis
- Slake Durability
- Porosity/Saturation/Void Ratio

In addition to the horizontal directional drilling, S&ME is contracted by KYTC to perform conventional vertical drilling and surface and borehole geophysics along the tunnels. The vertical drilling will occur after the surface geophysics and at least one of the horizontal directional coreholes are complete. The vertical drilling will include inclined coreholes and seeks to further define anomalies identified in the horizontal holes and the surface geophysics. The laboratory testing program described above will also apply to the vertical borings.

The Devico System used at the Louisville Bridges tunnel job consists of the DeviDrill, the PeeWee tool, and the DeviFlex. The DeviDrill is the steerable corebarrel while both the PeeWee and DeviFlex are used to measure the physical parameters of the borehole. The principle behind the DeviDrill corebarrel is a drive shaft running through a bushing offset from the centre line of the tool. Expanding pads operated by a differential pressure is keeping the DeviDrill in a fixed toolface while drilling in a curve. The inner assembly carries an inner tube collecting the core, a muleshole system, and an instrument barrel with the survey tool recording inclination and tool orientation. Data is stored inside the tool and downloaded wirelessly to a PDA after each run.

The PeeWee is a miniature electronic multishot based on the same reliable technology as the DeviTool Standard. The PeeWee uses three high-accuracy magnetometers and accelerometers. It records inclination, azimuth, toolface, temperature, gravity vector, magnetic field vector, magnetic dip angle, and battery status.

DeviFlex is a non- magnetic electronic multishot for surveying inside casings and drill strings by simply using the wireline system. The DeviFlex is less prone to magnetic disturbances. The DeviFlex tool consists of two independent measuring systems. Three accelerometers and four strain gauges are used to calculate inclination and change in azimuth. In addition, the DeviFlex records and stores gravity vector, temperature, and battery capacity

GEOLOGIC SETTING

The Louisville Bridges Twin Tunnels will encounter three rock formations along the alignment. The Silurian aged Louisville Limestone is the uppermost formation at the project site and is comprised of soluble limestone. The Louisville Limestone is mostly thin-bedded gray dolomitic limestone and gray calcitic dolomite, commonly in lumpy or irregular beds. Shale, in partings and very thin beds, constitutes a few percent, and very sparse chert is present in nodules and thin layers. In the project site, the Louisville Limestone is finely crystalline calcitic dolomite; the sparse fossils are dolomitized and include crinoid columnals, brachiopods, horn corals, and colonial corals.

From an engineering perspective, the Louisville Limestone is characterized by solution enlarged joints and bedding planes. Deep weathering and sinkhole formation are common. The primary impact for conventional building and roadway construction is the presence of latent drop-outs and a highly variable top of rock profile. The residuum derived from the Louisville Limestone is predominantly fat clay with limestone slabs and can exhibit problematic shrink and swell characteristics. For the tunnel, the Louisville Limestone presents several potential problems most associated with the discontinuities such as solution enlarged joints (both horizontal and vertical), solutioning along bedding planes, voids, and sinkholes. The Louisville Limestone can also produce significant groundwater flows after rain events. Water flow is largely along open joints, fractures and bedding planes.

The Waldron Shale is immediately below the Louisville Limestone. In the project site, the Waldron Shale is about xx feet thick. The Waldron Shale is composed of greenish-gray shale and minor gray dolomite; probably at least 95 percent is shale. The shale is dolomitic and weathers with angular fracture or crude fissility, eventually producing a plastic clay. The dolomite is clayey and occurs in irregular masses, lumps, and thin discontinuous beds. Fossils, which are sparse in both the shale and the dolomite, include brachiopods, crinoid columnals, gastropods, and bryozoans. At the tunnel site, the Waldron Shale ranges in thickness from 9 to 15 feet The basal contact with the underlying Laurel Dolomite is conformable and sharp.

The Waldron Shale breaks down when exposed to water and air. This formation is problematic in conventional earthwork construction as those unfamiliar with its properties, mistakenly place the shale as a durable shot rock fill. Over time the shale will degrade causing structurally significant settlement of buildings and roadways. The Waldron Shale presents a challenge to the construction of the tunnel as the shale is prone to delaminating and degrading during construction of the tunnel. In addition, the Shale will undergo a change in it physical properties over time after the.

The Laurel Dolomite underlies the Waldron Shale. The Laurel Dolomite is composed 95 percent or more of gray dolomite with minor greenish-gray shale and sparse gray limestone.

CHALLENGES

There have been several challenges along the way including contract procurement, Indiana bats, finding available drills, to the unique physics of horizontal drilling. It took about eight months from the time we submitted our initial proposal to KYTC in April 2010 until we obtained our Notice to Proceed and signed contract at the end of December 2010. During that time, the mineral markets heated up, with gold leading the way at over $1500 per ounce. In that eight months we went from having a pick of drills to not having any drills. In the same time, the Devico teams also became committed to long term mineral exploration support. We literally spent months calling every continent but Antarctica looking for an underground drill rig and crew. Eventually, in March of this year (2011) both Major Drilling and TECH Directional freed up crews (at the same time!) and we began drilling in March 2011.

There have been numerous small issues that we have worked through. It has been interesting getting the bits, reaming shells, coring tools, water pressure, and other variables matched up to achieve the desired production rate. In addition the directional coring using the Devico system is as much art as it is science. It is amazing to watch the TECH Directional technician George Small, work with the Devico tools to keep the drill hole on the desired path. The horizontal drilling process is much different that vertical coring. The drill team must constantly battle the bias of the drill to work with gravity to pull the drill string downward, off the desired drill path. We also must contend with friction, or what the drill industry terms at “torque” as the drill string lays on the borehole the entire time it is operating. We are using combinations of drilling fluids and polymers to reduce the friction along the borehole and increase the efficiency of removing the cutting to the surface.

Upcoming challenges include televiewer, packer testing and grouting up the borehole.

CONCLUSION

The horizontal directional drilling has proved to be a cost effective means to explore the geologic conditions along the tunnel. The entire program of the horizontal directional drilling, conventional surface drilling, and surface and borehole geophysics will cost about one third of the exploration tunnel. At the end we will grout the holes up and reclaim our drill site. After we are through we will have an extensive understanding of the subsurface conditions along the tunnel, at one third the cost, and the local residents will not have to deal with a 12 foot diameter tunnel and 150,000 cubic yard face up as part of the design process.

HQ casing in face of shotcrete drill pit, Pilot Tool at end of HQ drill string

Troubleshooting Meeting

Drill Pit with S&ME drilling technicians

Drill Pit Set up (David Durman and Roger Hibbets)

George Small with Tech Directional making adjustment to Devico Core barrel

Justin Beckham and David Durman preparing to measure the hole location

Recovered Louisville Limestone

Recovered Louisville Limestone

SOIL NAILS AT GATEWAY TO NEBRASKA

Lok M. Sharma, M.S., P.E., Senior Principal
Terracon Consultants, Inc.
13910 W 96th Terrace
Lenexa, KS 66215
(913) 492-7777
lmsharma@terracon.com

Binod K. Sapkota, Ph.D., P.E., Geotechnical Engineering Specialist
Bechtel Oil, Gas and Chemicals, Inc.
3000 Post Oak Boulevard
Houston, TX 77056
(713) 235-3043
bksapkot@bechtel.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgement

The writers are indebted to George Schuler of the Judy Company for providing site photographs and due cooperation in successful completion of this project. Our thanks are extended to the Nebraska Department of Roads for assignment.

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

INTRODUCTION

The Gateway to Nebraska project consists of widening Interstate 80 from west of the existing bridge over Missouri River to 24th Street in Omaha, Nebraska near 13th Street. The widening project involved cutting into a hillside on the north side of Interstate 80. Cuts on the order of 46 feet were required necessitating the use of retaining walls to stay within the Interstate right-of-way. Comparison of various options of retaining walls resulted in adopting a three-tiered soil nail retaining wall. The soil nail wall had several advantages over other systems including reduced excavation, smaller impact on existing slope and flexibility in the facing design. Being a top down construction, the soil nail wall also provided speed in construction. The following paper describes the design and construction of a three tiered retaining wall using soil nails in a loessial deposit.

The soil nail retaining walls were designed using the FHWA design guidelines as per Nebraska Department of Roads (NDOR). The location of the project is indicated in Figure 1.

Figure 1 – Project Location, Omaha, NE

There are several walls that were designed and constructed to accommodate the highway widening at this project site. The walls were numbered from 1 to 7. Wall numbers 4, 5 and 6 are the three walls of the three-tiered retaining wall that are presented in this paper. Wall No. 4 being the lowest tier, wall No. 5 the middle tier and wall No. 6 the top tier. The total length of the three-tiered wall is approximately 1,380 feet (from station 1414+60 to station 1428+40). The maximum height of each tier varies from approximately 10 feet to 15 feet for a total height of about 46 including the undercut for the roadway. The natural ground above the top tier wall is sloped at approximately 3H:1V. A schematic of the walls is shown in Figure 2.

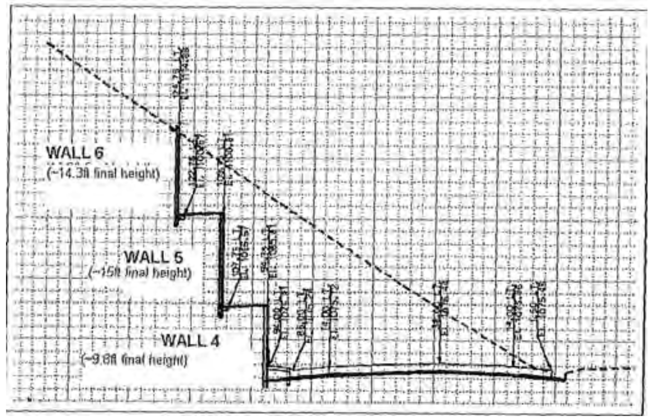


Figure 2 – Three-Tiered Wall

The soil nail wall at this site was a design build assignment. The following paper describes how the global stability was analyzed using both Limit Equilibrium Method (LEM) and FLAC numerical analysis method.

SUBSURFACE AND SITE CONDITIONS

The geotechnical subsurface exploration had been provided by NDOR. A copy of the geotechnical report " Final Foundation Report – Interstate 80: Missouri River to 13th Street, Omaha NE, Report No: NH-80-9(899): CN 22132, dated April 10, 2009" was provided to us. There were specific borings along the proposed location of the wall. Data from four borings and three Cone Penetrometer Tests (CPT) were pertinent to the wall design. Borings MR1 and MR2 and CPT MR-1 were performed at about the crest elevation of the wall and Boring MS1 and MS2, and CPTs ms1 and ms2 were performed at about the toe elevation of the proposed wall. Table 1 below provides the soil profile obtained from the borings.

Table 1 – Soil Profile

	Borings				Remarks
	MR1	MR2	MS1	MS2	
Location	1424+00, 146 LT	1417+50, 100LT	1423+40, 20 LT	1418+80, 25LT	Highest top and lowest bottom elevations of the walls- Wall-4: 1095.92 – 1065.87 Wall-5: 1112.27 – 1078.29 Wall-6: 1117.45 – 1095.89
Surface Elevation	1120.5	1118.0	1076.0	1082.0	
Boring Depth	41	30.1	36	30.7	
Bottom of Boring Elev.	1079.5	1087.9	1040.0	1051.3	
Soil Encountered					
	El 1120.5-1079.5 Lean CLAY (CL) trace sand, very stiff (Peoria)	El 1118.0–1087.9 Lean CLAY (CL) trace sand, very stiff (Peoria)	El 1076.0–1054.5 Lean CLAY (CL) trace sand, very stiff, (Peoria)	El 1082.0 – 1066.0 Lean CLAY (CL) trace sand, very stiff, (Peoria)	
			EL 1054.4-1045.5 Fat CLAY (CH) very stiff (Alluvium)	El 1066.0 – 1056.0 Lean CLAY (CL) trace sand, stiff (Loveland)	
			El. 1045.5-1040 Poorly graded SAND (SP) (Alluvium)	El 1056-1051.3 Lean CLAY, stiff (Alluvium)	

Water level was not reported in any of these four borings. Except at CPT ms1 located at elevation 1076 feet at Station 1423+50, pore water pressure was not measured at other two CPTs ms1 and MR1. Pore water pressure recorded below elevation 1072 feet at CPT ms1. For the soil nail wall design, water table is considered at elevation 1070 feet.

Based on the NDOR geotechnical report, published literature and our experience, the following soil data have been used

Soil Type	Lean Clay (Loess)
Moist Unit Weight	120 pcf
Ultimate Friction Angle	27 degree
Ultimate Cohesion	150 psf
Young's Modulus	1x10^5 psf (For FLAC analysis)
Poisson's Ratio	0.35 (For FLAC analysis)
Ultimate Tension	75 psf (Tension cut-off) (For FLAC analysis)
Ultimate Soil Nail Grout/Soil Bond Strength	10 psi

SOIL NAIL WALL

The design of a three-tiered wall presented certain challenges at this site. The subsurface conditions largely involved “Peoria” loess deposits. Use of soil nails in loessial deposit is always looked upon with caution. The sensitive nature of loess, especially the loss of strength in the presence of water is of major concern. The Peoria loess at this site was found to be very stiff with dense structure.

The three tiered soil nail wall was designed using the FHWA design guidelines. Starting from the top tier, each tier of wall was considered as a surcharge on the lower wall for global stability analysis using Limit Equilibrium Method (LEM) computer software “SNAIL” developed by CalTrans. The LEM analysis was verified using the “FLAC” computer program. The FLAC analysis model assessed global stability and also calculated the pull out stresses in soil nails. The lengths of the nails were primarily controlled by global stability.

The following structural data that meet the NDOR specification and FHWA Circular 7, were used for the soil nails:

Nail diameter	1-in diameter
Nail lengths	Variable
Nail strength	75 ksi yield strength steel bars
Steel Plates and Studs	36 ksi grade steel for plates
	60 ksi for welded studs
Facing reinforcement bars and wire mesh	60 ksi grade steel

The following structural data were used for the soil nail wall facings:

Concrete (CIP for Permanent Facing)	8 inch thick, 4000 psi grade concrete
Shotcrete (Temporary Facing)	4 inch thick 4000 psi grade shotcrete
Grout (for soil nail wall)	4000 psi grade

The soil nail wall design was performed primarily using SnailWin Version 3.0 soil nail wall design software developed by the California Department of Transportation (CALTRANS). Because SnailWin can accommodate soil nails in only one tier at a time, the analysis of the middle tier wall (Wall 5) was performed by considering loads due to the gravity weight of Wall 6 as surcharge at the top of Wall 5. Similarly, in the analysis of Wall 4, gravity weight of Wall 5 and Wall 6 were considered as surcharge at the top of the Wall 4. In the analysis and design of three-tiered wall: wall 4, wall 5 and wall 6 (bottom, middle and top tier respectively) were considered at their maximum combined height near station 1424+00. Figure 2 shows the cross-section at station 1424+00.

Analysis was performed for both static and seismic loading conditions. A live load surcharge of 75 psf represents the live load due to lawn mower or similar used under static load condition. In the analysis with seismic loading, an equivalent pseudo static load was considered. Based on the USGS National Seismic Hazards Map for Omaha, Nebraska, the peak ground acceleration 0.01819g representing 10% probability of excedence in 50 years was considered. Appropriate soil amplification, as recommended by FHWA Circular 7 was used in the calculation of design horizontal acceleration 0.034g.

FLAC ANALYSIS

FLAC (Fast Lagrangian Analysis of Continua) analysis was performed to design the required length of the nails for three tiered wall. FLAC is a numerical modeling program that uses finite difference coding commonly used for soil and rock analysis. Its dynamic option allows analyzing response of earth structure subjected to seismic loads. The FLAC version 6 was used in this analysis. The analyses were performed in two dimensional plain strain conditions with material constitutive model based on Mohr-Coulomb behavior.

Soil Parameters for FLAC Analysis

Soil Type	Lean Clay
Design Water Table	Elevation 1070 feet (perched water table)
Moist Unit weight	115 pcf above water table
	120 pcf below water table
Young's modulus	1x10^5 psf
Poisson's Ratio	0.35
Ultimate Friction angle	27 degrees
Ultimate Cohesion	150 psf
Ultimate Tension	75 psf
Ultimate Grout/Soil Bond strength	10 psi

Soil Nail Parameters

Nail Diameter	1 inch
Nail Length	Variable
Nail Strength	75 ksi yield
Nail Young's Modulus	4.18x10^9 psf (2.9x10^9 psi)
Bond Stiffness	5.4x10^7 psf
Bond strength	3000 lb/ft
Grout/soil bond friction	19 degrees
Hole Diameter	8 inches

Wall Facing Parameters

Thickness	1 ft (4 in. shotcrete+8 in. concrete)
Young's Modulus	4.5x10^8 psf
Poisson's Ratio	0.25
Compressive Strength	4000 psi
Tensile strength	10% of compressive strength

The analysis was performed using the above data. Figure 3 shows calculated Factor of Safety against a global slope failure of the pre-construction slope. The existing pre-construction slope had a Factor of Safety (FOS) of 1.85.

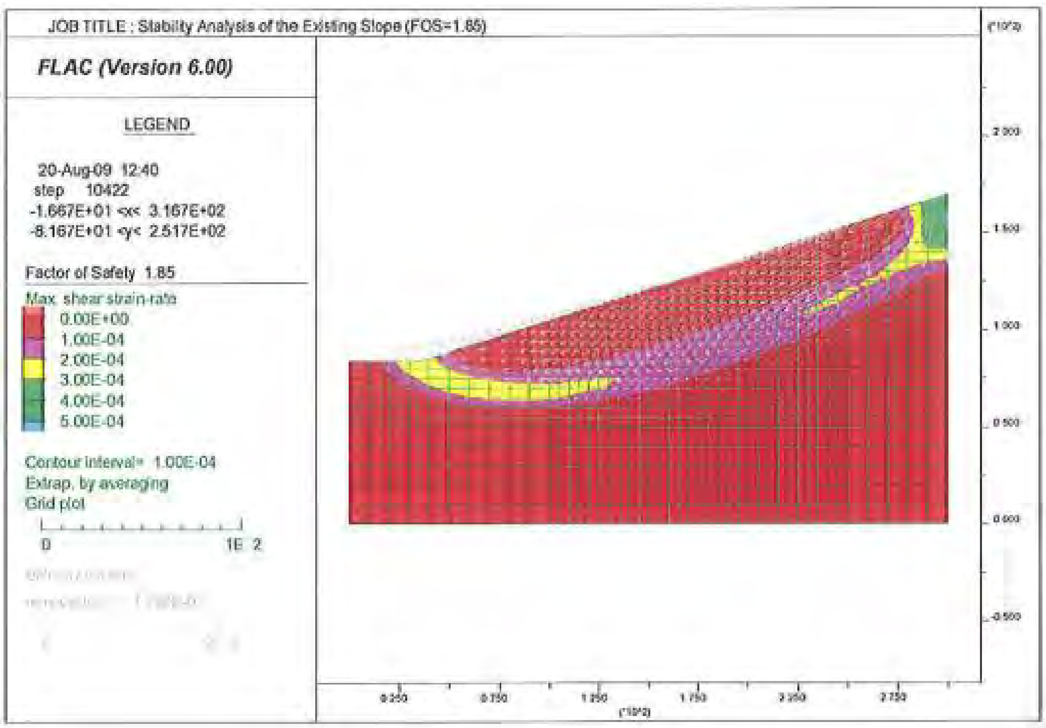


Figure 3 – FLAC Analysis of Existing Slope

For stability analysis of Upper Wall 6: considered 4 rows of soil nails at 4.5 vertical and 5 foot horizontal spacing. Figures 4 and 5 present the analysis results. Figure 5 presents the results of axial tension on the soil nails computed by FLAC. Similar analyses were performed for walls 5 and 4. Results are shown in Figures 6, 7, 8, 9 and 10. The figures also show assumed groundwater levels and distribution of finite difference zones (grid). In the model the vertical sides (at ends) were considered to be fixed against x-displacements and free in y-displacements. The base of the finite difference model was considered free in x-displacements and fixed in y-displacements.

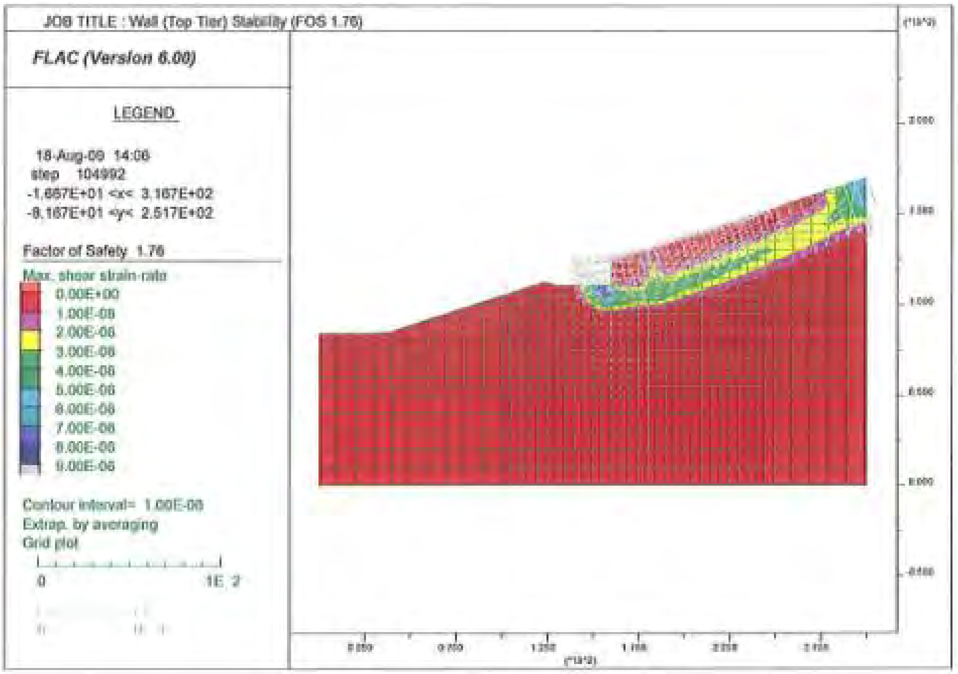


Figure 4 – Wall 6

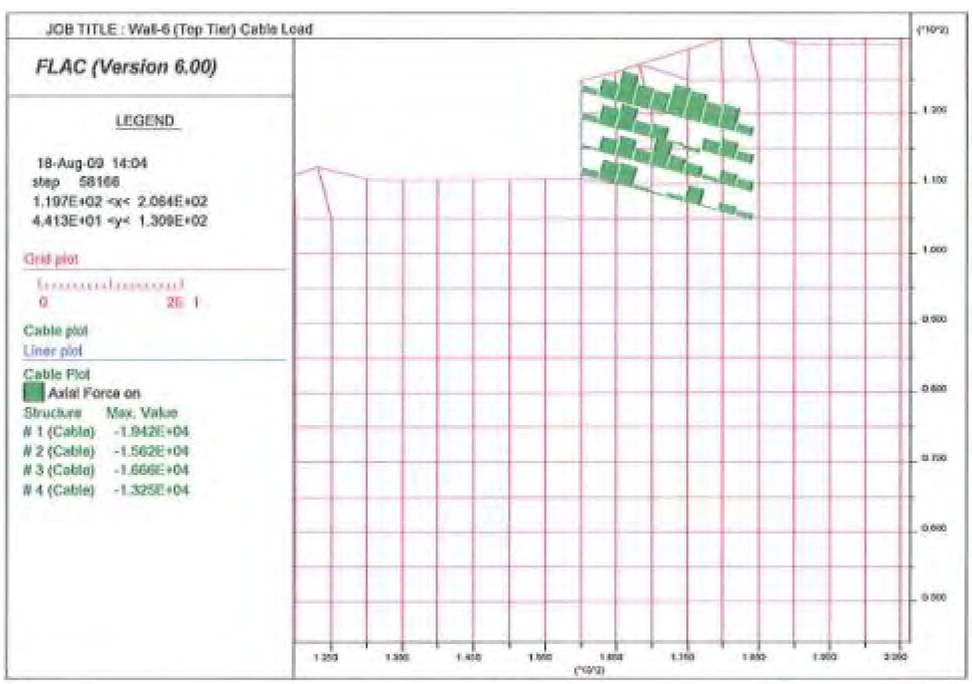


Figure 5 – Wall 6

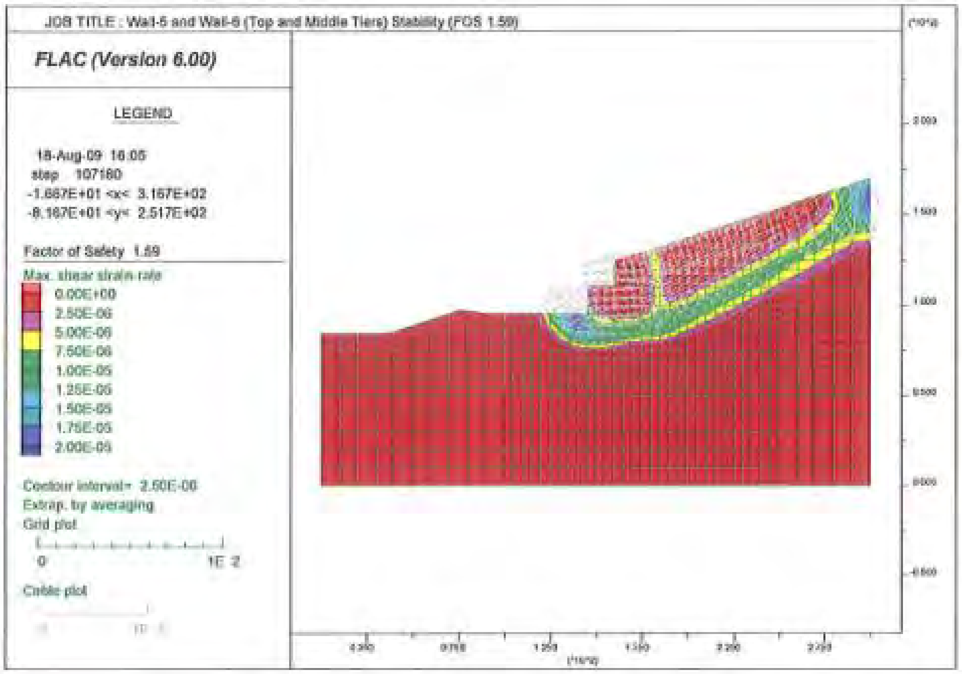


Figure 6 – Wall 5 and 6

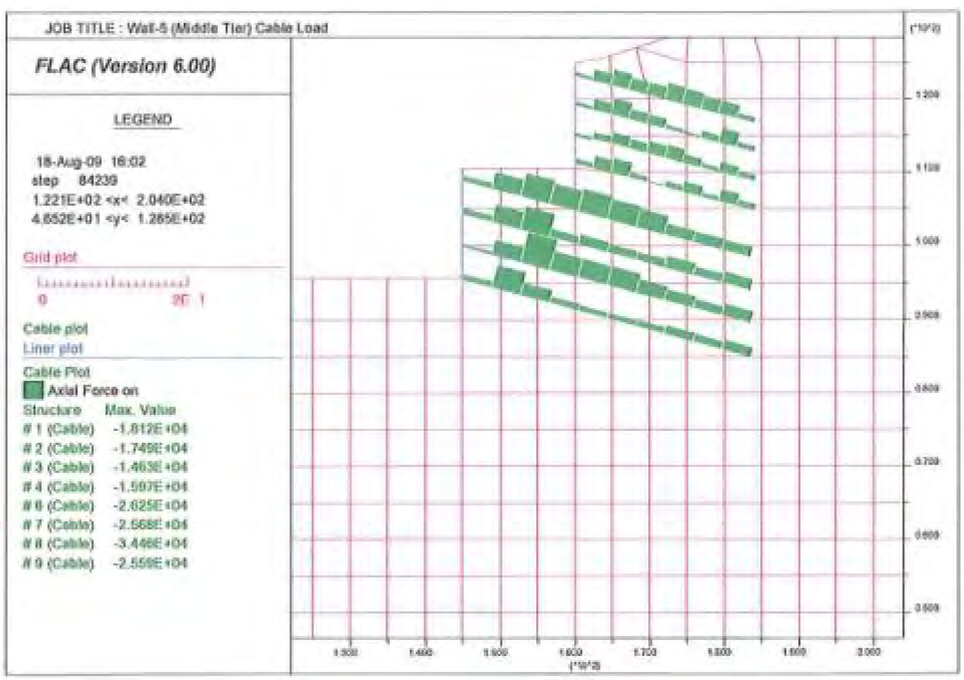


Figure 7 – Walls 5 and 6

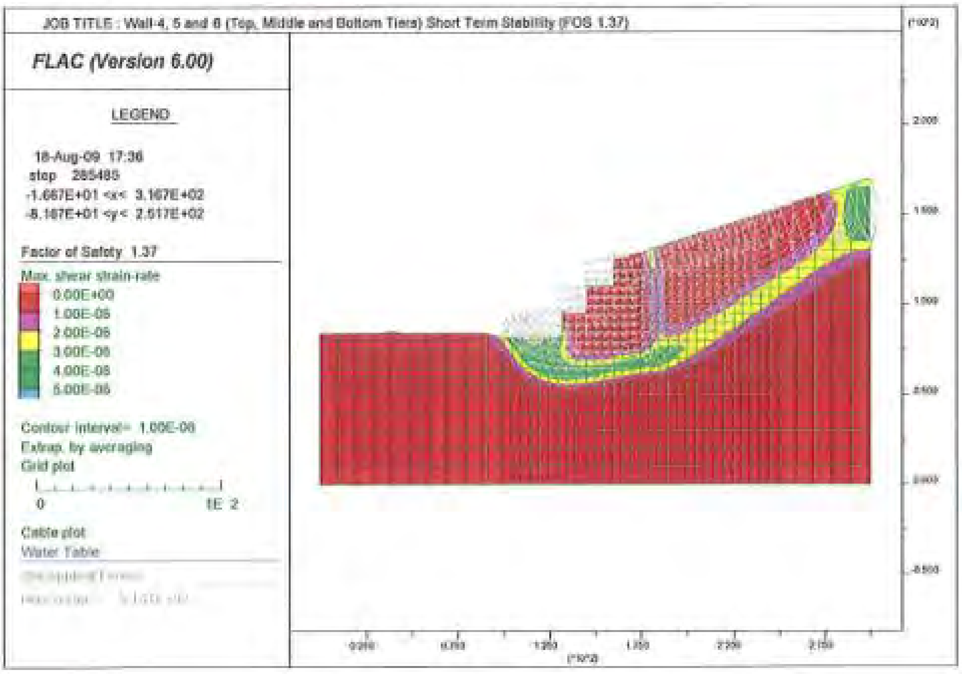


Figure 8 – Walls 4, 5 and 6

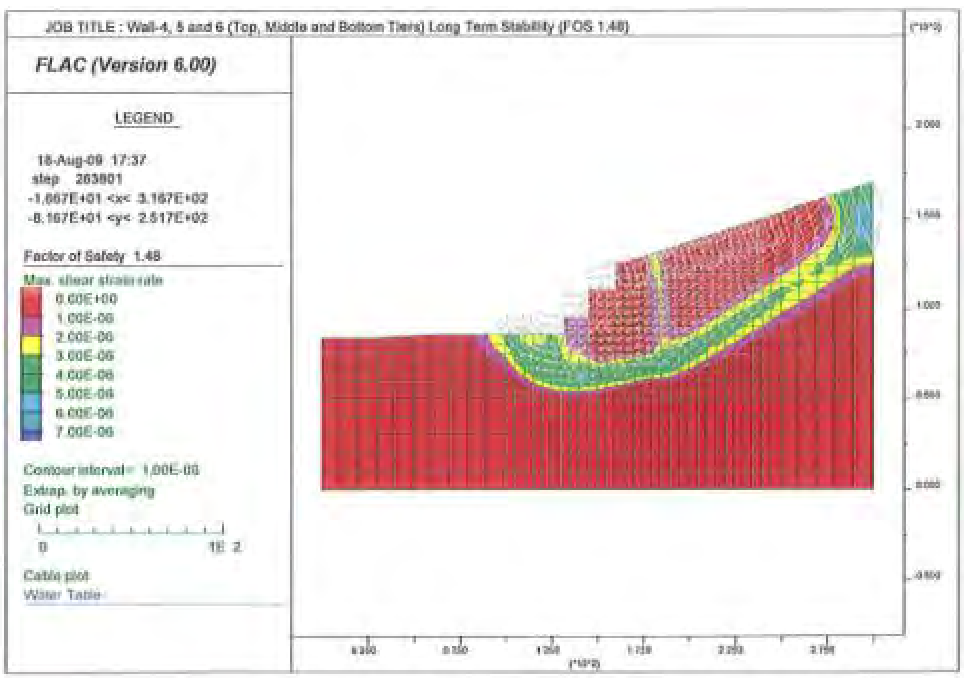


Figure 9 – Walls 4, 5 and 6

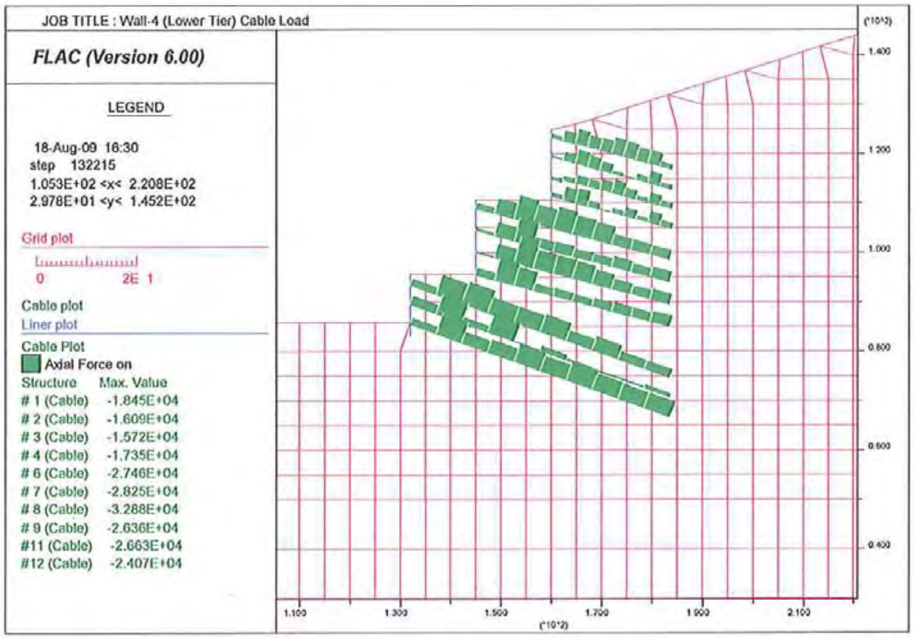


Figure 10 – Walls 4, 5 and 6

SN**AIL ANALYSIS**

SNAIL is a computer program developed by California Transportation Department (CalTran).The program is based on two-dimensional limit equilibrium method (LEM) that considers force equilibrium only. The limit equilibrium method of slope analysis is based on the principals of statics and remains a useful tool for stability analysis. The limit equilibrium method of slope stability analysis does not satisfy displacement compatibility when the material behavior tends to be elastic-plastic. The FLAC modeling presents stresses and strain based analysis and satisfy the issue of displacement compatibility.

The Windows based version of SNAIL – SNAILWIN was used in the analysis.

SNAIL program allows installation of soils nails in only one wall face (that is, on one tier only). Analysis of the upper single tier is thus permissible; however, for lower tiers, the back slope above the walls was modified to represent a surcharge on the lower walls. Figures 11, 12, and 13 present the representative cross-sections adopted for SNAIL.

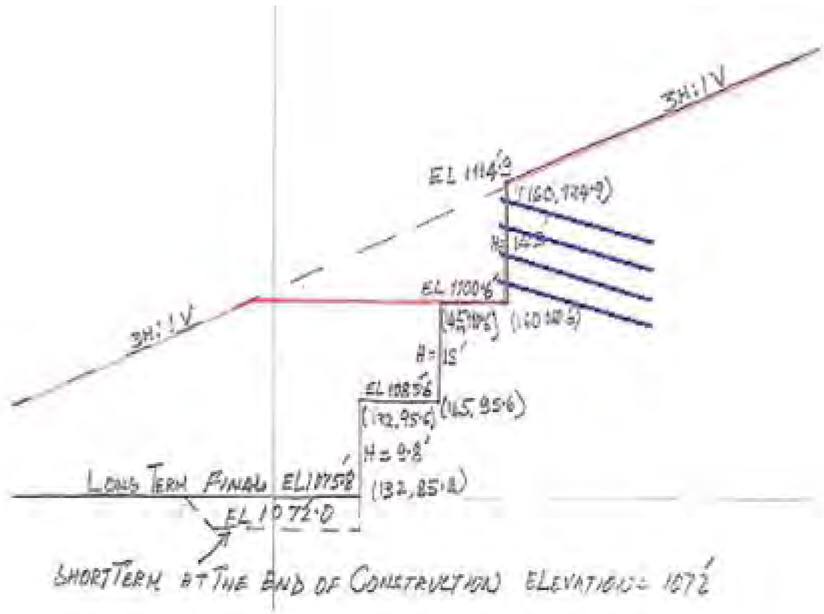


Figure 11 – Wall 6, SNAIL Analysis

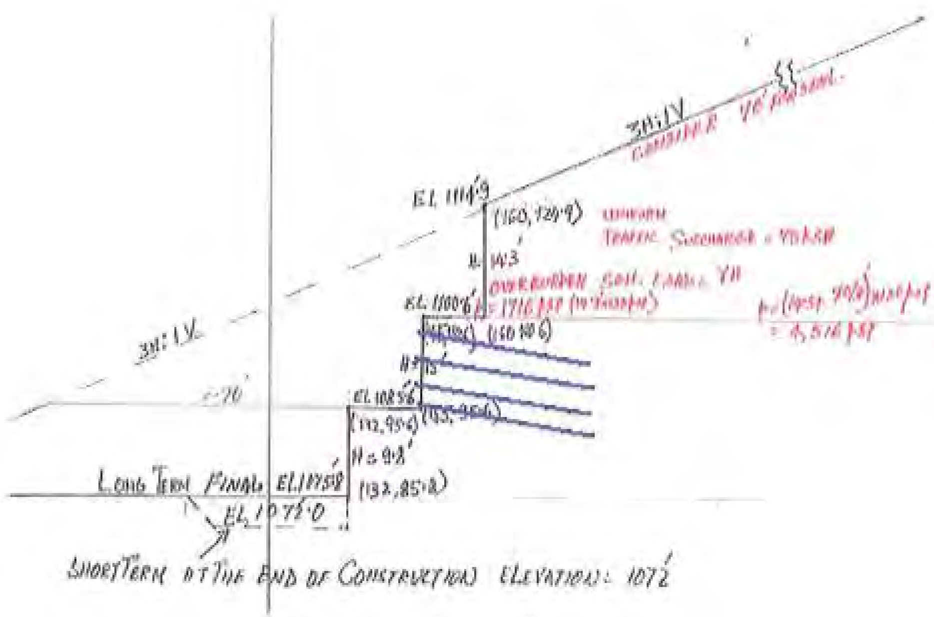


Figure 12 – Wall 5, SNAIL Analysis

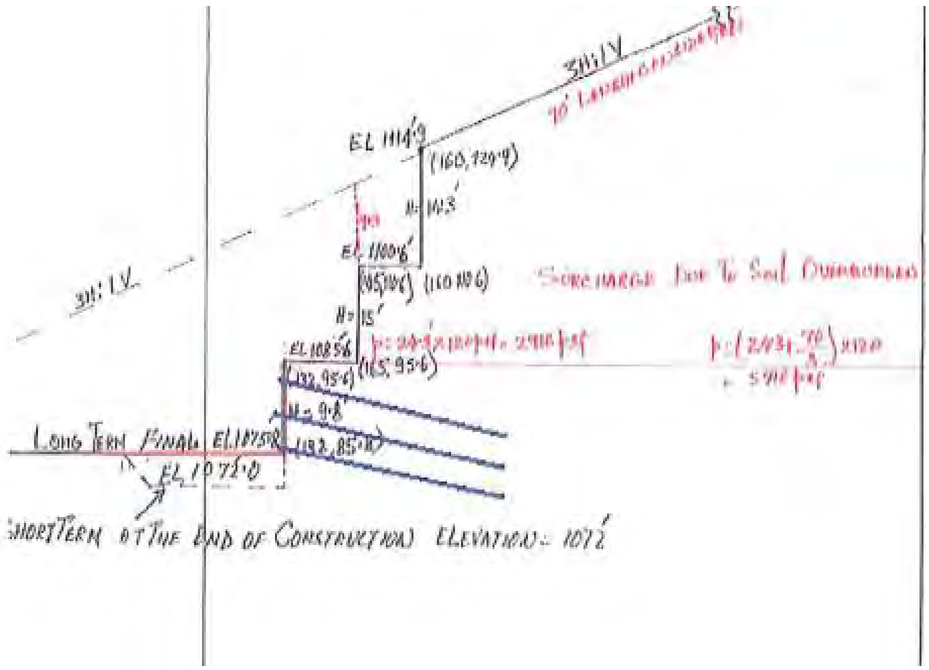


Figure 13 – Wall 4, SNAIL Analysis

The following Table 3 presents the results of SNAIL and FLAC analyses.

Table 3 – Results of Global Stability Analysis

	Top Tier (Wall 6)	Middle Tier (Wall 5)	Bottom Tier (Wall 4)
Maximum Wall Height (ft) (approximate)	14.3 (with 3H:1V back slope)	15 ft (Wall 6 above it)	9.8 ft (Wall 5 and 6 above it)
Nail Length (ft)	25	40	55
FOS from FLAC	1.76	1.59	1.48 / 1.37*
FOS from SNAIL	1.60 (1.48)	1.68 (1.57)	1.53 (1.48)
Design Nail Length (ft)	25	40	55
Design Nail Rows and Nominal Spacing	4 rows vertical 4.75ft horizontal 5ft	3 rows vertical 4.5ft horizontal 5ft	3 rows vertical 4ft horizontal 5ft

Note:
FOS in Parenthesis in with SNAIL = FOS under seismic conditions. All other FOS are under static conditions

* FOS 1.37 is for the short term height of Wall 4

EXTERNAL STABILITY OF SOIL NAIL BLOCK

The external stability of the soil nail wall involves sliding, overturning and bearing capacity check. Figure 14, 15 and 16 show the configurations used in the external stability analysis for static and seismic conditions. For seismic stability Mononobe-Okabe Equation was used to compute pressures. All computed FOSs were acceptable.

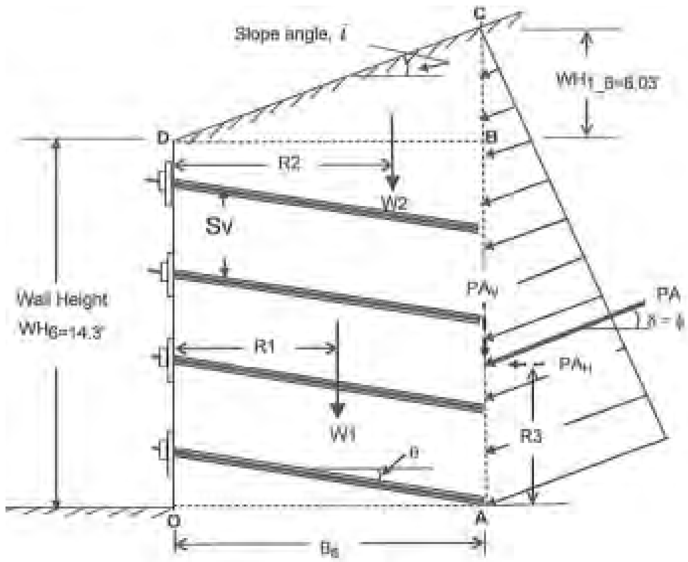


Figure 14 – External Stability – Wall 6

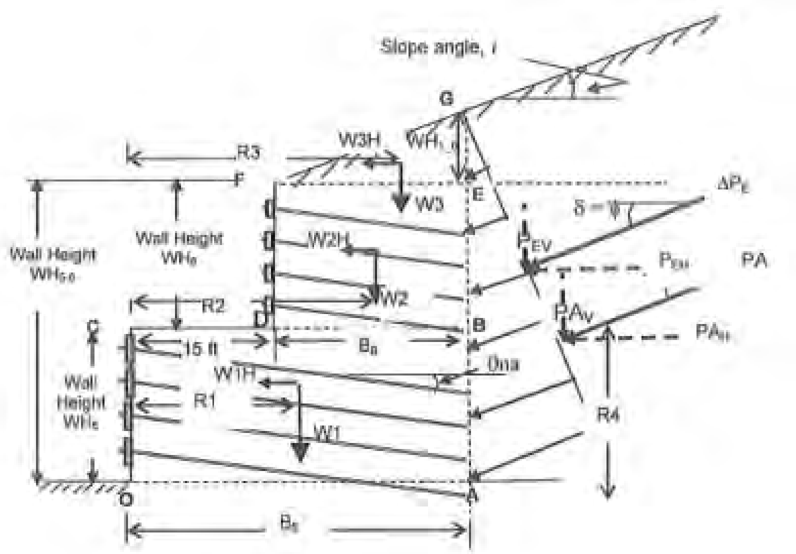


Figure 15 – External Stability – Walls 5 and 6

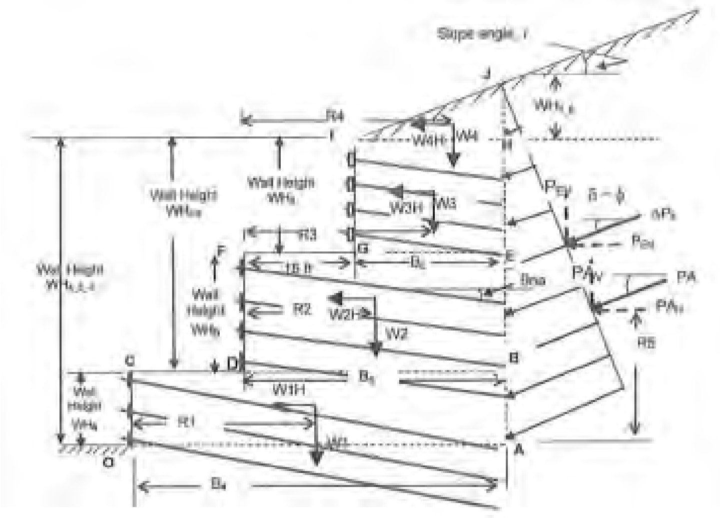


Figure 16 – External Stability – Walls 4, 5 and 6

CONSTRUCTION

The construction of walls was preceded by a number of proof and verification tests. All proof test nails were sacrificial. Verification tests were performed on selected production nails. Extensive testing was justified due to variability in soil conditions. At one location of the wall an old trash dump was encountered (Photograph 1). Excavation of the trash and replacement with well compacted soil was implemented. Proof tests of nails were performed through the backfill soil areas. The construction followed the normal procedures for soil nail wall construction in a top down manner, excavating in stages for each level of soil nail, applying the construction shotcrete facing and then proceed to the next level. The loess deposit was generally a stiff to very stiff material with dense structure. Photographs 2, 3, 4 and 5 show the construction work in progress.

Photo 1

Photo 2

Photo 3

Photo 4

Photo 5

Due to high visibility, the upper and the bottom tiers surfaces were provided with recessed notches in the wall for aesthetics. Also the facing of the wall was finished with a shotcrete-in-place and carved and stained surface to give a bedrock appearance as shown in Photographs 6, 7 and 8. Photograph 9 shows an overall appearance of the three tiered soil nail wall.

Photo 6

Photo 7

Photo 8

Photo 9

The soil nail walls have been constructed with surface mounted monitoring hubs. The monitoring is being performed by NDOR. The data that has been available to date indicated deformations are well within the acceptable limits.

CONCLUSIONS

The FLAC numerical modeling allowed a displacement based slope stability analysis, while limit equilibrium method presented the conventional analysis for global stability of a tiered retaining wall. The design procedures for a tiered Soil Nail wall are not appropriately available and simplification of design in terms of considering the upper walls as surcharge on the lower wall appeared to have provided adequate design of the soil nail wall at this site. The FHWA soil nail design methodology for the soil nails and the facing design is found to be appropriate.

With the co-operation of the site contractors and the NDOR, a well designed wall with extensive analysis has been constructed. The wall has presented itself as the most economical solution for this site. With the high visibility, the appearance of the wall blends well. The wall has become a showcase project due to its vicinity to the urban environment.

REFERENCES:

- Federal Highway Administration (2003) "Geotechnical Engineering Circular No. 7, Soil Nail Walls", FHWA 0-IF-03-017, Carlos A. Lazarate, Victor Elias, R. David Espinoza, Paul J. Sabatini

- Fast Lagrangian Analysis of Continua (FLAC) ver. 6.0, Itasca Consulting Group, Inc.

- Terracon Consultants, Inc., Design report of Soil Nail Walls, I-80 Missouri River to 13th Street, Omaha, NE, August 2009.

Landslide Stabilization along the Ohio River Using Cantilevered Stub Piers

Swaminathan Srinivasan, P.E., M. ASCE
Terracon Consultants, Inc.
611 Lunken Park Drive
Cincinnati, OH 45226
(513) 321-5816, ext. 356
ssrinivasan@terracon.com

Jess A. Schroeder, P.E., M. ASCE
Terracon Consultants, Inc.
611 Lunken Park Drive
Cincinnati, OH 45226
(513) 321-5816, ext. 356
jaschroeder@terracon.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to than the following individuals for their contributions in the work described:

Joseph Smithson, P.E. – Ohio Department of Transportation
Andrew P. Bodocsi, P.E. PhD, - Terracon Consultants, Inc.
Ernesto A. Muccillo, P.E. – Terracon Consultants, Inc.

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)
All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

Landslide activity along U.S. 50 in Cincinnati, Ohio has caused roadway damage for decades. After a necessary closure of 3 lanes due to slope movements, emergency stabilization measures were undertaken to protect the roadway by providing a “pseudo” long-term solution (target 3 to 5 years) necessitated by ODOT budget constraints.

The landslide shear plane was near the top of a sloping bedrock surface as much as 50 feet below grade. Drilled shafts were installed 40 feet downslope of the roadway shoulder. The shafts were heavily reinforced across the shear plane but steel reinforcing did not extend the full length of the shafts and was stopped well short of the ground surface. The goal was to provide shear resistance across the failure plane, forcing the theoretical failure surface higher into the overburden soil profile, resulting in a comparatively higher safety factor against slope failure. These “Stub Piers” were installed and found to meet all of the project goals.

The stub piers and surrounding ground were instrumented and preliminary analysis of collected data showed earth pressures and horizontal deflections were over-predicted in the original design. Instrumentation by means of inclinometers, vibrating wire earth pressure cells, and strain gages has been monitored over a period of about 6 years since construction of the Stub Piers and results indicate this option offers much more than a short-term solution to the problem.

INTRODUCTION

Landslide activity has occurred along U.S. Rt. 50 in western Cincinnati, Ohio for many decades. The site is located between North Bend and Addyston, OH, on the right descending (cutting) bank of the Ohio River, at about river mile 485.25. The landslide activity along this area has been on-going for many years. Slope and road movements have required periodic repairs over recent decades. The railroad tracks downslope of the roadway also show signs of horizontal displacement and periodic repair. Visual evidence suggests the shear plane extends below the roadway at deep levels and extends out into the Ohio River.

In brief review, the road elevation at the time of our original site study was at about 508 to 516 ft., increasing in an East-Northeast direction. A weed and brush-covered slope extended southwest and downward toward the Ohio River at about 3H:1V. The slope rose more than 100 feet above the roadway. On the downhill side of U.S. 50, grade sloped down about 15 to 20 feet in elevation to a railroad right-of-way at about elevation 490 ft. The riverbank then sloped down at about 2.5H to 3H:1V to the water's edge. Normal pool elevation of the Ohio River is 455 ft.

H. C. Nutting, a Terracon Company (Terracon) was retained by the Ohio Department of Transportation (ODOT) to perform a geotechnical study that included 17 test borings and inclinometer monitoring at 4 locations. After only a few weeks of monitoring, the inclinometer casings sheared off about 50 feet below grade, near the soil / bedrock interface. Soon after, the roadway distress worsened, causing ODOT to close 3 of the 4 lanes to traffic and reroute traffic onto the remaining lane and shoulder. Terracon was asked to develop a stabilization design under emergency repair conditions. However, funds were limited at the time, necessitating a direction by ODOT that the solution be at least "pseudo" short-term (3 to 5 years).

The on-going landslide displayed deep-seated movement extending down to the top of bedrock, about 40 to 50 feet below present grade. The toe of the slide most likely extended out into the Ohio River. Terracon was asked to develop a stabilization design under emergency repair conditions. However, funds were limited at the time, necessitating a direction by ODOT that the solution be at least "pseudo" short-term (lasting perhaps 3 to 5 years).

The use of a toe berm or MSE-type retaining wall was not considered practical or feasible for remediation due to the ODOT right-of-way limitations and also because such a repair method would add unwanted load and driving forces to the landslide. Such a load could possibly accelerate slope movements.

The use of a "soil nail launcher" was also discussed with ODOT. This method of remediation was not considered feasible either. The slide plane extends to bedrock and the soil nails installed by this launching technique would not likely extend deep enough nor provide the level of shear and passive restraint needed.

The most appropriate and effective long-term remedial measure appeared to be the construction of a soldier pile or drilled pier wall containing multiple rows of tieback anchors. The anchor installation would likely involve substantial excavation for equipment access to install multiple tiers of tieback anchors. While effective, this method would involve significant cost. After discussions with ODOT, it was our understanding that a sufficient budget was not currently available for “permanent” repair. Instead, ODOT requested a recommendation from Terracon for a “temporary” repair. The primary goal was to allow U.S. 50 to be reopened and maintained open for some period of time (3 to 5 years). This period of time would allow for budget and plans to proceed with a more permanent solution.

Due to the significant depth to bedrock and the deep shear plane, the use of “stub piers” was proposed by Terracon as the “pseudo-temporary” repair. A series of heavily-reinforced drilled piers were designed and constructed. The pier reinforcement was somewhat unique when considering more standard practice in the Cincinnati local area. Details are presented in the following paragraphs, as well as instrumentation results.

GEOLOGIC SETTING

The overburden profile consists of cohesive embankment fill, alluvium, colluvium, and residuum. Fill ranges from 10 to 25 feet deep and is underlain by alluvium that is interbedded and sometimes lying atop colluvium. Colluvial clays are formed by action of gravity and have slickensides with random orientation. Residuum is also present in some areas at a thickness of about 3 feet. Residuum is a soil formed from the in-place weathering of the underlying parent bedrock.

Bedrock lies between 31 and 50 feet deep. Typically, gray shale and limestone occurs. However, about 3 feet of brown weathered shale with limestone occurs in some locations above the gray shale. The horizontally-bedded shale and limestone belongs to the Kope Formation (Ordovician System) and includes shale that rates as very soft to soft in terms of bedrock hardness. There are numerous documented landslides in this local geologic setting. Shale comprises about 90% of the Kope’s mass. Very hard limestone makes up the remainder, occurring in layers up to about 1.5 inches thick. Figure 1 provides a general subsurface profile illustration.

The Ohio River in this area has a normal pool elevation of 455 feet and official flood elevation of 485 feet. The 100-year flood elevation is 501 feet while the highest recorded river level in Cincinnati occurred during the 1937 flood at elevation 512 feet. With the U.S. 50 roadway elevation at 508 to 516 feet and the railroad at 490 feet, at least the lower portions of this slope are subject to periodic flooding and river drawdown conditions. These conditions worsen the overall slope instability.

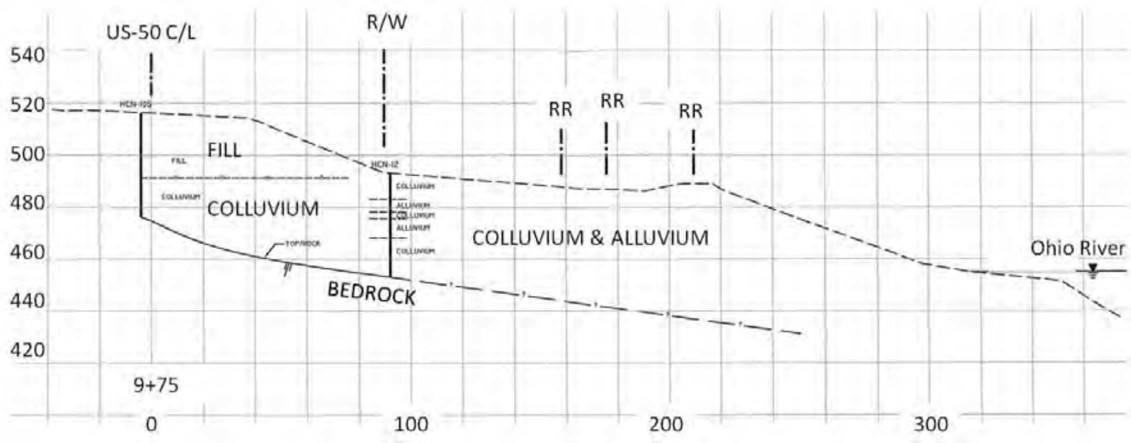


FIG. 1: Typical Subsurface Profile

RIVER STAGE AND PRECIPITATION ANALYSIS

Hagerty (1983) studied the combined effects of elevated river stage and precipitation on riverbank stability along the Ohio River. That research evaluated 10-day cumulative rainfall combined with river stage. The writers have some experience with this and have seen relatively good correlation between significant combinations of the coincidental occurrence of these two factors with actual observed slope movements along the Ohio River. A similar review was performed for the subject site. Reference is made to Figure 2 showing date versus both the Ohio River elevation and 10-day-cumulative precipitation over a period spanning from November 2004 through December 2006. The 10-day cumulative precipitation plotted for any given day was computed by adding daily precipitation for that day plus the previous 9 days. We have also assumed any recorded snowfall depth to be equivalent to rainfall depth at a ratio of 10%. In other words, a snowfall of say, 1.1 inches was assumed to equal 0.11 inches of rain, as an approximation.

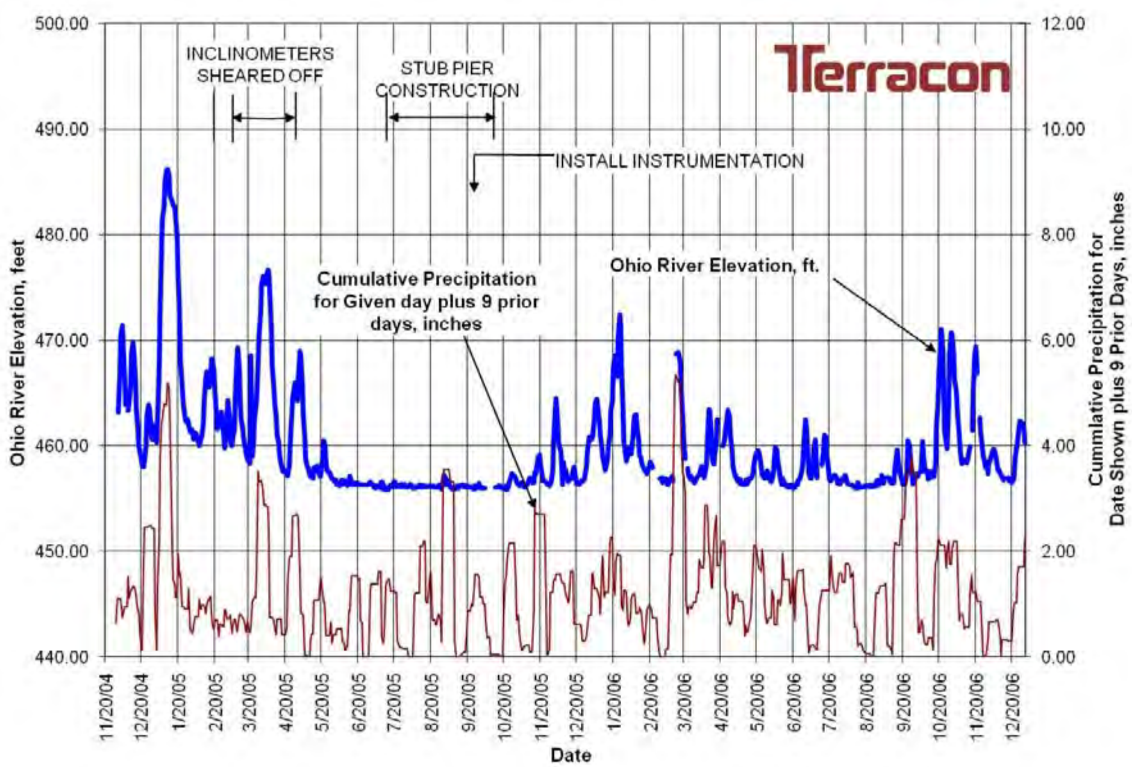


FIG. 2: Ohio River Stage in Cincinnati vs. 10-Day Cumulative Precipitation

When both the river elevation and 10-day cumulative precipitation curves are reviewed in unison, some rather significant events become evident:

1. Event No. 1: Approximate period between 1/6/05 and 1/14/05.
2. Event No. 2: Approximate period between 3/27/05 and 4/6/05.
3. Event No. 3: Approximate period between 4/22/05 and 5/3/05.
4. Event No. 4: Approximate period between 3/11/06 and 3/21/06.

It is interesting that these events correlate rather well to observed slope movements during the course of this study. For example, the inclinometer casings installed during the initial study sheared off in a relatively short period about March or April, 2005, corresponding to both events 2 and 3 listed above. Event No. 1 was just prior to that time period and may have actually set the whole slope in motion at the time the initial test borings were being drilled. Event No. 1 appears to be the most significant for the duration shown on Figure 2.

As shown, the river and rainfall behaviors were more normal during the time of construction (Summer 2005). The new instrumentation program began in October 2005. Collected data from some of the devices suggested a slight acceleration in movements and earth pressure build-up during a period of about March / April 2006 (Event No. 4), but overall, the stub pier resistance showed little impact of Event No. 4 on the hillside.

This analysis has shown reasonably good comparison between observed accelerated slope movements and a combination of elevated river stage (and associated drawdown) and elevated events of cumulative 10-day precipitation.

STUB PIER DESIGN APPROACH

The assumed repair method included a single row of straight-sided drilled piers socketed into bedrock. Due to the thickness of overburden, a tieback anchor system would be required to support these shafts as a more permanent repair method. However, as directed by ODOT, the primary goal here was to develop a temporary repair scheme within a specific budget. Therefore, it was assumed that the reinforced concrete piers only extend part of the way upward through the overburden soils. These "stub piers" were assumed to be closely spaced where soil arching could be assumed to make the piers behave as a continuous wall. The piers would therefore force a theoretical shear plane upward from the bedrock surface to above the pier butt elevation.

The selected design consisted of a single row of cantilevered drilled shafts located within the right-of-way about 40 feet downslope of the roadway shoulder. The shafts would be socketed into bedrock. The innovative and cost-effective aspect of this scheme involved the steel-reinforcing length. Only the zone near the deep shear plane would be heavily reinforced, thus creating shear pin-type support. The structural steel would be terminated as much as 35 feet short of the ground surface.

From an analytical point, the short-term solution criterion was quantified by slope stability analyses. Laboratory tests were conducted and soil parameters were then adjusted slightly for the failed slope condition (safety factor of 1.0) and observed shear plane depths. Then, the shear plane was forced upward to the planned top-of-steel elevation of the stub piers. This process resulted in a theoretical safety factor increase from the original 1.0 to about 1.2 (see Figure 3). ODOT was conferred with and they agreed with this potential improvement, as a short-term solution.

Stub pier design details were then developed. The lateral earth pressure was estimated assuming triangular pressure distribution from the ground level to the shear plane. This resulted in a trapezoidal-shaped earth pressure diagram acting on the piers. For potential arching effects above the steel, it was assumed that the contributing pressure extended to one pier diameter above the top-of-steel. This estimated earth pressure was also checked using slope stability analysis to compute the resisting pressure required to generate a safety factor of 1.2. Refer to Figure 4 for schematics of the assumed earth pressure diagram.

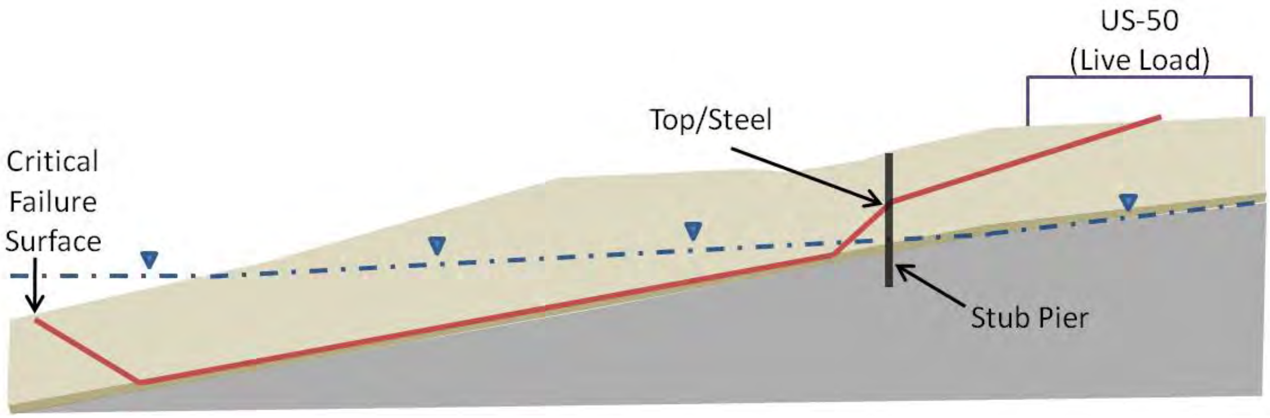


FIG. 3: Slope stability schematic.

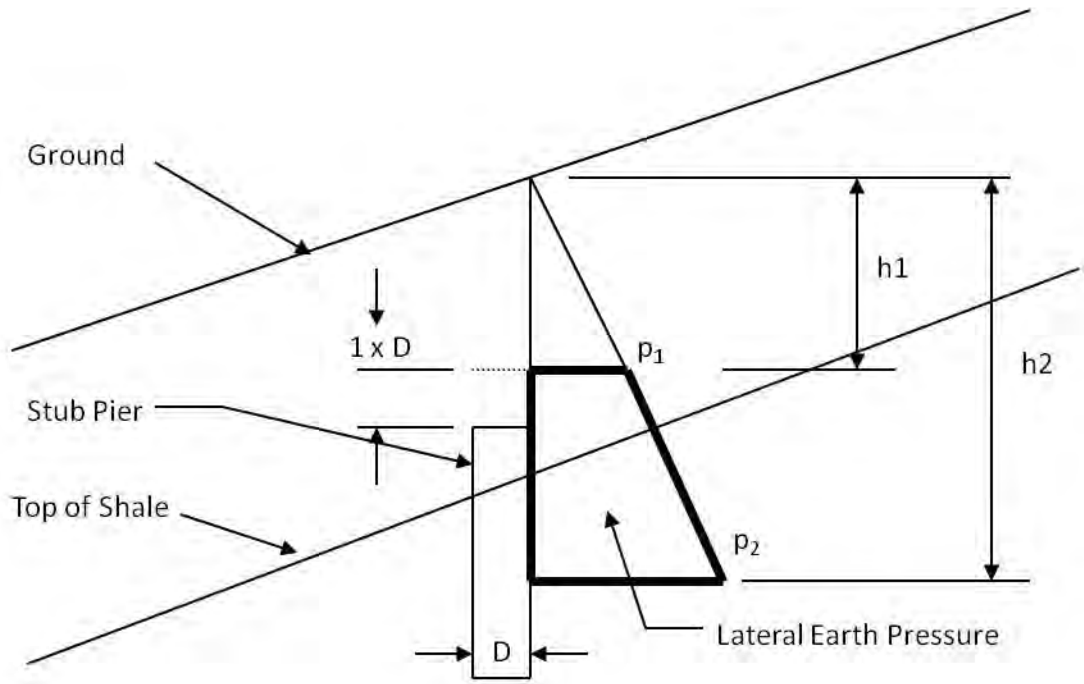


FIG. 4: Earth Pressure Schematic

Stub pier design was developed using the LPILE computer program. The drilled shafts included 30 and 36-inch diameter units and were socketed 10 to 15-ft. into gray unweathered shale bedrock. The steel reinforcement within the drilled shafts consisted of rolled steel sections that included HP14X73, W18X119, and W24X117. In some cases, additional bending resistance was necessary and developed by welding a steel plate to the uphill face of the beam. The steel extended to the bottom of the hole; however, it was limited in length and only extended about 20-ft. above the top-of-rock. Therefore, steel beam lengths ranged from 30 to 35-ft. and stopped well short of the ground surface. The top-of-steel was essentially determined to be the top-of-shaft, thereby assuming that shear failure of the slope could occur at the top-of-steel. The shaft opening above the steel was backfilled with either unreinforced structural concrete or a lean concrete fill, as determined by ODOT in the field. Based on visually observed limits of the most severe landslide movements, Terracon provided suggestions on various options of drilled shaft coverage to ODOT. After review of that information, ODOT elected to have 154 shafts installed on 5-ft. centers.

Due to the limited height of the reinforced section of these shafts (with their tops occurring well below grade), they were essentially deemed to act as shear pins installed across the deep failure plane. For the presentation purposes these shafts have been termed “Stub Piers.”

CONSTRUCTION

The 154 Stub Piers were installed from July to September 2005 under an emergency repair contract. The roadway was repaved on October 6 and 7, 2005, adding upwards of 2 feet of new asphalt in some areas to relevel the road. Traffic was reopened on October 7, 2005.

Information supplied by ODOT indicated the cost for stub pier installation was about $500,000.00 (in 2005 dollars). This cost included drilling, reinforcing, and backfilling 154 stub piers. As-built quantities included 8386 feet of shaft drilling, 1485 cu. yds. of concrete backfill, 553 cu. yds. of flowable fill backfill, and 273 tons of structural steel beams plus stiffening plates.

INSTRUMENTATION

ODOT approved a Terracon-recommended instrumentation program to order to monitor the slope, verify that the stub piers were meeting design goals, and to help confirm design assumptions. This program began shortly after construction was underway. Locations for instrumentation devices were selected for their critical locations, as well as to coordinate with the contractor’s activities and schedule.

The instrumentation program consisted of the following:

1. Five Inclinometers were installed within selected Stub Piers.

2. Four Inclinometers were installed about 30 feet upslope of selected Stub Piers.

3. Two Inclinometers were installed about 10 feet downslope of selected Stub Piers.

An inclinometer consists of a specially grooved PVC pipe that is socketed into bedrock or another fixed reference. Readings are taken by lowering the inclinometer probe down the pipe to obtain a profile of the horizontal displacement from its original position.

4. Three Push-In Earth Pressure Cells (Geokon Model 4830) were installed within boreholes located about 8 to 10 feet upslope of selected Stub Piers. These devices were located about 40 to 45 ft. below grade and were installed with the intent of being just above the bedrock surface (close to the interpreted shear plane). These devices measure total pressure in the soil.

5. At two piers, six strain gages were installed per pier (four on the tension side and two on the compression side. The strain gages (Geocon Model 4000 Strain Gages, weldable mounting blocks, plucking coil and thermistor) were weldable vibrating wire gages that were welded directly to the steel beam/soldier pile. A thermistor is integrated into the strain gages to account for temperature induced strain. Individual pieces of angle iron were welded over the strain gages to prevent damage during concrete placement.

The strain gage cables were extended up the two respective stub piers to the ground surface. These cables, as well as the earth pressure cell cables, were routed laterally to a terminal box, which was installed on a post embedded within the top of a nearby Stub Pier. The lateral distance from the terminal box to either of the strain gage-instrumented piers was about 40 feet.

INSTUMENTATION DATA REVIEW

Measured horizontal deflections over a 6-year period following Stub Pier construction indicates the Stub Piers have performed well. The horizontal deflections at the top-of-steel beam elevation at the five instrumented piers are less than 0.75 inches. The average deflection rate for these inclinometers is less than 0.1 inch per year. For comparison, horizontal deflection was predicted during design using the computer program LPILE. The theoretical value at the top-of-steel was about 4-inches and thus, well above values measured from the inclinometers embedded within the piers. While the readings at the ground surface are less meaningful, since the upper elevations of the inclinometer casing may be embedded in no more than unreinforced flowable fill, horizontal deflections are less than 2 inches and appear to be increasing at a rate of about 0.3-inch per year.

There were four inclinometers installed about 20 to 30 feet upslope of the Stub Piers and they indicate horizontal deflections less than about 1.25 inches (or a rate of less than 0.2 inches per year).

Finally, there were two inclinometers installed about 10 feet downslope of the piers. After six years, the maximum horizontal deflections (at grade) are less than 1 inch.

These measurements show there is still some residual creep movement downslope of the piers, but at a rate less than 0.2 inches per year.

Overall, the five inclinometers installed within the stub piers collectively showed the shear plane had indeed been successfully cut off and well-supported. Not all inclinometer data is reproduced here, but representative plots are shown on Figures 5a, b, and c.

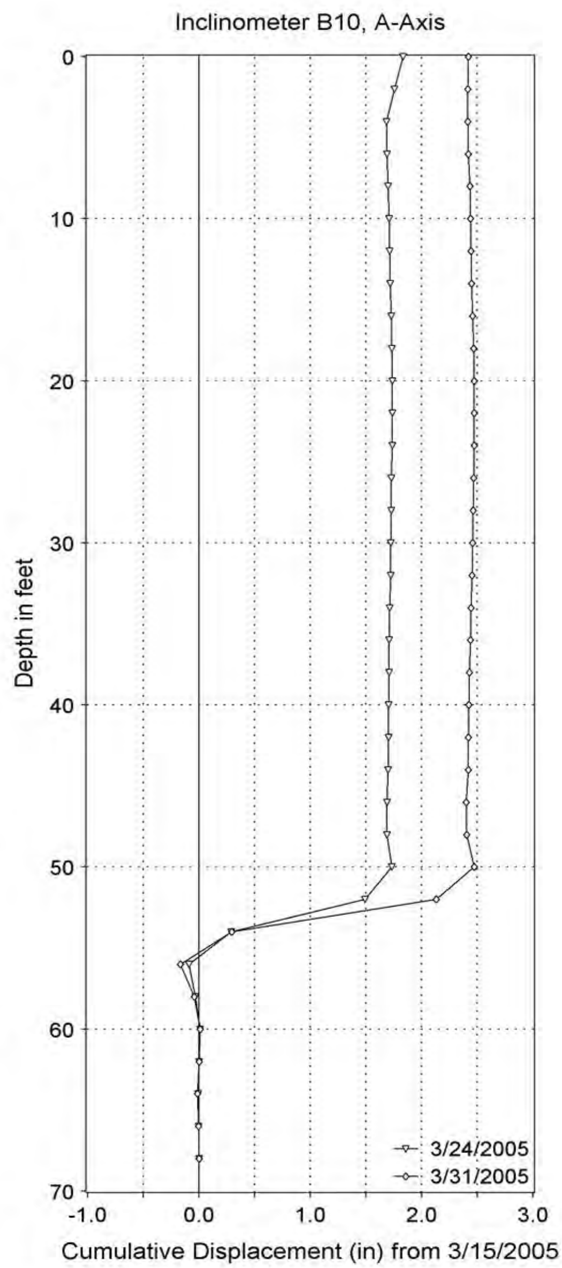
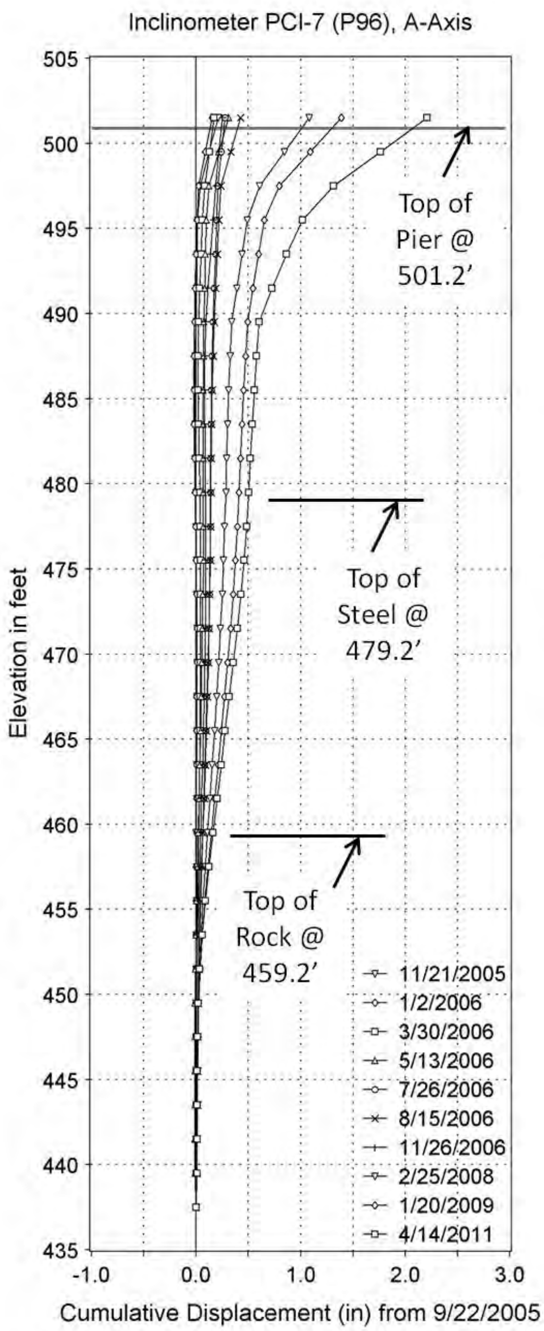


FIG. 5.a.: Typical inclinometer results showing horizontal displacement <u>before</u> construction.

Figure 5.a. on the previous page shows a typical inclinometer before pier construction. The shear plane is clearly evident near the deep soil / bedrock interface. In fact, some of these original inclinometer casings sheared off during the pre-design monitoring program.



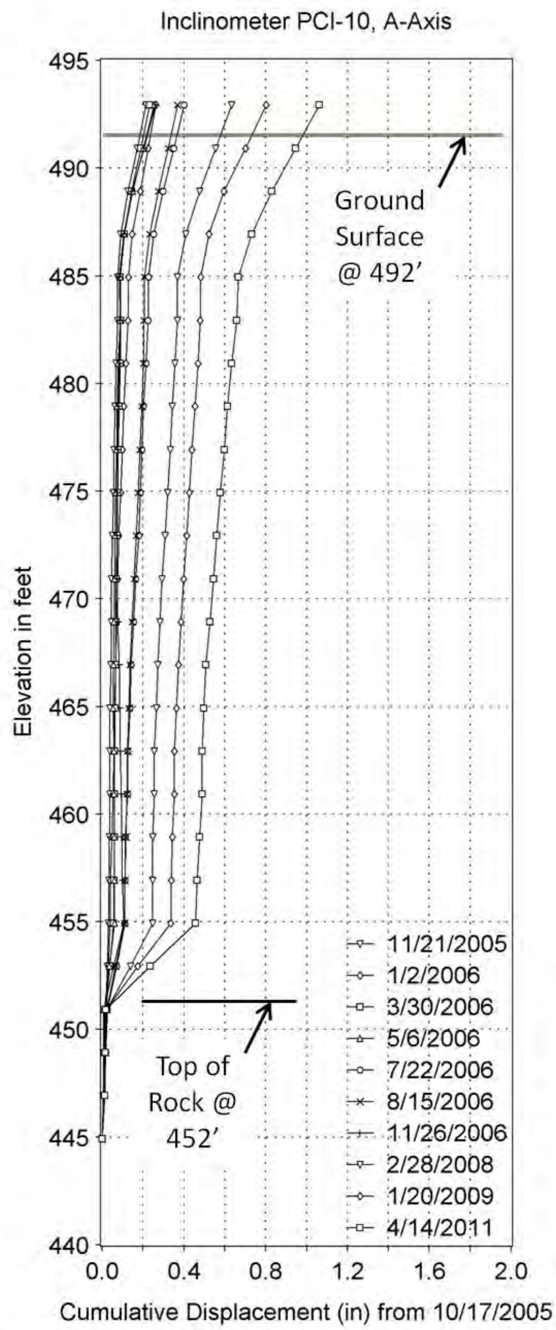


FIG. 5.b.: Within the Stub Pier. **FIG. 5c.: Downslope of Stub Piers.**

Figure 5.b. on the previous page shows a typical inclinometer installed within a stub pier. This inclinometer casing was anchored to the inside of the steel beam's web and flange before the steel was lowered into the pier excavation and then backfilled with concrete. In some cases, the contractor elected to fill the pier excavation above the top-of-steel with low strength flowable fill (which was allowed).

Figure 5.c. shows an inclinometer installed during construction and located <u>downslope</u> of the Stub Piers. Therefore, the inclinometer shown on Figure 5.c. was in an area of the slope left unsupported. Data suggests slight creep movement on the downhill side of the stub piers and a shear plane located at the soil / bedrock interface.

From the three installed earth pressure cells, measured horizontal earth pressures six years after construction ranged from 954 to 2708 lbs./sq. ft. While these devices were installed at relatively close spacing and similar depths, we suspect two of the devices may have rotated before being seated at the bottom of the borehole and therefore would not have had their sensors laying perpendicular to the slope forces. In any event, the maximum measured value (2708 lbs./sq. ft.) compared more closely to the assumed earth pressure based on conventional earth pressure theory. The original design assumptions generated a theoretical earth pressure of about 3470 lbs./sq. ft.

Theoretical analyses used conventional earth pressure theory to assume lateral earth pressure distribution on the stub piers. Also recall that vertical soil arching was assumed adding applied lateral pressure to a height of one pier diameter above the top-of-steel. LPILE analyses were conducted to determine the required pier size and steel reinforcement during design.

Comparisons were made between the maximum bending moments and average earth pressures between the original theoretical design analyses and those estimated from measured strain gage data. Results of those comparisons are described below.

Strain gages were installed on Stub Pier steel at Piers 96 and 110. For each of these piers, the depth to bedrock was over 40 feet. The measured or "apparent" strain was converted to bending strain by subtracting the calculated compressive strain due to the weight of the pier above (carried by steel and concrete) from the measured apparent strain. The bending stress and bending moment were then computed from the bending strain value at each strain gage location. The computed bending moments based on these measured strains were well below theoretical estimates generated from the original LPILE analyses. For example, bending moments on the tension side of the steel were computed from strain gage readings to be approximately 25 percent of the LPILE results. Additionally, the strain gage data generated bending moments significantly higher on the tension side than the compression side of the steel. One potential explanation could be that the concrete contribution in resisting bending is neglected in the analysis.

One significant inconsistency in the strain gage data occurs when earth pressures are back-calculated from the computed bending moments. These earth pressures are a fraction of those generated by earth pressure theory and are also well below those measured in the three earth pressure cells. There is no clear explanation for these results.

While there are some inconsistencies in the back-calculated bending moments and earth pressures, the overall monitoring program results suggest that the Stub Pier approach achieved the goal of creating short-term stabilization of the roadway embankment and may in fact provide much longer-term stabilization of this slope.

The owner (ODOT) realized a successful repair solution because the repair was designed and constructed quickly, where the 154 stub piers were installed and the roadway repaved in under 3 months time. The costs were significantly less than the alternative of a tieback-anchored drilled pier arrangement. A tieback approach would likely have involved excavating and installing multiple rows of tiebacks due to the 56 ft. (maximum) depth between ground level and the shear plane. Excavation materials would have had to be removed from the site to avoid stockpile loads, only to be returned later for burying the deeper tiebacks. A much longer construction period would have been required at significant inconvenience to roadway users. Finally, a tieback anchor and drilled pier approach cost was estimated to be about 3 to 4 times the cost of the constructed stub pier approach. Finally, the stub pier approach appears to be functioning well after six years and may provide many more years of support, exceeding the original goal of providing a “short-term” solution.

REFERENCES

- Journal
  1. Hagerty, J. D., Sharifounnasab, M, and Spoor, M. F., (1983). "Riverbank Erosion – A Case Study." *Bulletin of the Association of Engineering Geologists*, Vol. XX, No. 4, pp. 411 – 437.

- Website
  1. U. S. Geological Survey, Ohio River at Cincinnati, No. 03255000. http://waterdata.usgs.gov/oh/nwis/dv?cb_00065=on&cb_00060.

  2. National Oceanic and Atmospheric Administration, Local Climatological Data for Cincinnati, Ohio. http://www.erh.noaa.gov/er/iln/climo/cvg.

Case Studies in Roadway Landslide Repair along Stream Sides, River Banks, Bluffs, and other Sensitive Riparian Areas

Colby Barrett
Soil Nail Launcher, Inc.
P.O. Box 4709
Grand Junction CO 81502
(303)-909-6083
colby@SNL1.com

James Chinchiolo
Landslide Solutions, Inc.
9362 Cedarview Way
Elk Grove, CA 95758
(916)-698-5133
James@landslidesolutions.com

Cameron Lobato
GeoStabilization, Inc.
2841 North Avenue
Grand Junction CO 81501
(970)-985-1066
Cameron@gsi3.com

Nathan Beard
Soil Nail Launcher, Inc.
P.O. Box 4709
Grand Junction CO 81502
(970)-985-9473
Nate@SNL1.com

Prepared for the 62nd Highway Geology Symposium, July 2011

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

Landslides and severe erosion along our nation's inland waterways and coastlines can present a real challenge to engineers, planners and designers. Not only are the regulatory hurdles associated with construction in sensitive riparian and coastal zones significant, many of the common tools available either present a large environmental impact or are not robust enough to handle the corrosion, scour, rapid drawdown, wave action and other erosion processes associated with riparian and coastal sites.

In the context of case studies from four riparian/coastal projects across the United States, this presentation outlines some of the new and innovative erosion control and landslide mitigation methods that are both robust enough to survive riparian/coastal conditions and specific enough to prevent unnecessary environmental impact. Relevant technologies include traditional soil nailing; high capacity tensioned wire mesh with vegetative inclusions and/or seeded turf reinforcement mats; reinforced architecturally sculpted shotcrete; scour micropiles; and fiberglass composite launched (or ballistic) soil nails/horizontal drains.

The case studies for this presentation include a Pacific coastal bluff erosion project using fiberglass ballistic soil nails and architecturally sculpted/stained shotcrete near Crescent City, California; a large landslide repair using integrated soil nails and reticulated micropiles near Pescadero, California; a lakeshore bluff stabilization along the shores of Lake Tahoe, California; and a riparian landslide repair in Virginia.

INTRODUCTION

Landslides and severe erosion are common in riparian areas. Often these sites are associated with weak alluvial deposits with a low associated slope factor of safety. Continual erosion along the base of slopes due to wave action and scour, water infiltration, and rapid drawdown often cause global factors of safety to drop below unity, causing often catastrophic failure.

Ironically, riparian and coastal sites are also some of the most difficult areas to repair due to the possibility of adverse environmental impact from reconstruction/remediation activities. Permitting, water quality concerns and site access restrictions often preclude the use of more traditional landslide repair methods such as rock buttresses, mass excavation, piling, etc.

The case studies outlined below are sites for which designers chose to use newer tools and technologies to repair the areas while minimizing environmental impact and regulatory permitting requirements.

Coastal Erosion Mitigation Using Fiberglass Launched Soil Nails, Crescent City, CA

Two sites were stabilized in unconsolidated terrace deposits overlying the St. George Siltstone. The southern site consisted of a 36 foot high sand bluff overlying siltstone while the northern site consisted of 6 to 8 feet of stiff, high plasticity clay overlying a non-durable siltstone cliff. The siltstone was subjected to “fresh water” wet-dry cycles because it is set back farther from the still water high tide level than adjacent cliff sites. Field evidence and laboratory testing suggest that fresh water wet-dry cycles were more likely to cause slaking and degradation.

The stiff clay within the upper slope is susceptible to toppling failures and slumps initiated along tension cracks in the stiff clay. Water accumulating in these tension cracks may trigger failure. Debris from these failures generally falls directly to the beach below eliminating any stabilizing effect of the slump debris. Failure in the stiff clay causes the most rapid observable cliff top recession. Failures in the siltstone occur as structurally controlled wedge failures or block falls as the properties of the intact rock mass degrade due to wet-dry cycles. The overall recession rate was 0.28 feet per year (8.5 cm/yr) for the period 1972 to 2005.

Figure 1 -- Northern Site Prior to Repair

Figure 2 -- Southern Site Prior to Repair

Traditional repairs in the area consisted of quarry stone revetment (large, angular, durable rock), which prevents direct wave attack against the cliff and acts as stabilization berms. This repair is both simple and economical, but requires direct contact with the fronting beach and therefore requires an extensive permit process prior to construction. In most cases, the California Coastal Commission disallows this repair as it is seen to degrade the scenic qualities of the shoreline.

The non-traditional repair at these sites consisted of ballistic soil nails faced with geologically sculpted shotcrete. The ballistic nails were perforated, thick-walled 1.5” O.D. fiberglass tubes shot into the ground, pressure grouted, which then accepted a 0.625”ϕ epoxy coated threaded bar in the tube annulus.

Figure 3 – Launching Soil Nails at the Northern Site

Fiberglass soil nails were chosen due to their resistance to corrosion. Grout take was high at the sand bluff with significant permeation of the adjacent ground, which increased bulk soil properties and bond strength. Sculpted and stained shotcrete tied to the nail tips provided a structural solution that matched the existing bluff color and texture.

Figure 4 – Completed Northern Site

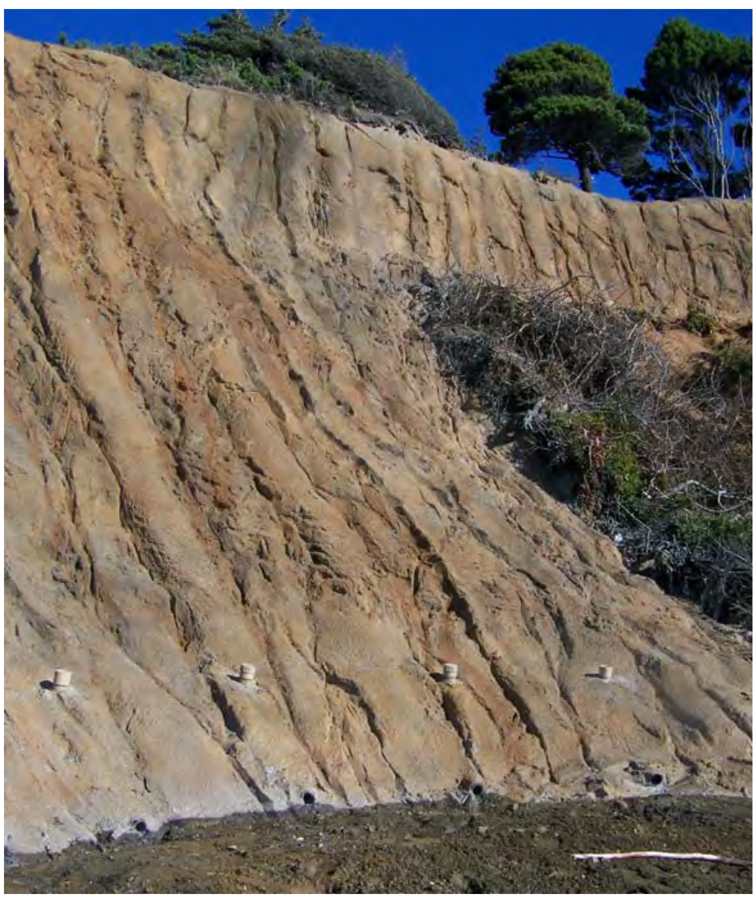


Figure 5 – Completed Southern Site

Both sites have performed well over the past three years despite being hit by two Pacific tsunamis during that time period.

Landslide Repair using Hollow-Core Soil Nails and Reticulated Micropiles, Pescadero, CA

A large landslide along Highway 1 in Pescadero, CA threatened to carry a large portion of the coastal bluff and highway into the ocean below. Caltrans officials were unable to obtain permits to realign the roadway, and all slide repair construction had to be conducted inside the Caltrans right-of-way.

Figure 6 – Scarp Cracking Extended across the Centerline

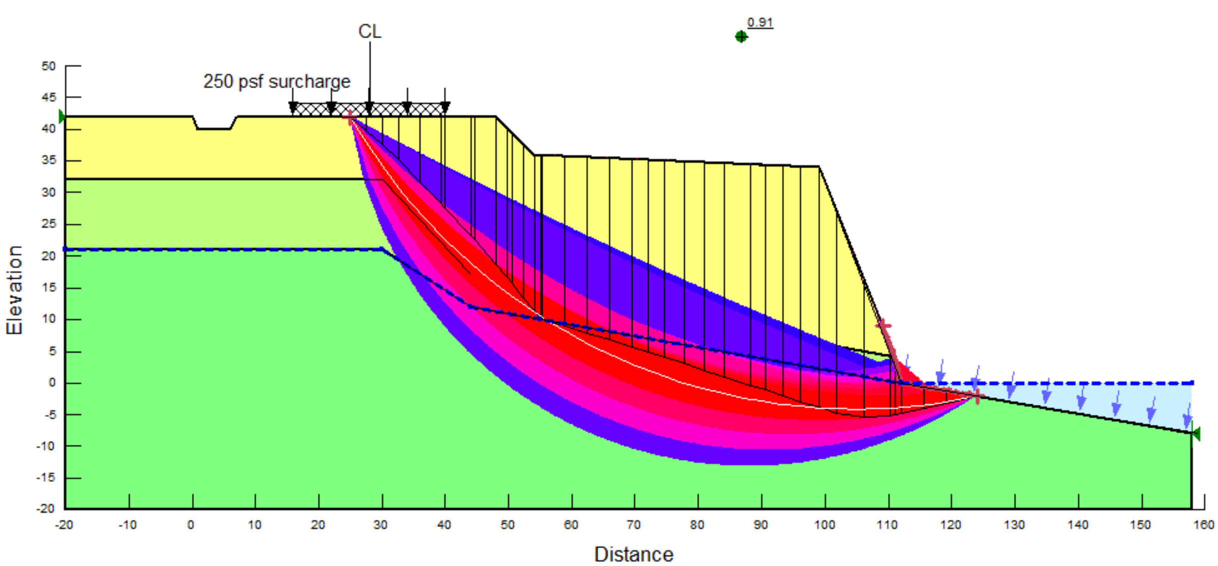


Figure 7 – Existing Failure Surface

Boring logs showed a series of mudstone, shale, and sandstone overlain by approximately ten feet of sand and silt. Inclinometer data showed that the slip plane at the outboard edge of pavement was in a weathered mudstone formation at approximately 28 feet deep. Within two weeks the inclinometer had moved 1.4 inches and subsequently sheared.

Traditional repairs would include a piling wall that would have cost approximately three million dollars and take more than two years to approve and construct. The alternate stabilization system described below took approximately five weeks to construct at a cost of approximately $550,000. Total project cost was under 1.2 million dollars with a total construction time of five months.

An array of 50-ft long drilled injection anchors, tied to an integral reticulated micropile array was installed to prevent further movement. After construction was completed, the entire structural solution was buried and later reseeded with native vegetation. All operations were constructed within the Caltrans right-of-way, did not require road realignment and maintained one lane of traffic throughout the project.

Figure 8 – Area after Repair

Post construction inclinometer data show that the rate of movement along the original slip plane has decreased from approximately 36 inches per year to 0.075 inches per year. Much of this movement can be attributed to the anchor and micropile array assuming the loads from the slide mass.

Lake Tahoe California, Lake Shore Shoreline Stabilization and Restoration

Bluff erosion and bank instability is an ever present problem along the shoreline of Lake Tahoe. At this site along Tahoe's west shore, an estate reconstruction plan included a housing structure just feet from the top edge of an unstable sand bluff, with areas at vertical and even negative batter.

Figure 9 – Lake View of Unstable Bluff Face

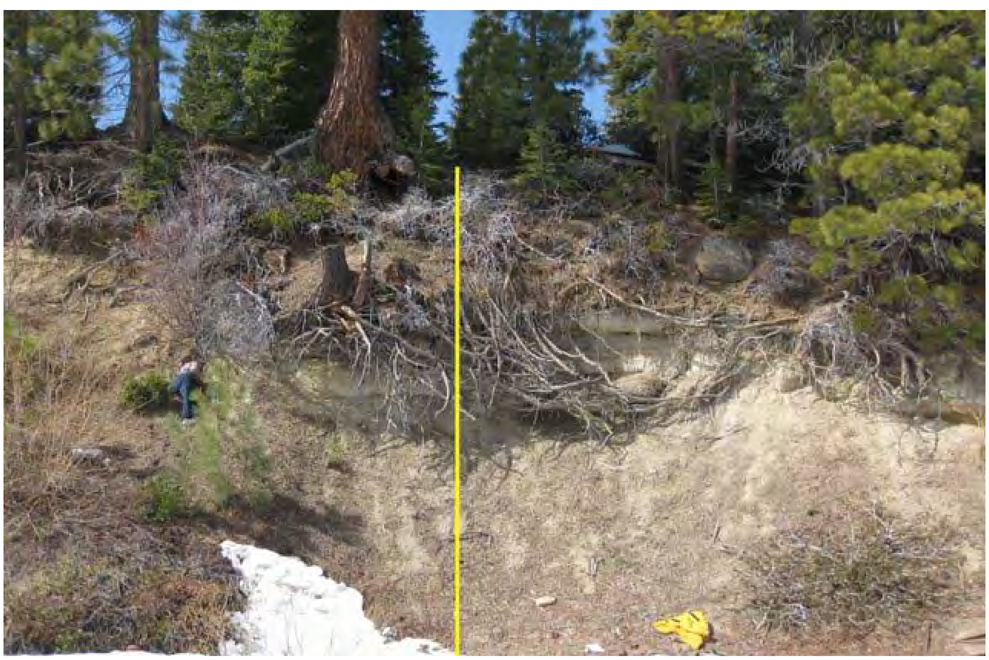


Figure 10 – Area of Severe Undercutting

Traditional repairs would have included mass re-grading to flatten the slope, rip-rap revetment, and a piling foundation for the housing structure. Stringent Tahoe Regional Planning Agency (TRPA) and local county permit requirements precluded any solution that would produce anything other than a "natural" bluff face or any activities that would impact lake water quality.

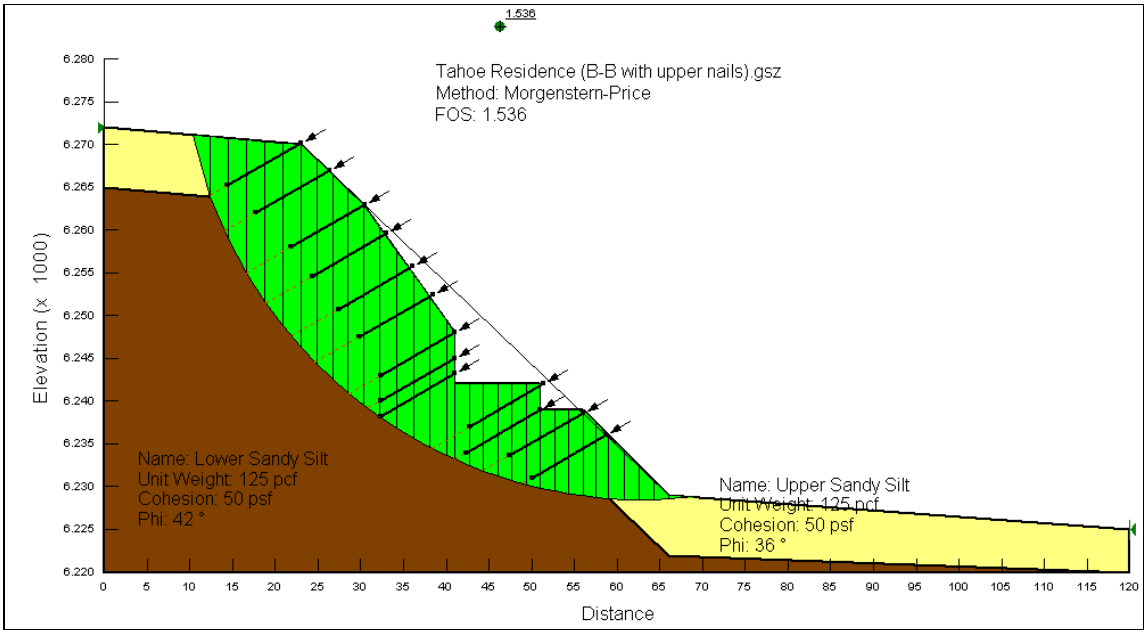


Figure 11 – Slope Stability Modeling with Launched Soil Nails

Due to the sandy silt bluff material, required vertical faces of up to six feet, and the semi-arid Lake Tahoe environment, designers chose a combination biotechnical and structural solution. The structural portion of the project consisted of launched galvanized steel soil nails that were pressure grouted after installation and then faced with a Zinc-Aluminum mischmetal alloy coated steel rockfall mesh which was post-tensioned against the slope face. The slope was then faced with pre-vegetated mats consisting of biodegradable coconut fiber geowebbing filled with a proprietary soil mix and planted with native vegetation appropriate for the high mountain desert area of Lake Tahoe.

Figure 12 – During Construction: After Installation of Launched Soil Nails Faced with High-Capacity Steel Mesh (Left) and Beginning to Install Biodegradable Coconut Coir Geoweb (Right)

Figure 13 – Completed Slope

Road Width Restoration, Route 621 near Craig Creek, Virginia

A steep creek bank along Route 621 near Craig Creek, Virginia slipped along an adverse dipped shale outcrop into the stream below. The Virginia Department of Transportation needed to rebuild their roadway shoulder without impacting the creek below and satisfy the adjacent landowner who had voiced concerns about the aesthetic and environmental impacts of any proposed repair.

Figure 14 – Project before Construction

Design engineers developed a plan that called for a negative batter micropile wall with integral tiebacks. This solution provided approximately six feet of additional shoulder but only extended five feet down the slope, thereby avoiding the riparian area below and lessening the visual impact to the adjacent landowner.

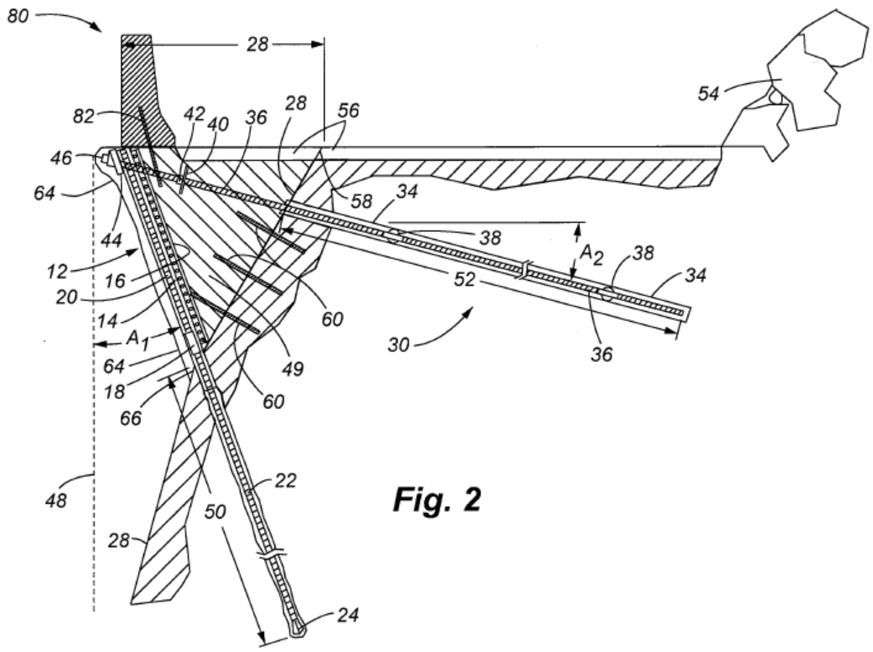


Figure 15 – Schematic of Negative Batter Micropile Wall (Adapted from U.S. Patent Application #12,785,321)

As a “first-of-its-kind” project, VDOT and the contractor teamed up to develop a method to rapidly construct the wall, which posed a real challenge due to the wall alignment being nearly 30 degrees out of vertical plumb. Construction proceeded rapidly and the final product met both the environmental and aesthetic site constraints.

Figure 16 – Wall after Construction

Note that from the roadway platform, the wall does not appear any different from a vertical wall. Similarly, from the vantage point of the landowner who views the wall from across the creek, the negative batter is not apparent

Figure 17 – Six Foot Wide Roadway Shoulder after Construction

Figure 18 – View of Wall from Adjacent Property

REFERENCES:

1. Chinchiolo, J. Lake Tahoe California Residential Lake Shore Bluff Stabilization and Restoration. In *Proceedings of the 2011 International Erosion Control Association Annual Conference,* Orlando, FL, 2011.
2. Devin, S. A Geomorphic Model for Coastal Cliff Recession near Crescent City, California. In *Conference Proceedings of the 40th Annual Symposium on Engineering Geology and Geotechnical Engineering*, Logan, UT, 2006.

62nd Highway Geology Symposium
Lexington, KY
July 25-27, 2011

The Geology of Kentucky

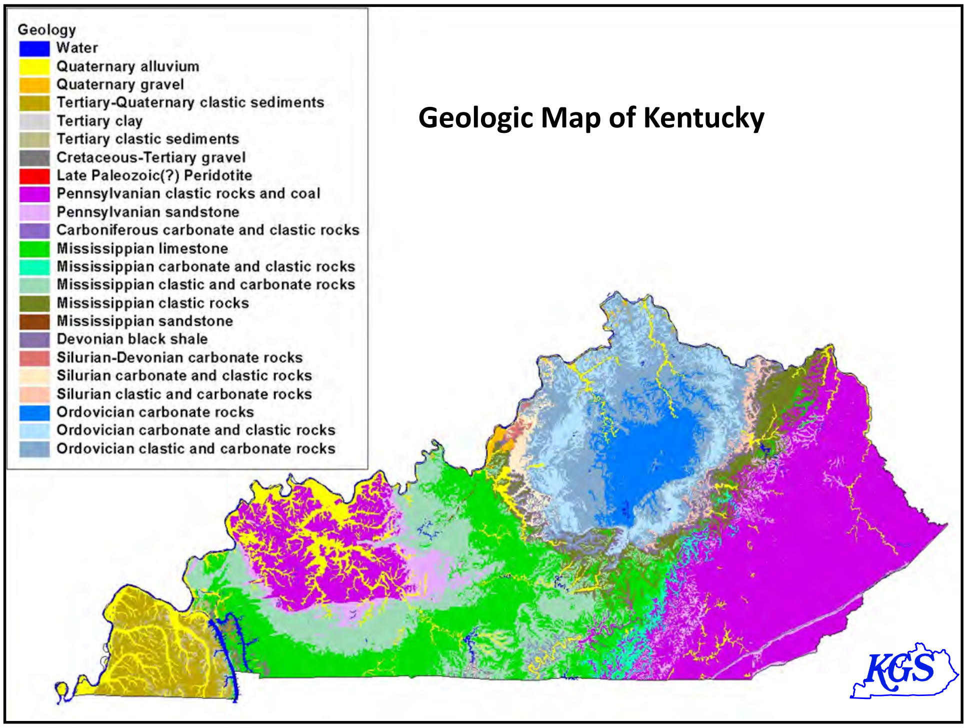
Jim Cobb, State Geologist and Director,
Kentucky Geological Survey,
University of Kentucky

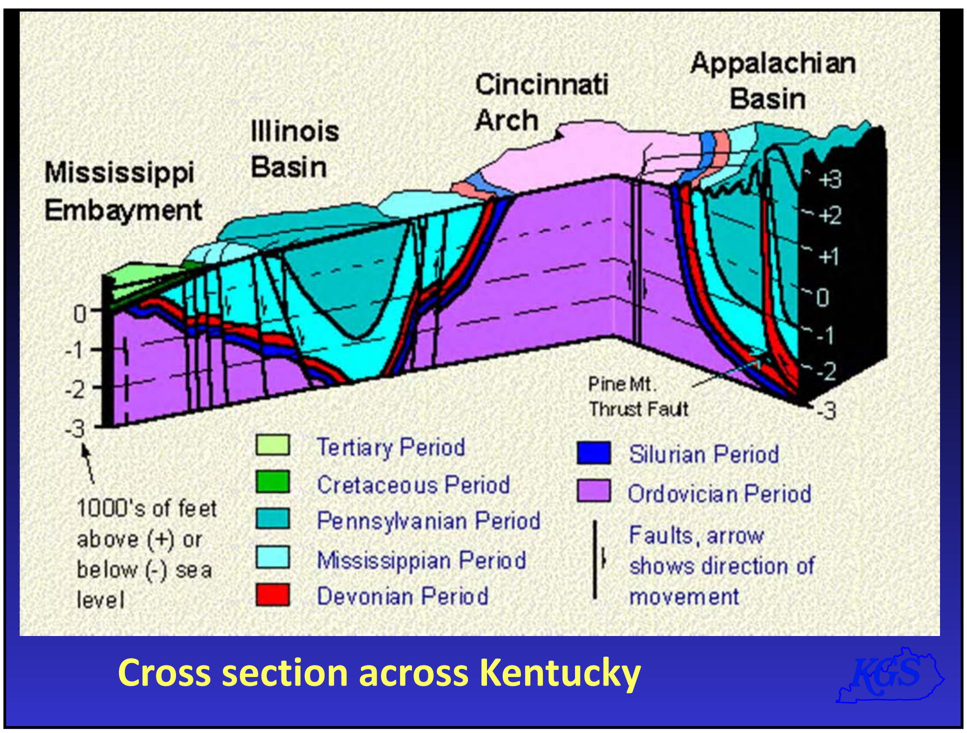
Geology since 1838

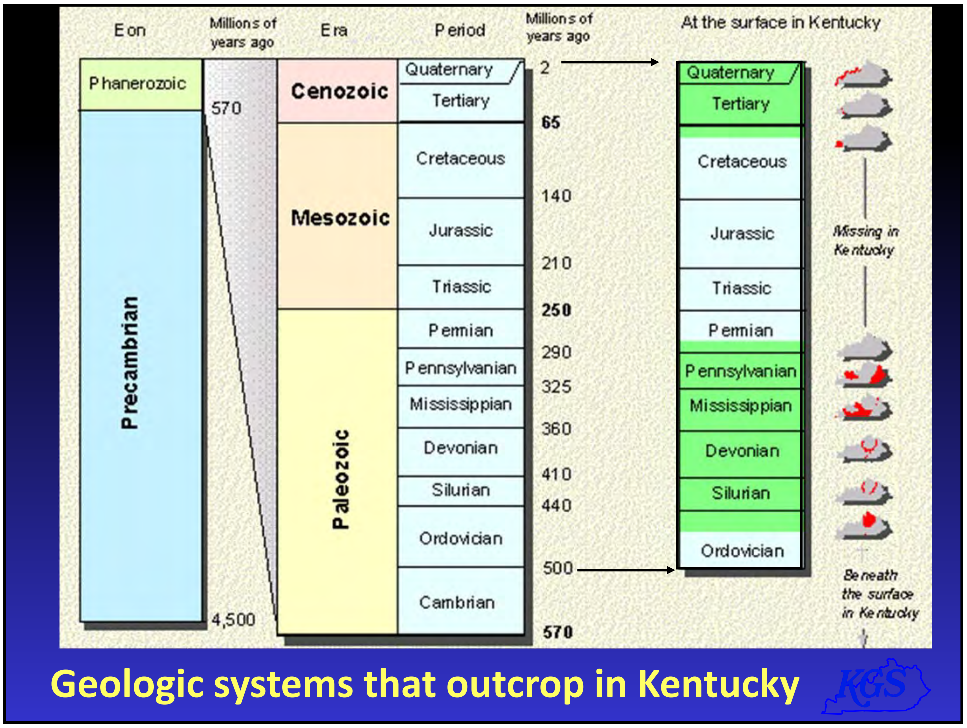
Kentucky Geological Survey, University of Kentucky

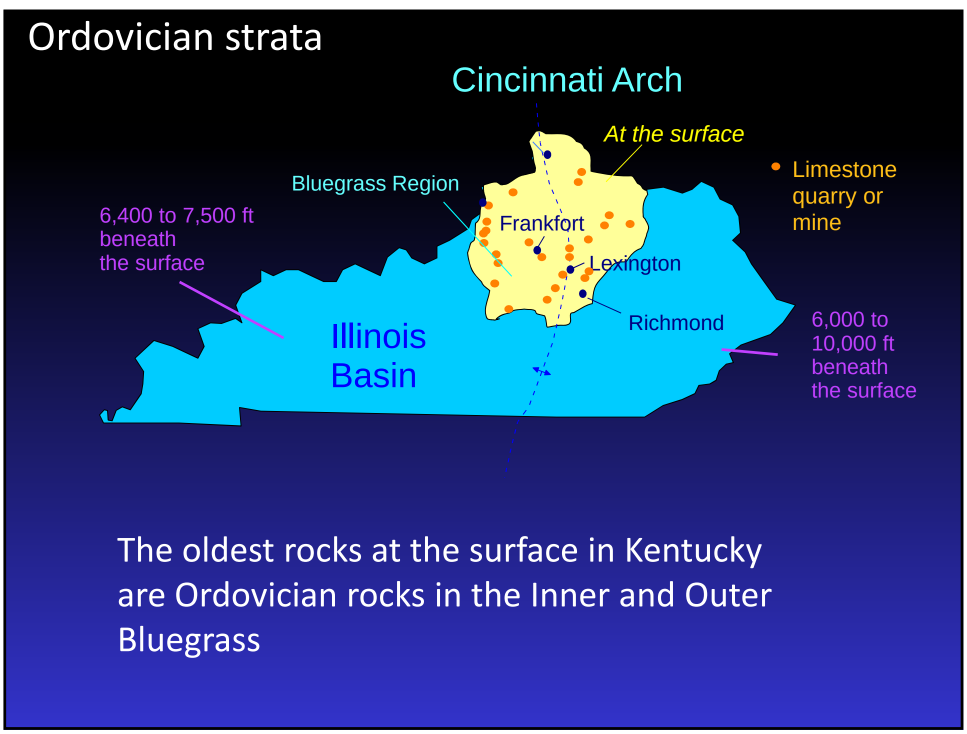
Geology and Physiography

NCGMP Mapping Progress


Geology
Water
Quaternary alluvium
Quaternary gravel
Tertiary-Quaternary clastic sediments
Tertiary clay
Tertiary clastic sediments
Cretaceous-Tertiary gravel
Late Paleozoic(?) Peridotite
Pennsylvanian clastic rocks and coal
Pennsylvanian sandstone
Carboniferous carbonate and clastic rocks
Mississippian limestone
Mississippian carbonate and clastic rocks
Mississippian clastic and carbonate rocks
Mississippian clastic rocks
Mississippian sandstone
Devonian black shale
Silurian-Devonian carbonate rocks
Silurian carbonate and clastic rocks
Silurian clastic and carbonate rocks
Ordovician carbonate rocks
Ordovician carbonate and clastic rocks
Ordovician clastic and carbonate rocks
Geologic Map of Kentucky
KGS


Mississippi Embayment
Illinois Basin
Cincinnati Arch
Appalachian Basin
+3
+2
+1
0
-1
-2
-3
Pine Mt. Thrust Fault
1000's of feet above (+) or below (-) sea level
Tertiary Period
Cretaceous Period
Pennsylvanian Period
Mississippian Period
Devonian Period
Silurian Period
Ordovician Period
Faults, arrow shows direction of movement
Cross section across Kentucky
KGS


Eon
Millions of years ago
Era
Period
Millions of years ago
At the surface in Kentucky
Phanerozoic
Precambrian
570
4,500
Cenozoic
Mesozoic
Paleozoic
Quaternary
Tertiary
Cretaceous
Jurassic
Triassic
Permian
Pennsylvanian
Mississippian
Devonian
Silurian
Ordovician
Cambrian
2
65
140
210
250
290
325
360
410
440
500
570
Quaternary
Tertiary
Cretaceous
Jurassic
Triassic
Permian
Pennsylvanian
Mississippian
Devonian
Silurian
Ordovician
Missing in Kentucky
Beneath the surface in Kentucky
Geologic systems that outcrop in Kentucky

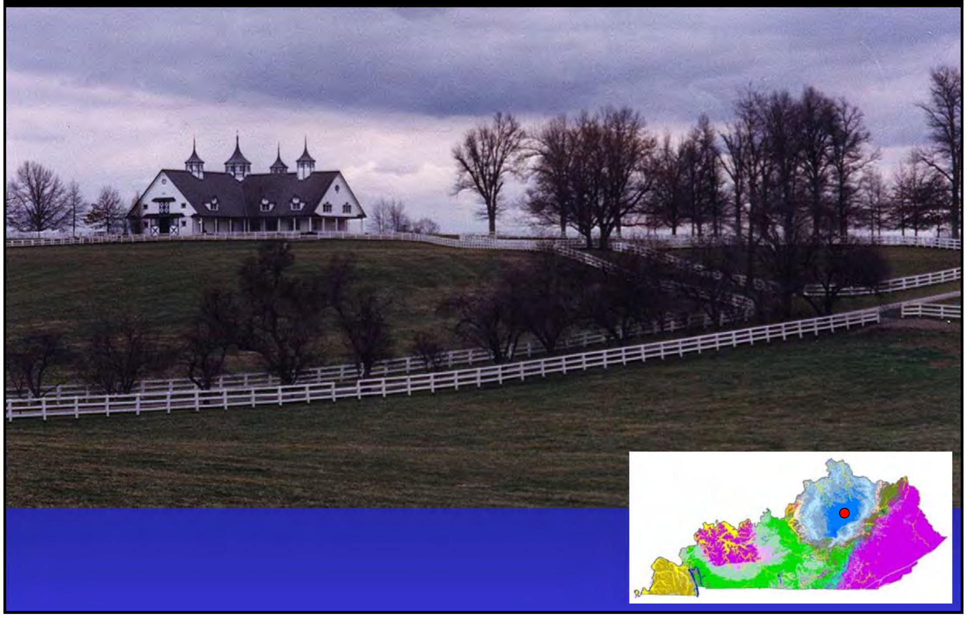

Ordovician strata
Cincinnati Arch
At the surface
Limestone quarry or mine
Bluegrass Region
6,400 to 7,500 ft beneath the surface
Frankfort
Lexington
Richmond
Illinois Basin
6,000 to 10,000 ft beneath the surface
The oldest rocks at the surface in Kentucky are Ordovician rocks in the Inner and Outer Bluegrass

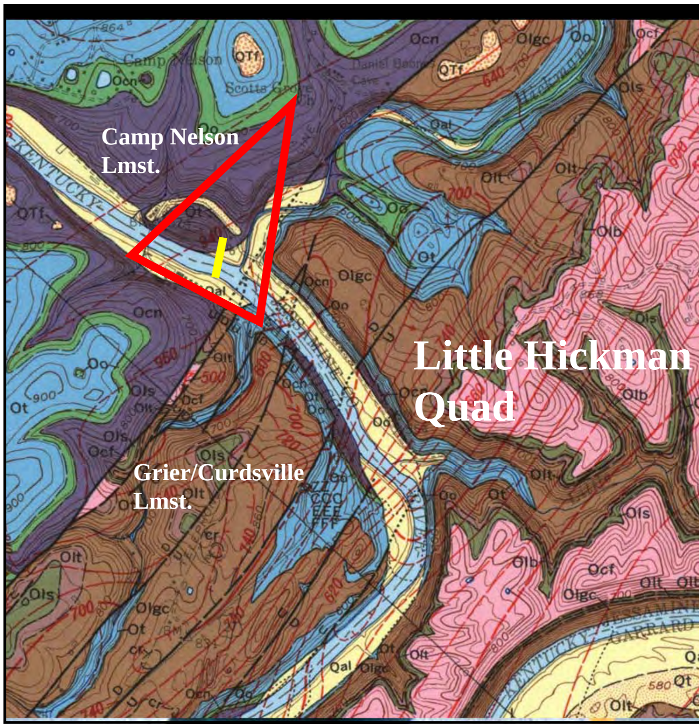
The Bluegrass Region is the Horse Capital of the World because of geology!

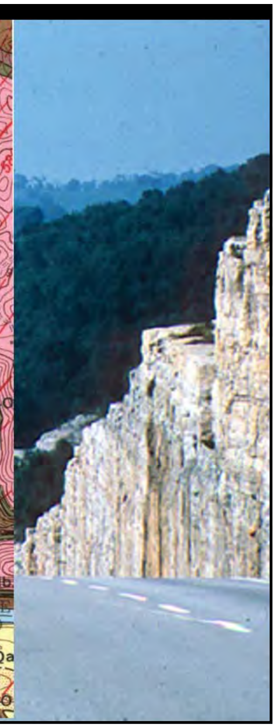
Calcium and phosphate-rich limestone help build strong bones.

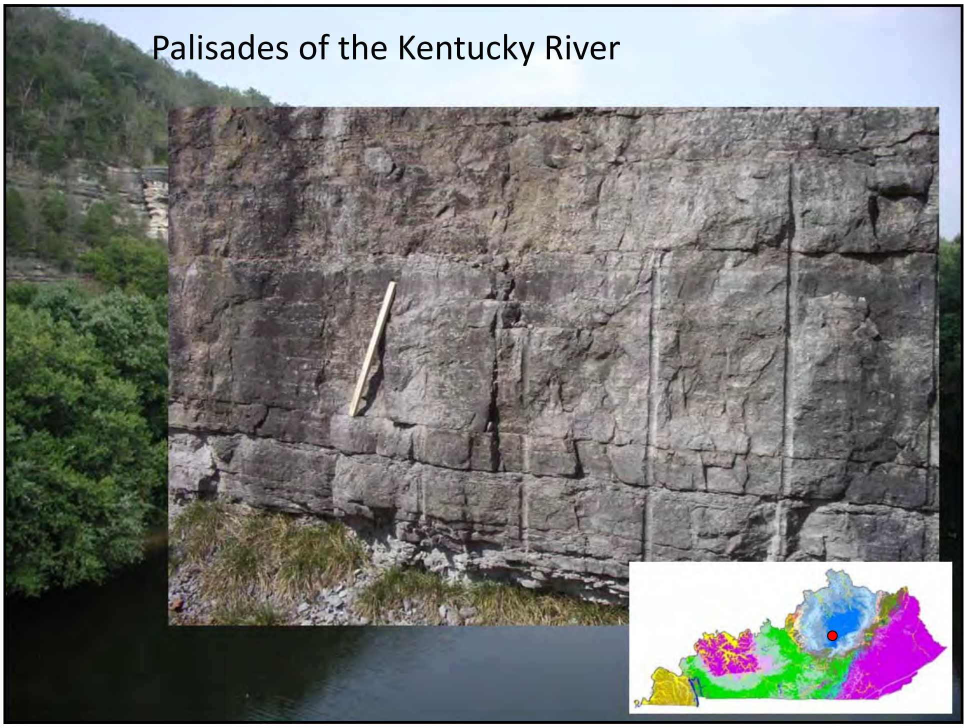
Bluegrass Region whiskey distillery

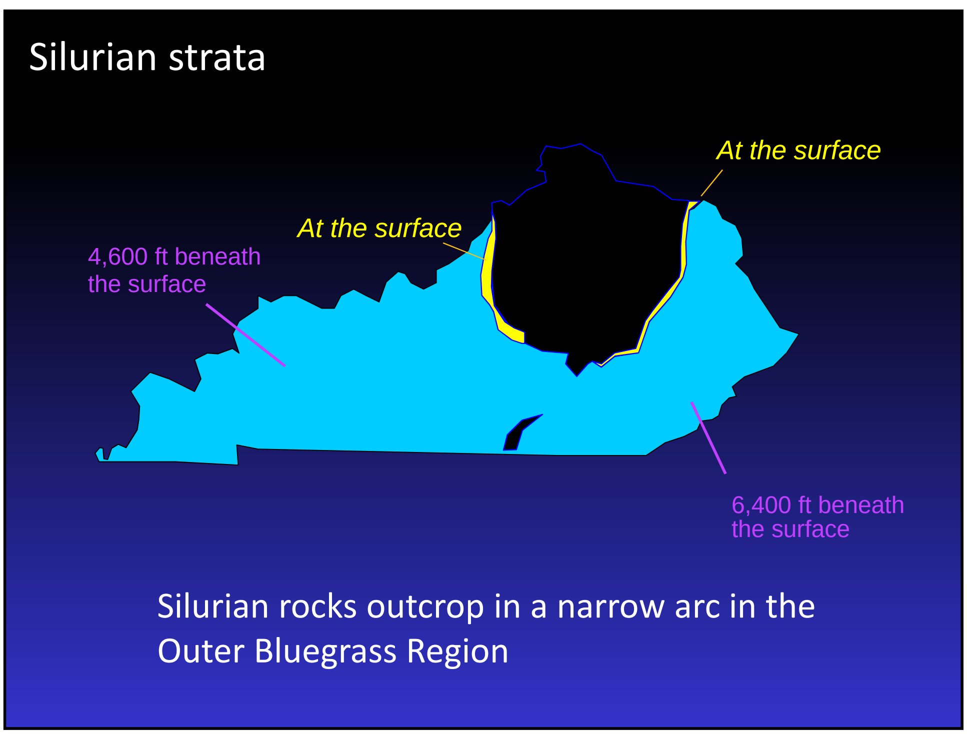
Calumet Farm in Bluegrass region



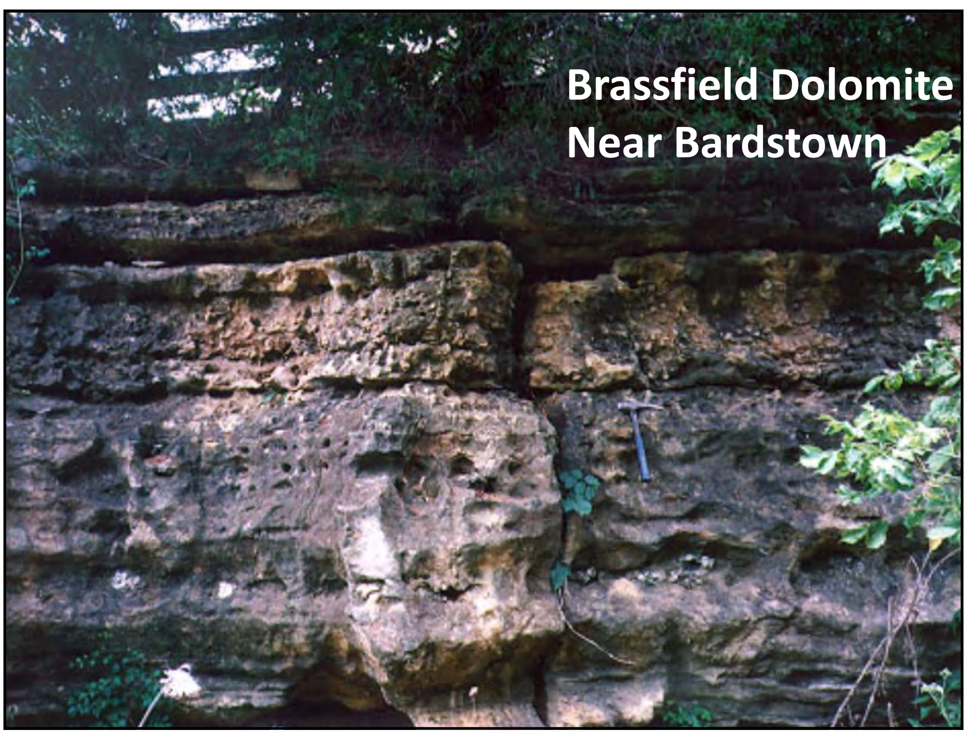




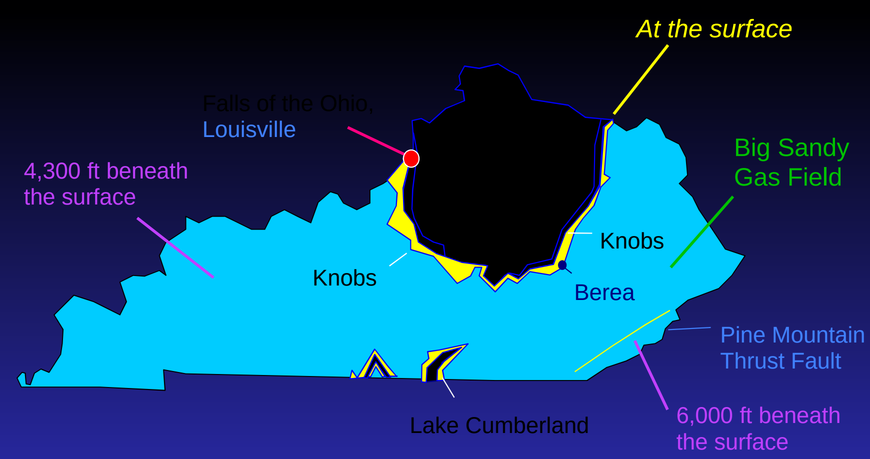

Palisades of the Kentucky River


Silurian strata
At the surface
At the surface
4,600 ft beneath
the surface
6,400 ft beneath
the surface
Silurian rocks outcrop in a narrow arc in the
Outer Bluegrass Region

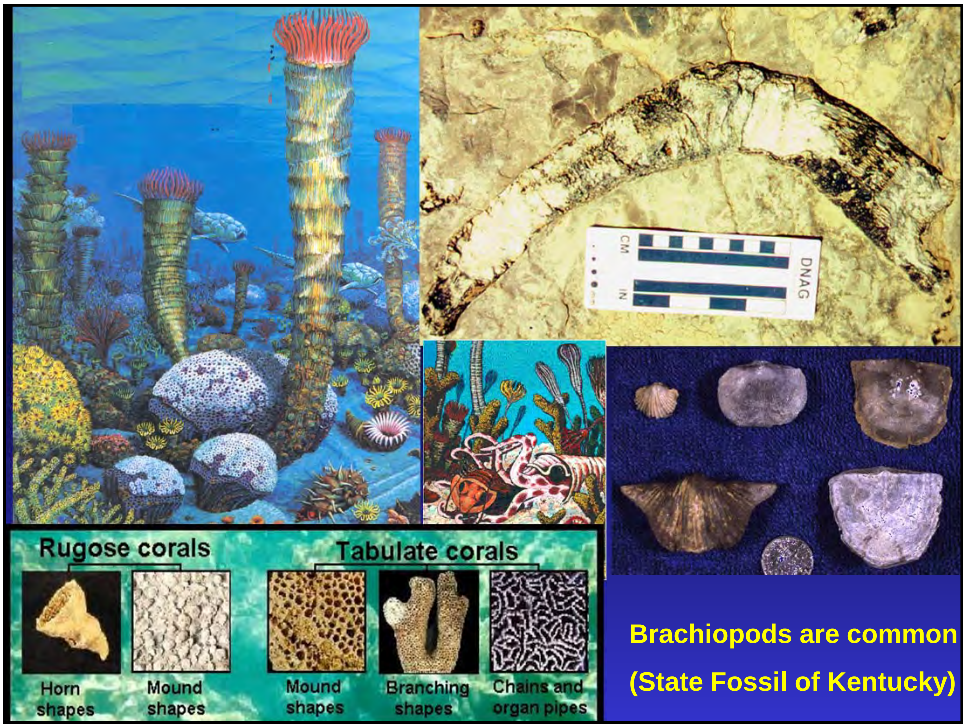

Atlas of Kentucky, 1998

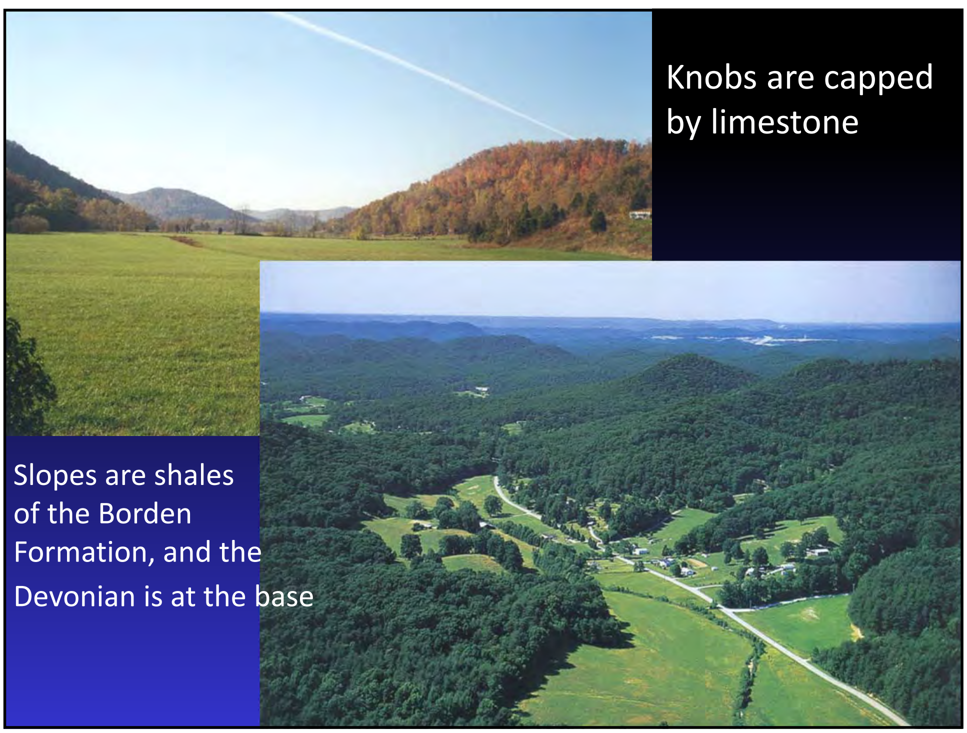

Brassfield Dolomite
Near Bardstown

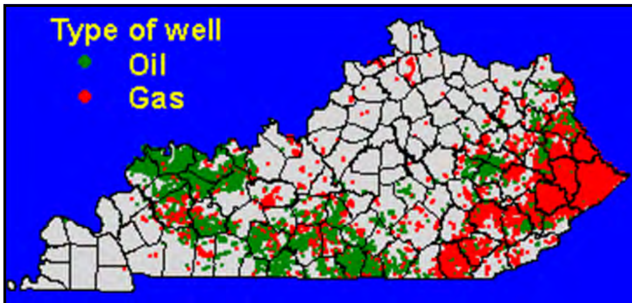
Devonian strata



Devonian rocks outcrop in a narrow ring in the Knobs Region


Rugose corals
Tabulate corals
Horn shapes
Mound shapes
Mound shapes
Branching shapes
Chains and organ pipes
Brachiopods are common
(State Fossil of Kentucky)

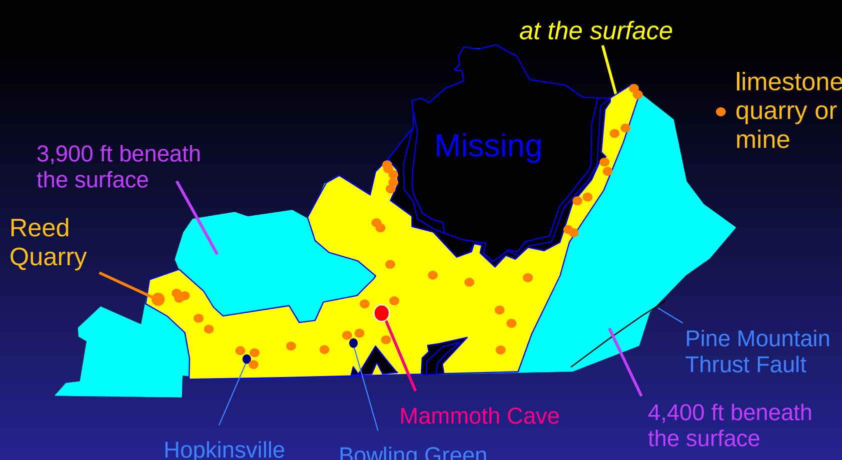

Knobs are capped by limestone
Slopes are shales of the Borden Formation, and the Devonian is at the base



Devonian black shales are the source rock for most of the hydrocarbons produced in the state.

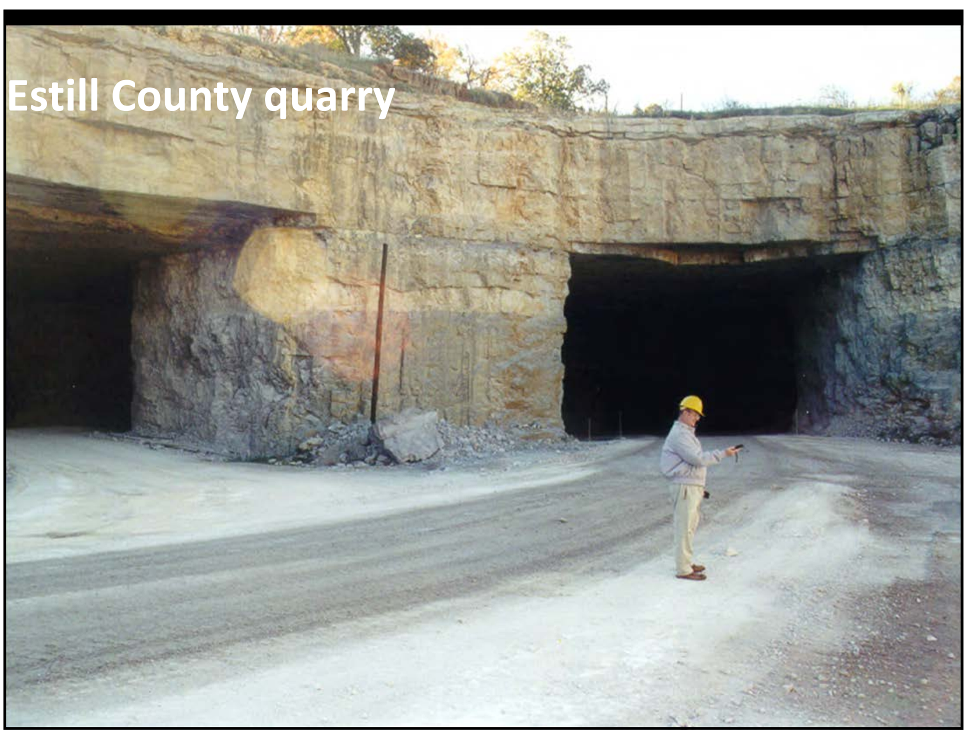
They are also the largest reservoirs of natural gas.

Mississippian strata



Mississippian rocks outcrop at the surface in the Interior Plateau Region


Mammoth Cave
335 miles of mapped passages
Sinkhole Plain
Barren County
Kentucky Atlas, 1998
(c)NPS
Karst Country!
• Springs
• Caves
• Sinkholes
• Sinking Streams
• Karst Windows


Estill County quarry

Bourbon County limestone quarry

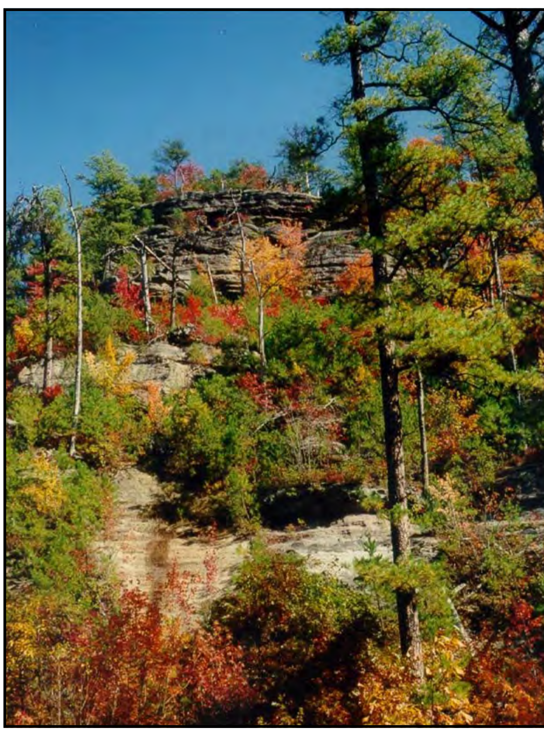
Stone statistics for Kentucky

1. Ky ranks 28th in nation in total nonfuel mineral production, $776 million. 95-100 quarries or mines in state.
2. Kentucky ranked 3rd in nation in lime production.
3. Kentucky ranked 12th in nation in crushed stone.
4. Kentucky ranked 19th in nation in Portland cement.
5. Limestone was the leading non-fuel mineral commodity in Ky, accounting for 53% of the States non-fuel mineral production.
6. Crushed stone used for construction aggregate, cement, chemical stone for power plants, bituminous asphalt.
7. USA Today report "Spending on local Projects Plummets", Mineral Production Value decreased by $30 million in crushed stone and sand and gravel in 2008 over 2007 values.

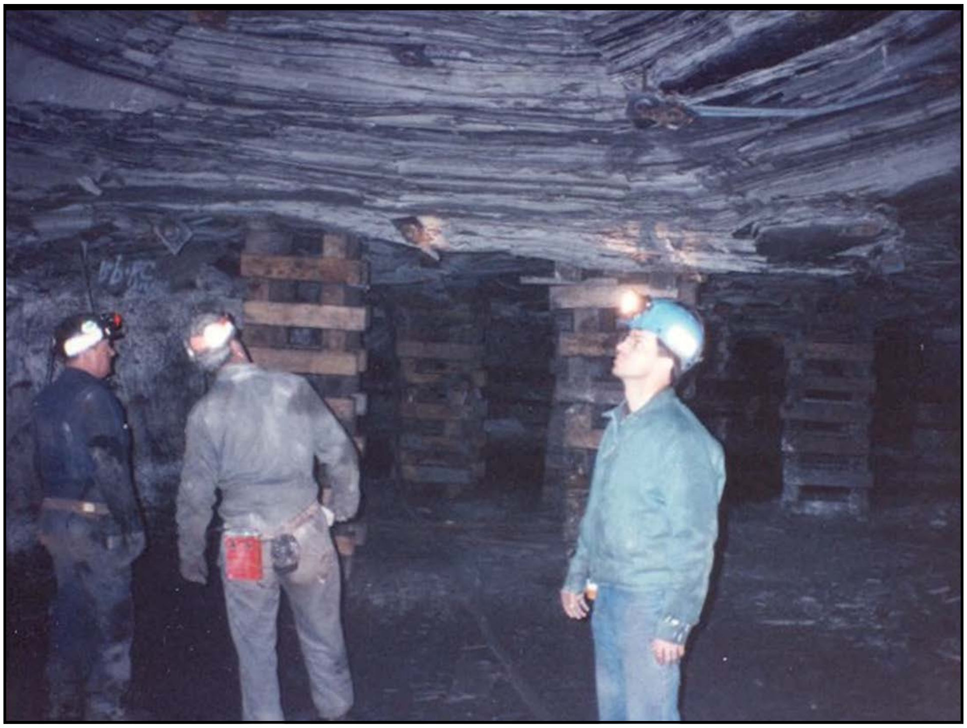
Pennsylvanian strata

360 to 408 mya

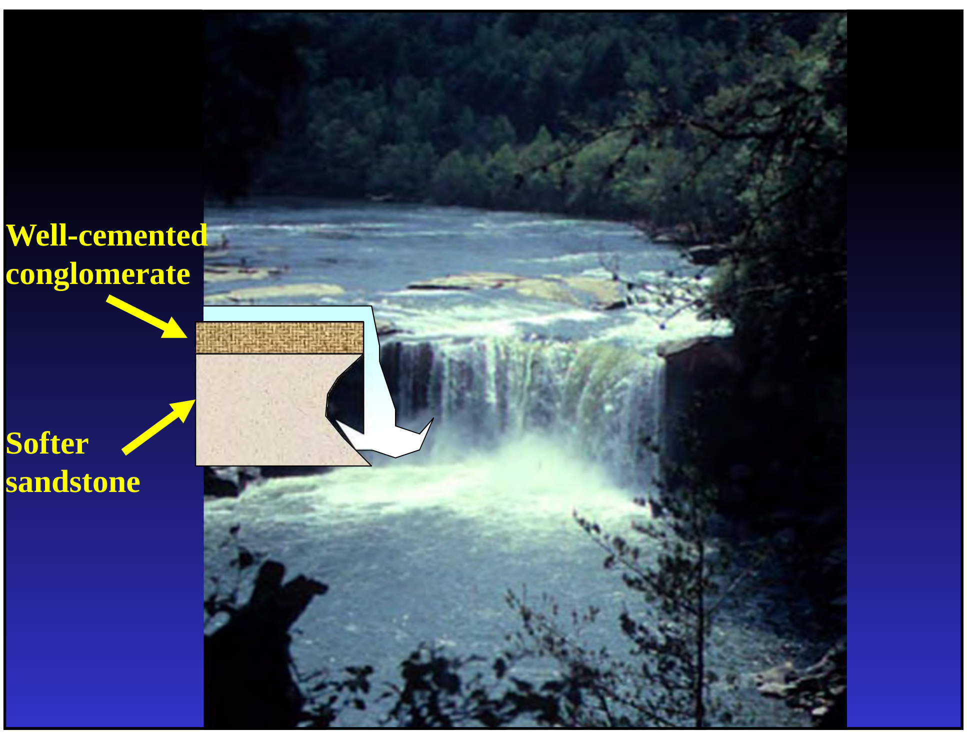
Pennsylvanian shale, sandstone, conglomerate, coal, and siltstone occur within Kentucky's two coal fields.

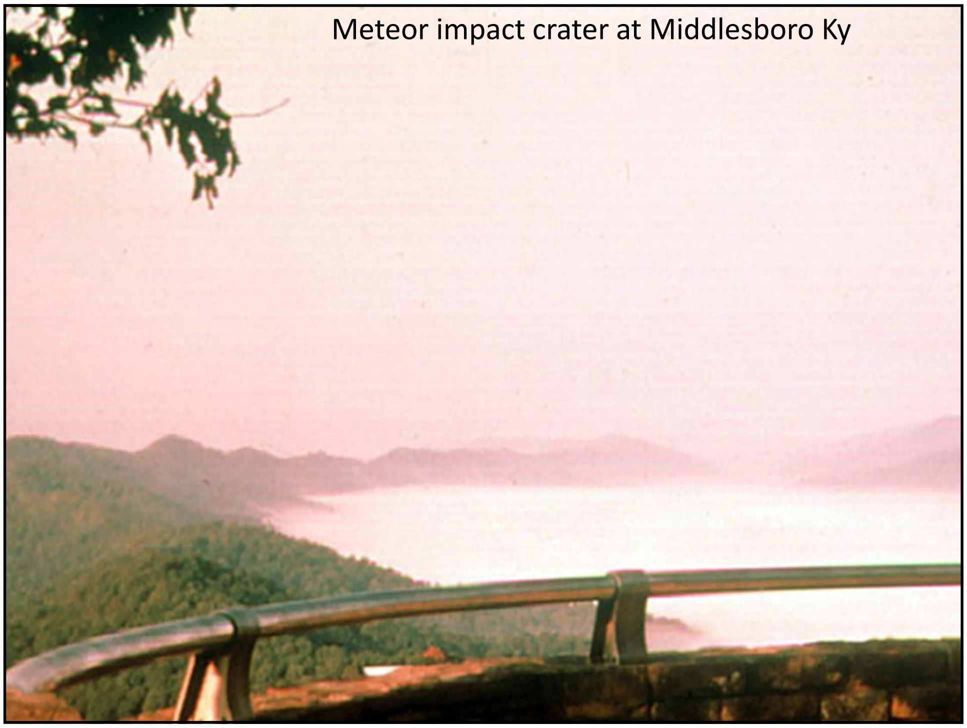


Many of the State's scenic parks owe their scenery to cliffs formed from Lower Pennsylvanian conglomerates.

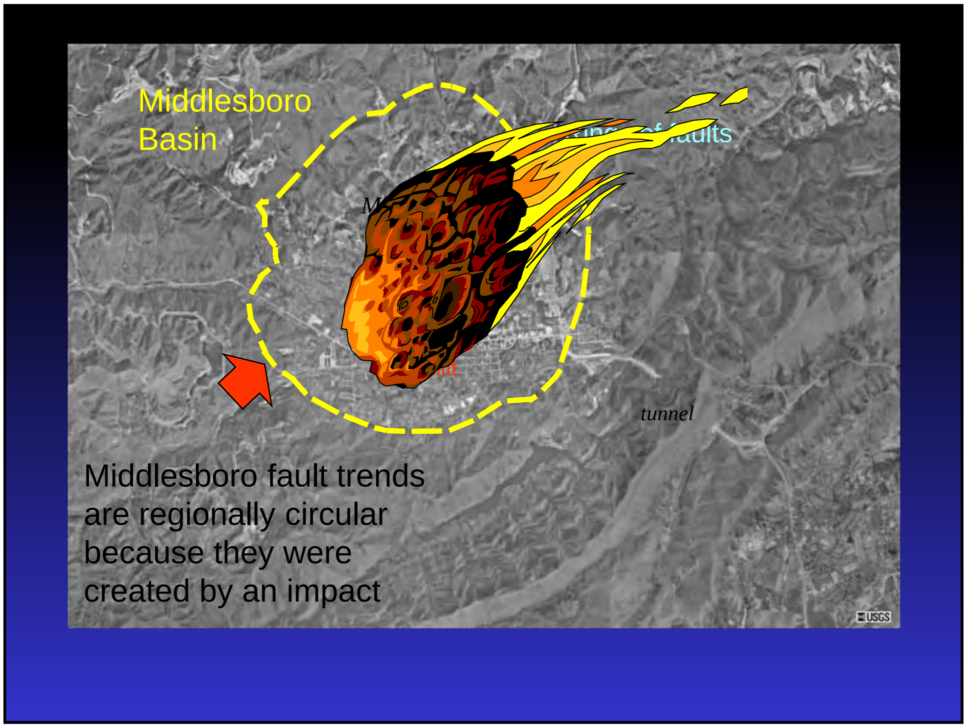


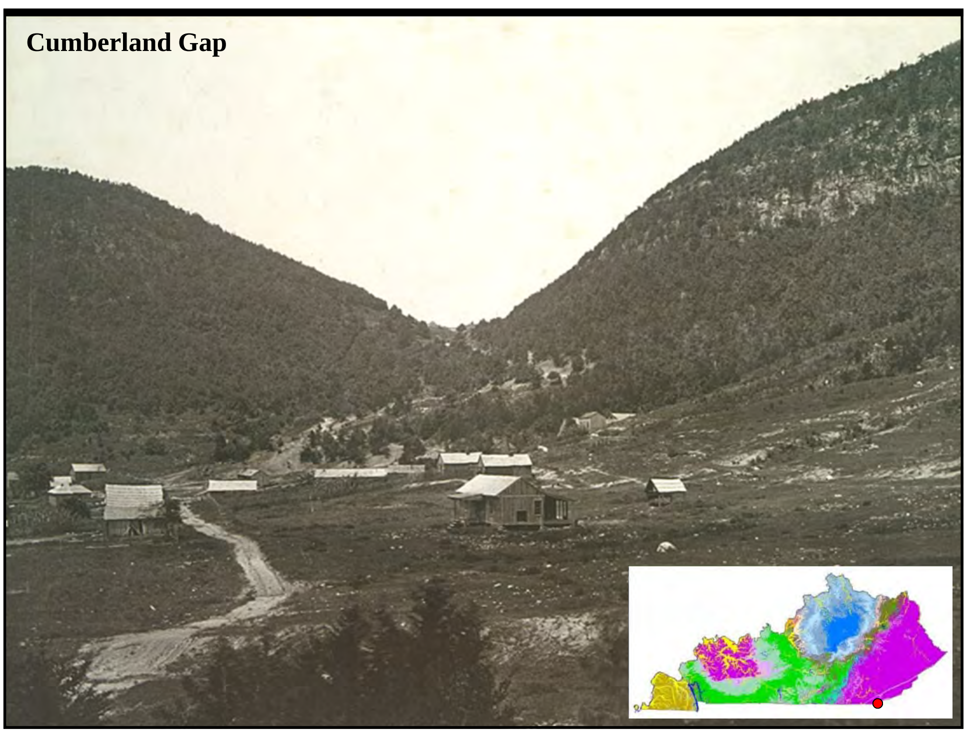
Edestus – Pennsylvanian shark jaw found above the Springfield coal in Webster Co. KY, 2011

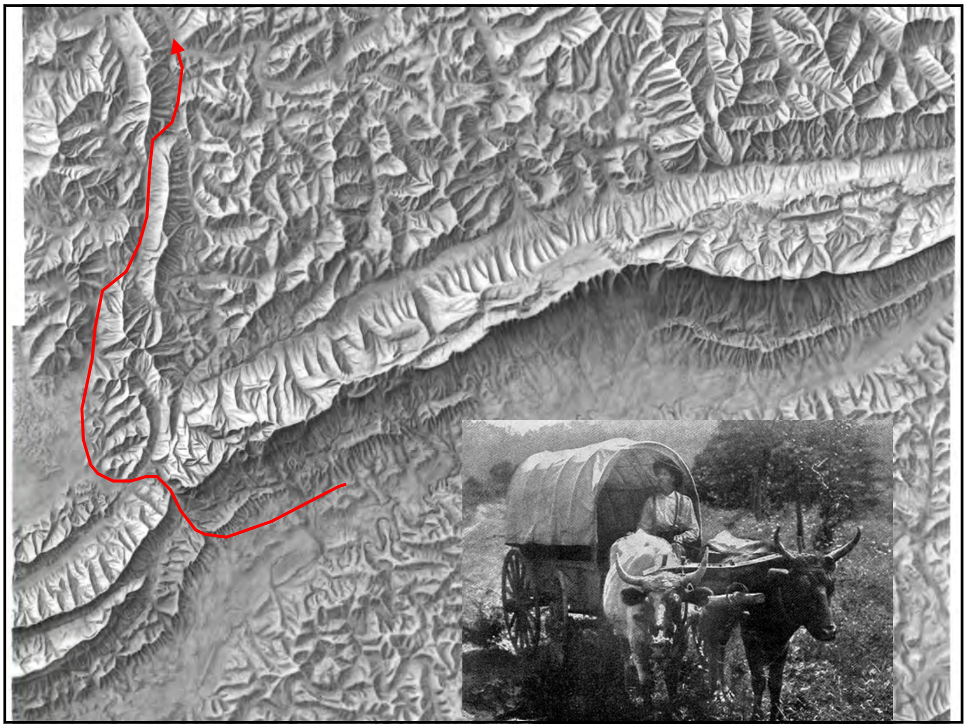

Well-cemented
conglomerate
Softer
sandstone

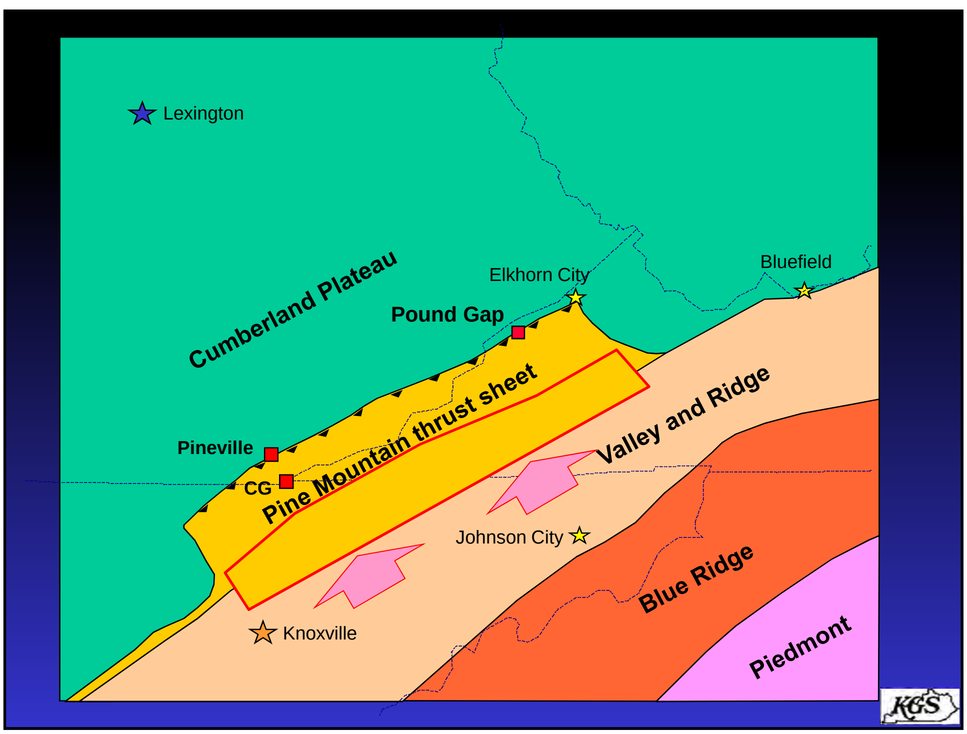

Meteor impact crater at Middlesboro Ky

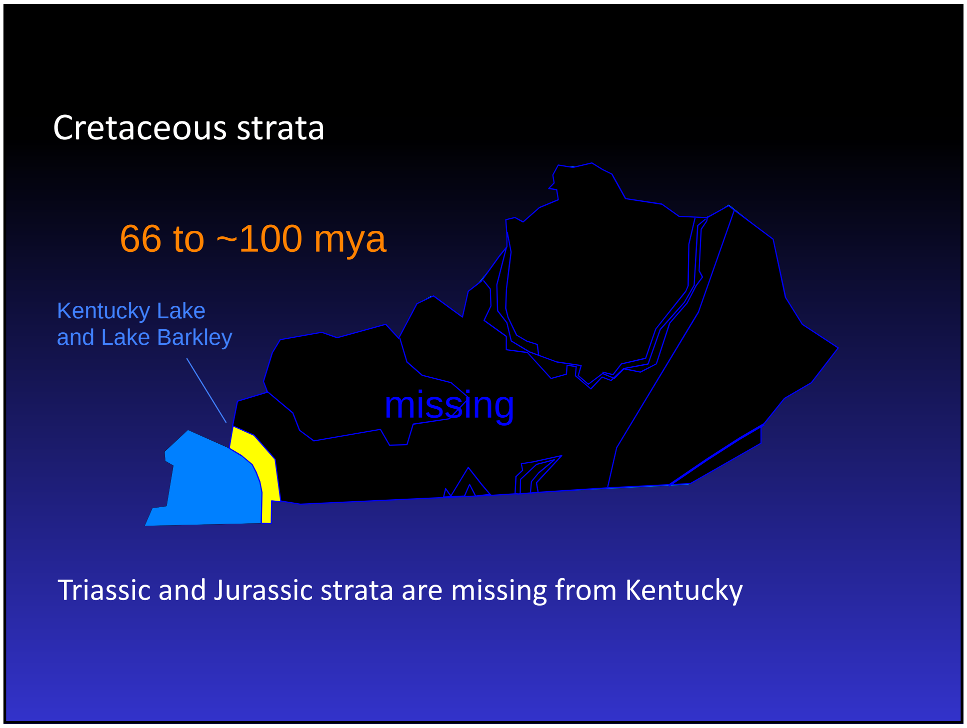

MIDDLESBORO
BASIN

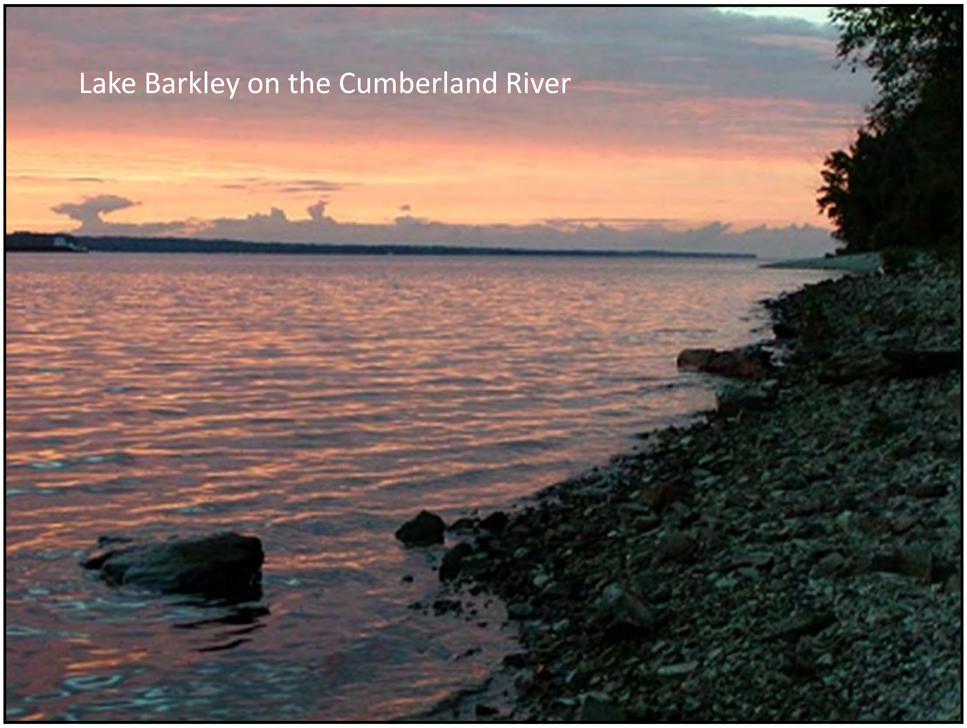

Middlesboro
Basin
tunnel
Middlesboro fault trends
are regionally circular
because they were
created by an impact

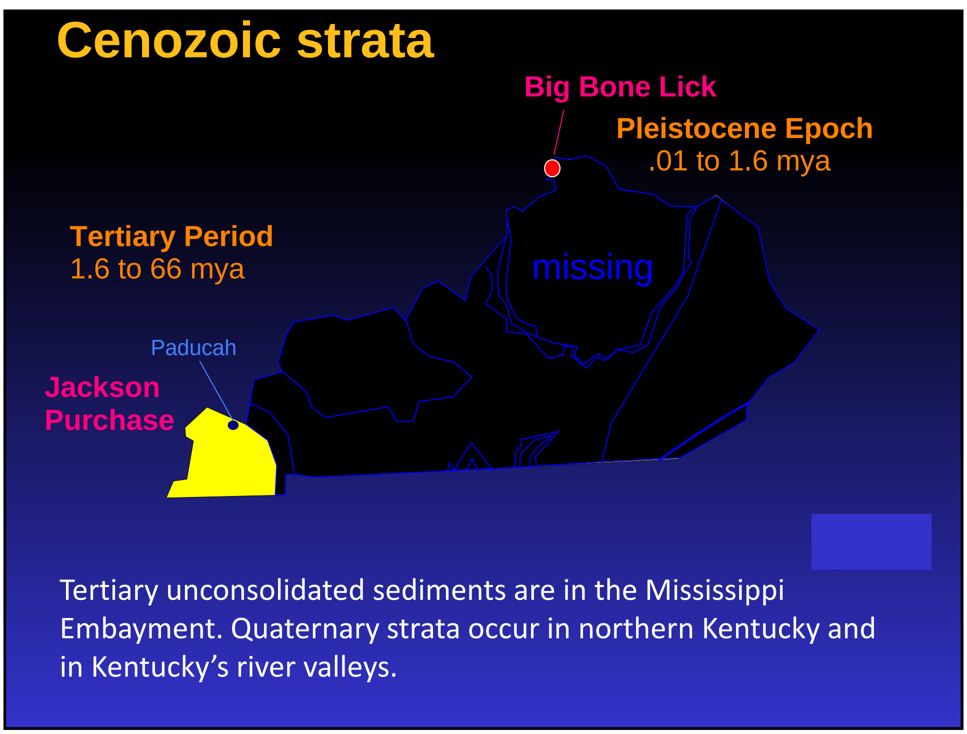

Cumberland Gap




Lexington
Cumberland Plateau
Elkhorn City
Bluefield
Pound Gap
Pine Mountain thrust sheet
Pineville
CG
Valley and Ridge
Johnson City
Knoxville
Blue Ridge
Piedmont
KGS

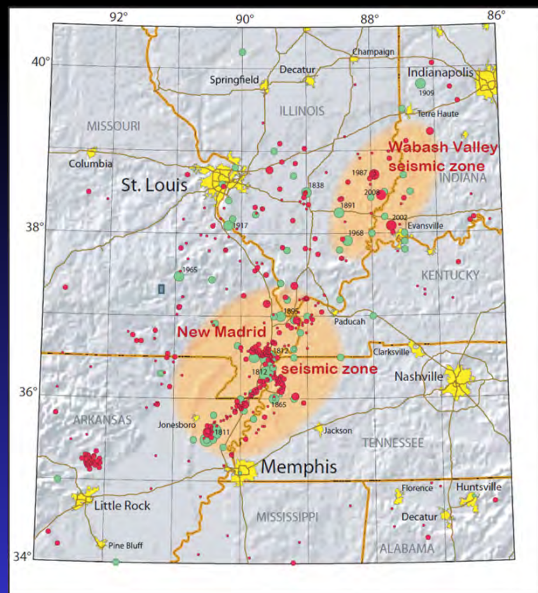

Cretaceous strata
66 to ~100 mya
Kentucky Lake
and Lake Barkley
missing
Triassic and Jurassic strata are missing from Kentucky


Lake Barkley on the Cumberland River


Cenozoic strata
Big Bone Lick
Pleistocene Epoch
.01 to 1.6 mya
Tertiary Period
1.6 to 66 mya
missing
Paducah
Jackson
Purchase
Tertiary unconsolidated sediments are in the Mississippi Embayment. Quaternary strata occur in northern Kentucky and in Kentucky's river valleys.

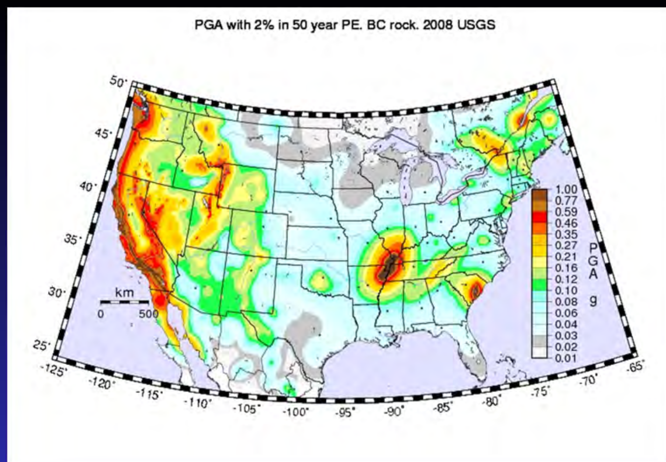
Lignite formed in the Tertiary in oxbow lakes where clays accumulated in thick deposits. Kentucky is the third leading producer of ball clay in the Nation.

New Madrid and Wabash valley Seismic Zones



Reelfoot Lake - Created by the Earthquake of 1811-1812

USGS National Seismic Hazard Map (2% in 50yr.)



www.uky.edu/KGS/

STABILIZATION OF A LARGE WEDGE FAILURE UTILIZING A PASSIVE ANCHOR SYSTEM, INTERSTATE 40, NORTHCAROLINA, PIGEON RIVER GORGE.

Jody Kuhne

North Carolina Dept. of Transportation, Geotechnical Engineering Unit

P.O. Box 3279

Asheville, NC 28802

828-298-3228

jkuhne@ncdot.gov

Shane Clark

North Carolina Dept. of Transportation, Geotechnical Engineering Unit

P.O. Box 3279

Asheville, NC 28802

828-298-3874

scclark@ncdot.gov

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to thank the following for their contribution to this project:

Brendan Fisher, Fisher and Strickler Rock Engineers

Janod, Ltd., Rock Remediation and Slope Stabilization Contractor

Phillips and Jordan, Inc, General Contractor

NCDOT Design Services, Surveying and Division 14 Construction

Ben Rivers, Federal Highways Administration

Carl Fiset, Roctest Geotechnical and Structural Instrumentation

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original authors.

ABSTRACT

On October 25, 2009, a 50,000 yd^3 rockslide blocked all four lanes of Interstate-40 in Western North Carolina at MM 2.5. Investigation determined that a remaining 350,000 yd^3 potential wedge failure would need to be removed or stabilized. After a failed bid for excavation, a contract for tensioned anchor stabilization under a 60-day contract was let. This article explains why design and construction shifted to a passive anchor system, the design considerations selected, utilization of SWEDGE software for design, completion of the project and installation of a monitoring system for future slope conditions and movement. The conclusion shows that a distinct set of site characteristics exists to have confidence in the passive model and solution.

Introduction

October 25, 2009, a 50,000 yd^3 rockslide blocked all four lanes of Interstate 40 at MM 2.5 in Western North Carolina. The incident occurred at 0300 hours with two vehicles striking the debris and resulting in only minor injuries. I-40 carries ADT of 26,000 vehicles with approximately 45% commercial haulers. The Appalachian Regional Commission concluded that the impact of this closure was a strain on global plus local economy of nearly $1,000,000 per day.[1]

Initial Assesment

The Geotechnical Engineering Unit of NCDOT was contacted and made on-site determination of immediate safety and preliminary information about the slide itself. The slide is a large wedge failure, with axis perpendicular to the highway alignment. A 200' repeating skewed tension crack provided the release necessary for failure and the original construction in the 1960's daylighted the toe of the wedge. The wedge continued 785' up the mountainside with approximately 75% of the material remaining suspended. The following Markland plot generated in Rockpack III software shows the highway alignment (the original constructed slope runs SE/NW at 0.75:1.0), wedge intersection, friction circle estimated from remaining intact rock resting on the failure planes and the wedge planes with 7^0 variability in waviness.

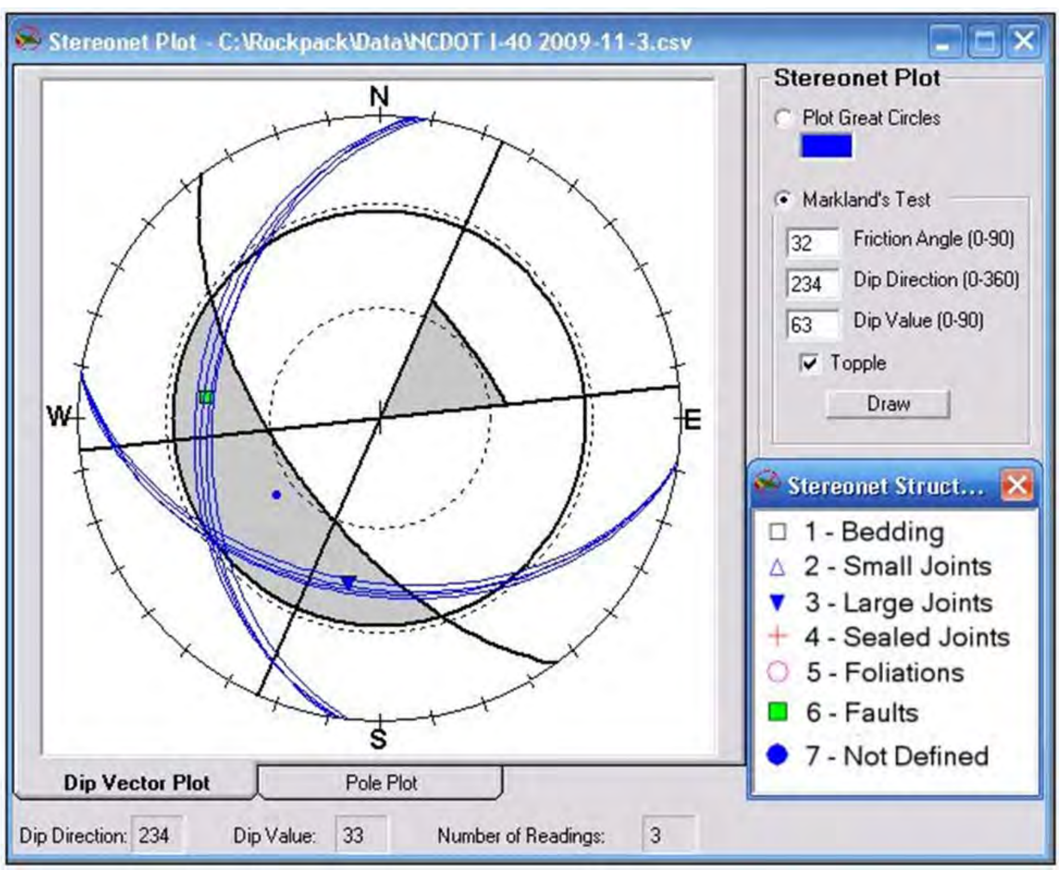


Fig. 1: Markland Plot of Wedge Geometry

The Markland plot shows the wedge sitting at near stability. The structure data was entered in the SWEDGE program and showed the existing FOS at 1.01. The conclusion was that the slide debris could be blasted and excavated with continuous survey and instrumentation with alarms to give warning if the remaining wedge began to fail.

Information including aerial photos, DTM's, survey control, Lidar, 3d modeling and CADD generation of alignments and cross sections were completed. This showed a remaining wedge volume of 350,000 cubic yards.

Fig. 2: Modeled image showing limits and volume of wedge.

Design Process

NCDOT concluded that the final mitigation should achieve the State's engineered cutslope standard FOS of 1.3. Included in the problem was a large, perched colluvial deposit on the right plane. This area had failed in the past and was the only recorded problem in this section in the past few decades. Two solutions were apparent: excavate the wedge limits or mechanically stabilize the mass with anchors.

The rock is massive quartzite with wide joint spacing, large block size and high strength (The Longarm Quartzite member of the Ocoee Supergroup). UC results showed strength of 14 – 25,000 psi. The failure planes were discrete and continuous with the right side showing more weathering tendency (thus, a lower estimated friction angle). A subsurface investigation was not performed because the materials and geometry of the slide were well defined in the field. This left water content as an unknown.

After evaluating cost estimates and future performance it was decided to generate a design and contract for excavation of the entire slide mass. Excavation of the site was risky due to the borderline stability and presented an unknown timeline since the slide mass was inaccessible; with natural slopes from 40^0 to vertical. The sole-bid contract for excavation was rejected by NCDOT as too expensive.

The next approach was a more quantifiable tensioned anchor installation. NCDOT hired Fisher and Strickler Rock Engineers to provide peer review for design. The SWEDGE program allowed for full wedge geometry input, anchor tension and sensitivity modeling of inputs such as water level. Concurrent cleanup with drilling and blasting operations were occurring at the site which generated more information about the capabilities of drills and accessing the wedge. NCDOT determined that 592 anchors of 140 kip capacity would be required to bring the wedge to a FOS of 1.3. The angle of installation for each anchor is preferential parallel to the wedge intersection azimuth and the incline angle was at 30^0. The wedge intersection dips at 35^0 and 30 was the maximum chosen to maintain positive upward tension while minimizing the considerable anchor lengths. Anchors longer than 100' were spaced on the slope in a 12' x12' pattern, lengths under 100' were placed on a 10' x 10' pattern (see Fig. 6).

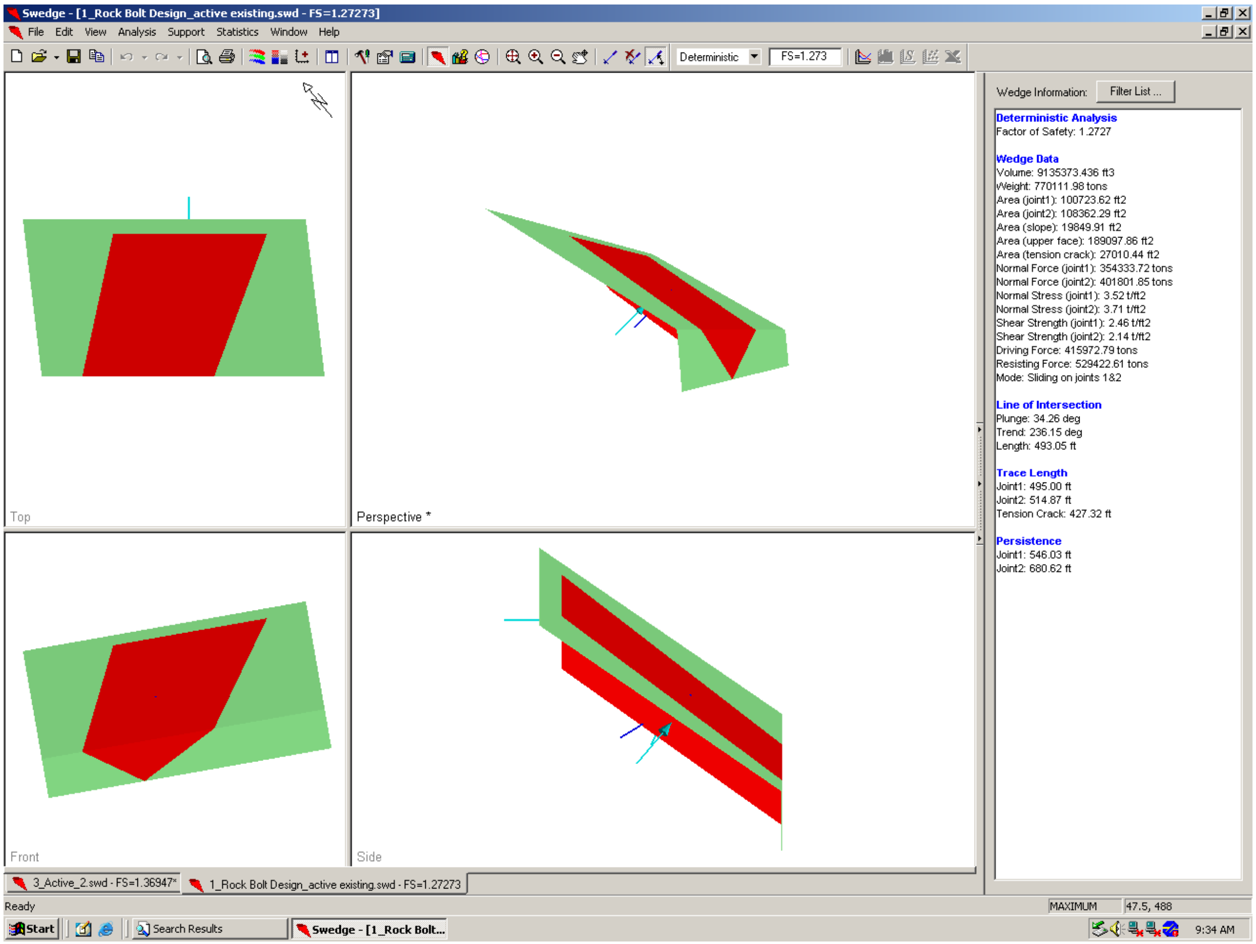


Fig. 3: SWEDGE Display of Wedge Geometry, Dimensions and Forces

Construction

The contract was awarded with a 60-day completion time for approximately 45,000 LF of bar anchor installation. Each hole had to be drilled within 3 degrees (dip and azimuth) to the preferred design orientation. It was immediately proposed to use strand anchors (as opposed to solid bar anchors as designed) due to the inaccessibility of the slope and the length and weight of the anchors, up to 120' and 2,000 lbs. Strand anchors require far more specific tolerances at the head assembly so a great deal of time went into face preparation- namely, sculpting natural rock faces and varied overburden into surfaces that would allow four-strand anchor alignment.

At the 30-day mark in the contract approximately 125 anchors were installed, 2/3rds of that number grouted and 12 tested and accepted. The majority of delay was due to weather and the amount of rock scarifying required for strand anchors. Once enough anchors reached testing stage it was discovered that the strand elongation (by their nature, significantly more than bar anchors) exceeded the jack extension, adding another difficulty to the tensioning.

Due to these and other exterior factors it was apparent that a new design approach might be required.

Redesign Utilizing Passive Anchors

The geology, base stability and unique features of the failure planes appeared conducive to installing passive anchors. That is anchors, which if engaged, would serve as active resistance to further movement but ultimately the durability of the rock plus the waviness and roughness would remain intact. There would be no recognized shear/dowel component of the design.

Two other things were learned during construction at this point: The contractor could drill accurately up to 150' depth utilizing a larger diameter hole than originally specified.

The most valuable resource pertaining to this design was by Spang and Egger, *Action of Fully-Grouted Bolts in Jointed Rock and Factors of Influence*[2]. Passive anchor support requires much higher design strength and corresponding bond length. The SWEDGE program allows for passive design, the particular measureable feature that factors heavily in passive vs. active/tensioned design is the waviness of the failure planes. The Contractor was able to source large enough bar anchor stock and had enough remaining hole locations to provide the design support.

An additional factor in the design switch was to delineate the installed tensioned anchors above from the passive below. The contractor started installation as a top-down operation so it allowed the active anchors (preventing failure) to remain above and the passive anchors (requiring miniscule failure to engage) below.

Fig. 4: Spaced Anchor Location in the Wedge

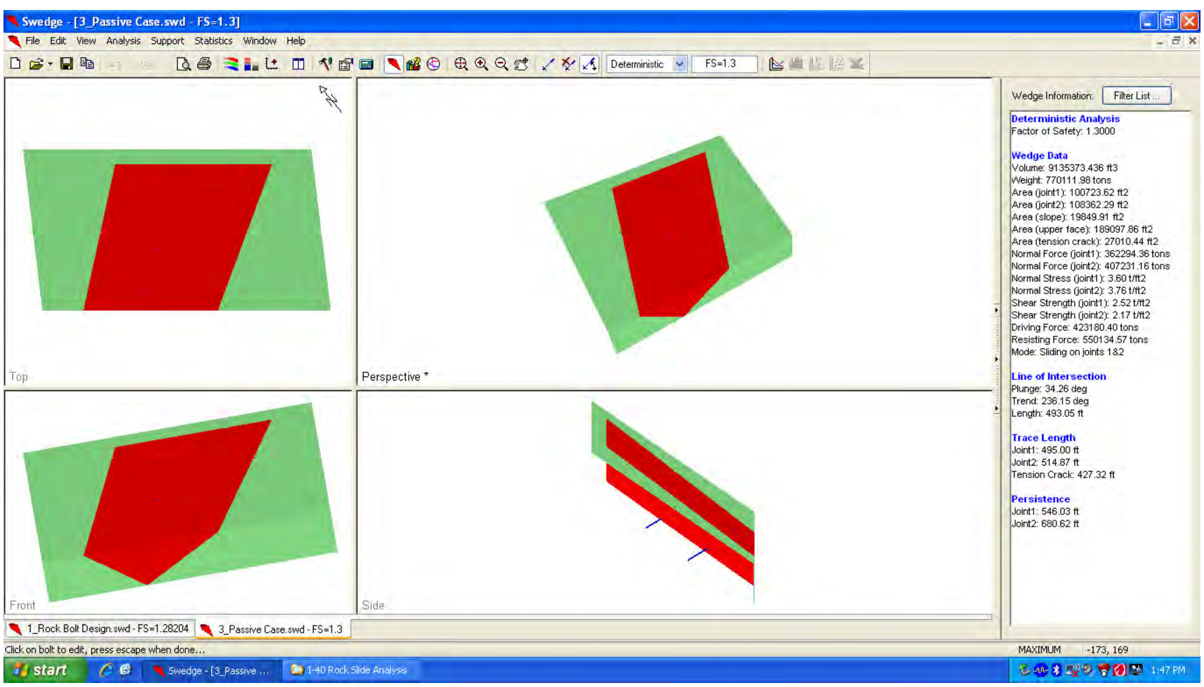


Fig. 5: Passive Anchor Analysis. Shows Installed Upper Active Component and Lower Passive Component

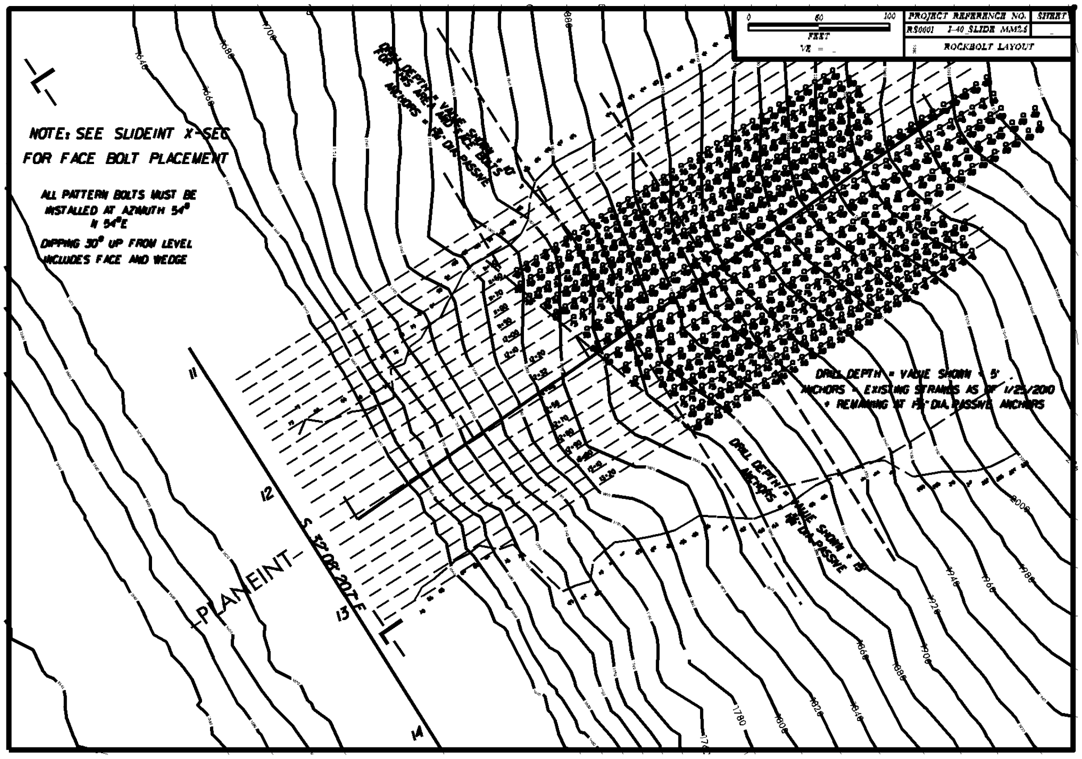


Fig. 6: CADD Depiction and Survey Anchor Locations

Construction, Testing and Monitoring

Construction rapidly advanced from this point forward. Passive anchors have no intrinsic test for integrity besides inspection so four pullout tests were performed to confirm the design rock bond strength of 150 psi. Pullout tests were conducted on short anchor sections to failure and exceeded values of 400 psi.

The FHWA concluded that the final condition should include instrumentation. The slope was fitted with 4 Roctest Bor-ex extensometers, 2 vibrating wire piezometers, 2 assemblies of five strain gauges spaced in proximity to the failure plane and one near the surface at the outer block and 2 Tuff Tilt 801 tiltmeters. This system is wired to a solar powered datalogger with cell phone access.

Conclusions

NCDOT Geotechnical Engineering does not usually design with passive anchor support as the primary philosophy. The following are factors which made passive anchors viable and necessary in this particular case:

1) Construction of the original active anchor design was too slow and pressures to open the highway were too intense to overrun the contract time.

2) NCDOT felt that the highway should have the vast majority of the anchors in before allowing traffic under any capacity.

3) Passive anchor construction is far easier than tensioned.

4) The rock type was durable and massive with high friction angles and waviness on the failure planes.

5) The rock at the failure plane contact was discreet and unlikely to bend or shear anchors before friction was overcome.

6) The drilling and downhole inspection of tensioned anchor holes showed that no movement along the failure plane had occurred- Vital for passive anchor success.

REFERENCES

1) HDR. *Economic Impact of Rockslides in Tennessee and North Carolina.* May 2010. Prepared for the Appalachian Regional Commission.

http://www.regiona.org/rpo/EconomicImpactofRockslidesinTnandNC.pdf

2) K. Sprang and P. Egger. Action of Fully-Grouted Bolts in Jointed Rock and Factors of Influence. *Rock Mechanics and Rock Engineering*, 23, 1990, pp. 201-229.

Rock Slope Remediation, Chester Vermont, Lessons Learned

Thomas D. Eliassen, P.G.
Vermont Agency of Transportation
1 National Life Drive
Montpelier, Vermont 05633
(802)-828-6916
tom.eliassen@state.vt.us

Peter C. Ingraham, P.E.
Golder Associates, Inc.
670 North Commercial Street
Manchester, New Hampshire 03101
(603) 668-0880
pingraham@golder.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to thank the following individuals/entities for their contributions in the work described:

Ken Robie, P.G. – Project Manager – VTrans
Philip Peloquin – Project Engineer – VTrans
Jay Smerekanicz, P.G. – Golder Associates Inc
Danny Breer – Resident Engineer – VTrans
Jason Waysville – Boswell Engineering

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

In the early 1960's the Vermont Agency of Transportation (VTrans) realigned VT-103 near Chester, Vermont, to replace an aging bridge and increase sight distance along the roadway. To construct the modified alignment, a steep rock cut was constructed, exposing persistent steeply dipping foliation joints slightly oblique to the roadway. As the rock cut aged, it required frequent maintenance to clear fallen rocks, some of which were over 30-tons in size. VTrans mitigated the dangerous rock cut in 2010. During design of the slope repairs and construction, several issues arose that are worth discussing. State DOTs often augment their engineering work force by contracting outside firms for developing plans and specifications. In cases where limited in-house staff are available, these consulting firms can provide additional technical resources to support the DOT and ensure state-of-the-art investigative, design and inspection services are applied. Paramount to the successful completion of rock slope and other geotechnical projects is continuity of the project team from design through construction. The dynamic nature of rock mitigation projects can lead to compromised project goals and schedule if the team or project governance is disrupted. This paper presents some 'lessons learned' during design and construction of the Chester slope repairs that may help other DOTs, municipalities and facility owners in their rock slope mitigation planning efforts. Lessons learned are presented that relate to potential contracting roadblocks, site access issues, construction sequencing and project estimating.

INTRODUCTION

Vermont is a mountainous state dominated by a series of northeast-southwest trending mountain ranges that reach heights of between 3,000 to 4,000 feet above sea level. The construction of Vermont's highways (especially Interstate highways) required the blasting of bedrock from the sides of the roadways at numerous locations in order to maintain grade. As these rock cuts aged and weathered, maintenance became necessary to clear rock from the base of the slopes and from slope faces to reduce the risk of rocks falling onto the roadways. Many of these cuts exceed 100 feet in height. In 1962, one such roadway (Vermont State Highway 103) in Chester, Vermont was re-aligned to eliminate a sharp curve and accommodate construction of a new bridge over

Figure 1 Chester project location map.

the Williams River. This re-alignment included re-cutting and trim blasting an existing rock exposure on the eastern side of the roadway. The project, located on VT-103 in southeastern Vermont, is approximately five miles north of the town of Chester (Figure 1). The highway in this area passes through a narrow valley with steep valley walls rising 500 feet on the west side and 300 feet on the east side. The valley is the southern extension of what is referred to as Perkinsville Gulf. In addition to the highway, the Green Mountain Railroad and the Williams River occupy the valley.

The Chester rock cut is approximately 120 feet in height and 1,200 feet in length. The rock in the cut is exposed as large tabular slabs that dip toward the roadway at about 60 to 65 degrees. The slope is heavily vegetated with both hardwood and softwood trees. A thin veneer of glacial till covers the upper portion of the slope. The rock cut is very wet and had a long history of winter ice and rockfalls. Figure 2 shows the subject rock cut prior to construction of repairs.

Figure 2 Photograph of VT-103 and rock cut (view is looking north).

GEOLOGY

The exposed bedrock on the project consists of silvery gray, lustrous, muscovite-garnet schist identified as the Gassetts Schist. The Gassetts schist is a member of the Precambrian age Cavendish Formation which occupies the western flank of the Chester Dome. The Chester Dome formed as the result of the Taconian and Avalonian oregenic events some 400 to 500 million years ago. This long history of oregenic events (compounded by post glacial isostatic rebound) have produced the discontinuities we see in the rock today. Figure 3 presents a generalized geologic bedrock map of Vermont and insert showing the Chester Dome. Figure 4 shows a cross section of the Chester Dome at the project location.

As a historic note (*1*), a mining operation took place briefly from about 1930 to 1945 beneath what is now the re-aligned roadway. This operation, known as the McGurry Garnet Mine processed mostly muscovite with smaller amounts of garnet. Some of the garnets in the Gassetts Schist measure up to 1-inch in diameter. This operation was most likely an open cut rather than an underground mine shaft.

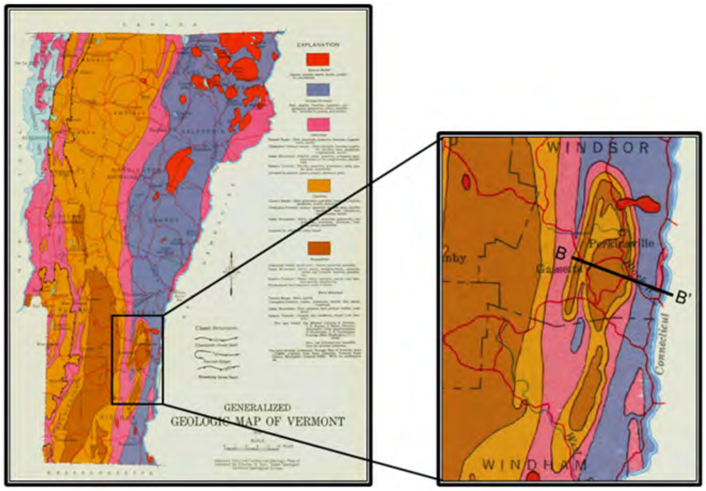


Figure 3 Generalized geologic map of Vermont and insert showing the Chester Dome.

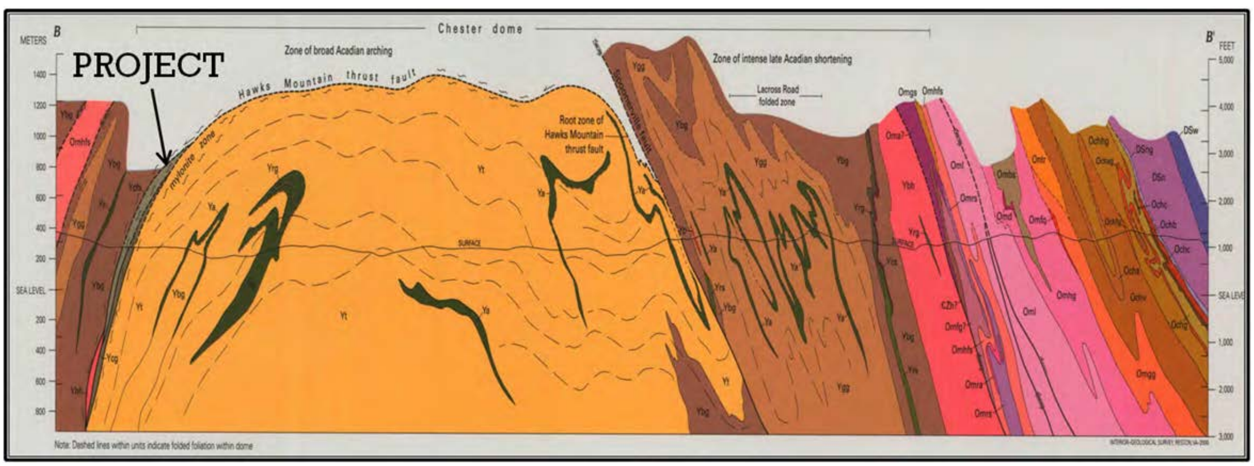


Figure 4 Cross section of the Chester Dome showing project location.

ROCKFALL HISTORY

There has been a long history of rockfalls at this cut location. In 1962, VT-103 was re-aligned to take out a sharp curve and to straighten the roadway to accommodate a new bridge crossing the Williams River (Figure 5). The re-alignment project cut the toe at a roughly 4 on 1 angle (standard for Vermont at the time) and exposed dipping foliation joints on the finished cut slopes. It is apparent that pre-split blasting techniques were not used, given a lack of half-barrel casts and the presence of intermittent production hole and shatter traces on the cut face.

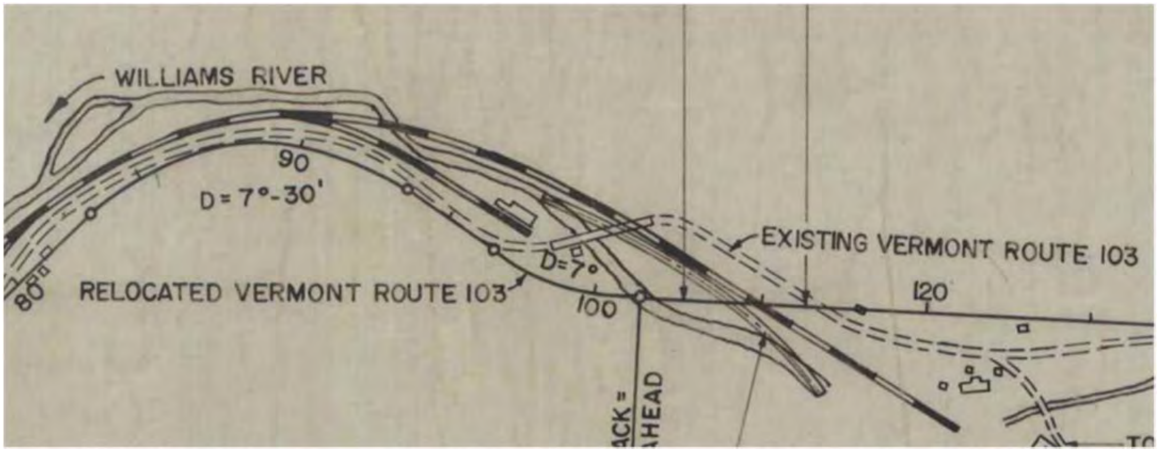


Figure 5 Schematic from 1962 construction plans showing realignment of VT-103.

As mentioned earlier, the rock slope is very wet and repeated freeze-thaw cycles have resulted in numerous rockfalls, mostly in early spring as the ice and snow cover melts. Individual rock blocks, some the size of pickup trucks have historically fallen in the past and have reached the roadway. VTrans district personnel have kept up with rock removal and maintenance each year either by breaking-up and removing the fallen rock using district forces or by contracting local contractors to perform this work. For the most part in Vermont, maintenance districts do not budget for rockfall clean-up or mitigation. Any substantial rock mitigation activities are usually absorbed in the VTrans general budget or monies are appropriated by issuing an emergency proclamation. Over time, these on-the-fly mitigation activities have added up to substantial amounts, so a longer term mitigation solution was sought for the slope.

ROCKFALL HAZARD RATING

In order to better manage Vermont's aging rock cuts, VTrans completed a Rockfall Hazard Rating study (RHRS) in 2007 (*2*) modeled after the FHWA/Oregon DOT system (*3*). The study consisted of identifying all rock cuts/exposures adjacent to federal and state highways throughout the state. Once identified, (over 3,600 rock cuts over 10 feet in height were identified), each cut was given a preliminary ranking ranging from low hazard (designated "C") to high hazard ("A"). Of the 3,600 cuts identified and ranked, 150 cuts were designated as "A" cuts requiring detailed investigations in order to develop a rating value that would reflect the relative degree of hazard the slope presented. The Chester rock cut was identified as an "A" cut and assigned a rating number of 585 - one of the "top ten" hazardous cuts throughout the state.

VT-103 in this area consists of two lanes with a pavement width of 43 feet. Percent decision site distance was about 50 feet and AADT was 6,700 vehicles per day. Truck traffic is quite heavy in this area. The roadway rises toward the north at a 3 % grade. The slope height was estimated at the time of the study to be 90 feet. Subsequent to clearing and grubbing during construction, it was found that the slope height was actually 120 feet. The ditch geometry was estimated as "Good" and the distance from the toe of the slope to the pavement was measured at 13 feet.

There were large tabular shaped overhanging blocks of rock on the slope, numerous unfavorable discontinuities, wet conditions, history of rockfalls and heavy vegetation. As anticipated, all of these conditions contributed to the high RHRS score.

PROJECT DEVELOPMENT

Based on the high RHRS value assigned and requests from the maintenance district to conduct some sort of mitigation to this cut, a program to fund, investigate, design and construct a rockfall mitigation project was developed. The VTrans geologist met with the Roadway Design Unit of the Agency to put together the necessary components necessary to eliminate or greatly reduce the hazard this rock exposure presented.

During the project development process it became clear that the assistance from an outside consulting firm experienced in rock slope engineering would be beneficial. VTrans has only one geologist and in order to perform all of the other duties the geologist was responsible for, the addition of consultants to the project would allow the project to go forward in a timely manner. Another reason to bring on an outside firm was that VTrans does not have the capability to perform rope access activities.

Experienced firms that perform rockfall investigation and design services have staff that are qualified/certified in rope access techniques that allow them to inspect all areas of the slope in detail. At the time of project initiation, VTrans had at its disposal, three geotechnical firms under contract for geotechnical services. One of these firms, Golder Associates has a reputation as an experienced rock slope engineering firm. Golder was asked to provide a proposal and was subsequently retained to provide rockfall mitigation design services for this project.

SITE INVESTIGATION AND GEOTECHNICAL REPORT

The scope of services established consisted of performing field activities which included:

- Observation of slope geology;
- Assessment of overall slope conditions;
- Identification of potential rock stabilization measures and recording them on digital photographs taken on April 16, 2008 by VTrans;
- Review of site conditions and access for field surveying; and
- Review of slope conditions and rope access observation of slope joints.

Detailed observations of rock slope conditions were made via rope rappel at various locations together with traditional geologic surveying along the base of the rock exposure. Measurements of joint conditions, asperities and joint orientations were completed along with measurement of rock blocks to estimate block stabilization requirements. Other field related activities included surveying in order to develop detailed topographic representations of the slope to be used in the design and contracting documents.

During the investigative phase of this project, access to the top of the slope was available from an adjacent landowner's property. At that time, the landowner was very helpful and offered access for the project (an abandoned old logging road ran just up slope of the cut face and would make great access to the work site).

Based on their investigation, Golder identified potential rock dowel locations, loose rocks to be scaled, dental shotcrete and buttress locations, and other potential stabilization measures. The locations of these mitigative measures were recorded on color photographs of the slope provided by VTrans.

Site observations from the ground surface and during rope rappel indicated that many of the rock blocks on the slope had dilated and weathered joint planes behind them, and likely could not be stabilized in place. Therefore rock scaling/removal was recommended to remove dilated rock blocks. Areas of limited trim blasting were identified to aid in the removal of larger rock blocks. Where rock blocks were bounded by less weathered joints, securing in place with passive grouted rock dowels that inhibit dilation needed for sliding along joints with asperities was recommended.

One area at the north end of the rock slope was identified that would likely require installation of a shotcrete buttress as the rock block lying above this area was thought to be too large for safe removal. After scaling and rock removal, additional areas were expected to be identified where shotcrete buttressing could restrict dilation and provide additional stabilization. The shotcrete buttresses would require rock dowels placed into the rock to provide reinforcement for the buttress.

Based on site observations and VTrans records, groundwater seeps were identified on the slope that would require the reduction in the groundwater level and hydrostatic pressure within the slope to reduce the driving forces on the rock blocks. Drains intercepting joints behind the rock face were recommended to lower water levels behind the rock slope face, removing groundwater seeping from the joints and reducing weathering, rockfalls and ice formation.

Due to the weathered and dilated nature of the rock blocks, Golder did not consider tensioned rock bolts in the design to avoid shifting or breaking thin rock slabs. However, it was recommended that some tensioned rock bolts be included in the contract as field conditions might call for locating some tensioned bolts during field design if conditions warranted them.

A geotechnical report detailing the observations made and recommendations offered in order to stabilize the slope. The report included the following:

- A discussion of the current site conditions,
- Discussions of the field activities conducted,
- Recommended mitigative techniques for slope stabilization,
- Stability analysis for the design of rock reinforcement,

- The development of plan sheets and typical drawings,
- The development of special provisions, and
- Recommendations for qualified engineering geological oversight for construction.

DESIGN

The locations of potential doweling, trim blasting, scaling, drainage and shotcrete buttressing were depicted on a set of photographs previously taken by VTrans. These photos were copied to plan design sheets and typical drawings for rock dowel and rock bolt installations were developed.

To develop a rock dowel design, Golder selected a representative rock block in the slope for further analysis. Based on the topographic conditions, geologic factors, joint spacing and estimated dimensions of the rock block, a limit equilibrium approach was used to evaluate the driving forces acting on the block (i.e., gravity, groundwater and earthquakes), and the resisting forces acting on the block (i.e., friction and cohesion), to determine how much additional resisting force would be required to develop an acceptable factor of safety. Once these forces were determined, rock dowel diameter, pattern, length, orientation and number to provide the additional resistance for the typical rock block were identified. Using the typical rock block as a guide, Golder then estimated the number of rock dowels needed for other rock blocks on the slope that potentially required stabilization. The rock dowel calculations and designs were intended to be a guide during rock remediation construction, as often scaling/rock removal exposes the need for rock dowel stabilization.

PLAN AND SPECIAL PROVISIONS DEVELOPMENT

As part of the geotechnical services, Golder was tasked with developing project plan sheets and special provisions. Based on the mitigative techniques recommended, Golder developed a set of special provisions describing the following:

- Hand Scaling (rope access),
- Removal of waste scaled rock,
- Rock doweling and rock bolting,
- Shotcrete,
- Trim blasting, and
- Rock drains.

Hand scaling requirements included demonstration that the contractor had at least 5 years of experience performing this type of work and a demonstrated history of projects with other DOTs and the pay item was by the crew hour (a crew was established as three scalers and one foreman). In order to better account for the volume of rock removed from the slope, payment was tied to a weight/volume relationship whereby a representative unit weight of the rock was used along with actual weights of loaded trucks to establish an equivalent cubic yard payment item. The contractor was required to provide portable scales for the weighing of empty and full trucks as the removal progressed.

Rock doweling was paid by the linear foot that consisted of galvanized continuous thread steel bars (Grade 90 (minimum) continuous thread bar) inserted in boreholes drilled into rock. A grout tube was required to be attached to the bar to tremie grout in place. These dowels were not tensioned (only wrench tightened using a galvanized steel plate and nut). The special provisions also contained requirements for the testing of select dowels for verification of pullout resistance. A typical drawing and special provision for tensioned rock bolts was developed with the anticipation that if needed, these guidance/contractual requirements were already in-place. As it turned out, no tensioned bolts were installed during this project.

Special provisions for shotcrete were developed based on requirements for soil nail wall shotcreting to ensure the nozzle man would be proficient at installing shotcrete around asperities and steel reinforcing bars. The specification required the shooting and testing of test panels containing steel reinforcement as well as minimum fiber reinforcement and strength parameters.

CONSTRUCTION

The Chester NH 025-1(41) project was advertized in January 2010. The construction contract was awarded to a local contractor (Morrill Construction Inc.) that had performed rock slope remediation type projects for VTrans in the past. One such project conducted in 1996 involved the installation of a wire mesh drape system on a 300+-foot high rock cut on Interstate I-91 and a 1,000 foot long rockfall catchment fence on U.S. 5 in Fairlee, Vermont. The prime contractor sub-contracted the specialty rock work for the Chester project to Midwest Rockfall of Henderson, Colorado.

Scaling of loose rock was conducted utilizing rope access techniques. Scaling was accomplished with aluminum shaft steel tipped mine scaling bars and various sized air bags. The scaling crew consisted of 3 scalers and 1 foreman. Crew members rappelled down the slope at intervals that allowed safe access to the areas needing scaling. In addition to scaling from ropes, loose rock in the lower portions of the slope was removed using machine scaling methods. Care was needed not to undermine the higher portions of the slope. The machine scaling had been inadvertently left out of the project documents, but was quickly identified as necessary and therefore special provisions were quickly drafted and this item added to the project. Rock was allowed to accumulate in the ditch and ultimately picked-up and loaded into dump trucks for transportation for disposal. Portable truck scales were set up at a convenient location off of the highway to measure scaled rock for payment purposes.

Following scaling, rock doweling was conducted utilizing a telescopic boom lift fitted with an air powered drill (Figure 6)

Figure 6 Photograph showing drilling operations for rock dowel installations.

Drill holes were advanced to depths of 15 to 20 feet at angles from horizontal of no greater than 10°. Once drilled and the hole cleaned of cuttings, galvanized 1.25-inch diameter continuously threaded steel bar (grade 150) were installed with PVC centralizers and grout tubing attached. After a number of dowels were inserted, grouting took place by pumping grout through the grout tubes. The annulus near the face of the dowel (bird's beak) was hand packed with grout and galvanized plates and nuts were affixed. In some areas spot doweling was required while in other areas, pattern doweling was performed. At a number of locations where water was observed seeping from the slope face, drain holes were drilled at a slight upwards angle to promote drainage.

Two areas were identified that would require trim blasting. One of these areas was a slab measuring approximately 30' X 20' X 5'. Shallow holes were drilled into the rock slab perpendicular to the slab face and trim blasting was performed in an attempt to dislodge the slab along a suspected open joint surface. This blast only removed about one third of the block, so a few short dowels were installed to secure the remaining portion of the block. After scaling, a fairly large inverted wedge of rock was identified that contained a number of through joints and so it was decided to remove this material by trim blasting. The same procedure (perpendicular drilling) was used here although because of its size, a number of trim blasts were conducted. Trim blasting was paid for under a separate item and therefore material removed by this method had to be segregated from the scaled rock for payment. The final volume of trim blasted rock increased from design of 150 yd^3 to 940 yd^3.

Estimating quantities for rock remediation projects can be very difficult. Bridges are easy in comparison to estimate. Concrete, steel and other manmade bridge components have simple clear dimensions and design is straight forward. Rock projects on the other hand are very dynamic where site conditions change, sometimes dramatically - and the range of contractor experience and capability can vary; particularly in a lowest bidder selection situation. All participants (client, contractor and funding authorities) need to understand and plan for changes during the completion of the project.

In this project, actual scaling hours required increased significantly above the original estimate. Whether or not a rock needs to be removed isn't known until a pry bar or air bag is put to it. Part of this increase was also associated with a large severely weathered open joint exposed only after the project was initiated. Payment for cleaning out this joint was made under the scaling bid item since this dangerous work was performed using rope access techniques. Only after the project was underway was it clear that additional trim blasting would be required. Rock doweling could have been performed in this area; however it would have required many dowels and removal of the block eliminated the problem and potential long-term maintenance costs. The increase in the amount of shotcrete applied on this project was directly attributable to sealing the large weathered joint at the northern portion of the project. Heavy vegetation had masked this open joint prior to the start of the project. The remaining pay items on the project were either estimated correctly or any cost increase of these items was insignificant. In aggregate, the project cost increased from $590,000 to $1,000,000 due to the increased level of effort required to address conditions exposed after clearing, grubbing and scaling.

The original design anticipated one small area would require the application of a shotcrete buttress. After scaling was performed, it became evident that shotcreting at that location would not be necessary. However, as discussed below, another area was identified that required the application of shotcrete to line a large open joint surface.

As the project was about to commence construction, several issues arose to complicate the construction process:

- The Vermont AG's office imposed new contract language for on-call professional service agreements that could not be signed by the three incumbent geotechnical firms providing support to VTrans;
- The site abutter learned that allowing access to the contractor would void his homeowners insurance;
- The division of work between the prime contractor and specialty rock contractor was unclear and involved high angle clearing and grubbing by personnel untrained in high angle work; and
- The proposed construction sequence by the contractor had clearing and grubbing, scaling and rock bolting happening concurrently, not recognizing the need to clear, scale and then reinforce the rock masses.

The project had been scheduled for construction with the intent of maintaining continuity of design support by Golder through construction. The new contract language imposed by the AG's office included broad form indemnification that made professional liability insurance commercially unavailable to consultants who signed the contract, so all three geotechnical firms with on-call agreements could not extend their contracts with the new language. (The objectionable language has been banned by state legislation in 17 states, and discussions between insurers, the American Council of Engineering Companies and the AG's office could not shift the Vermont's Attorney General's mind on the issue.) This would leave VTrans without engineering support for the rock slope work during construction, knowing full well that encountered conditions would require field fitting the design after scaling and locating and establishing the final number and length rock dowels/bolts to be installed.

Loss of the uphill site access complicated work on the slope tremendously. The right-of-way extended only 10 feet or so beyond the crest of the old rock cut, and materials and equipment for slope work had to be ferried to the top by hand or using the contractor's lift equipment, some of which was too small to reach the slope crest. Site access issues can result in construction claims if there is ambiguity on the part of any of the stakeholders.

Progress of the work was tortuous to start, with the earthmoving (prime) contractor providing clearing and grubbing services (limited exclusively to tree cutting and not root and soil removal) at the crest of the slope. The sequencing of the work was at times confusing, with the earthmoving contractor (prime) and specialty rock contractor trying to accomplish some tasks out of sequence. Clearing and grubbing work though explained in the standard specifications as the removal of trees (clearing) and removal of roots and objectionable material, was interpreted by the contractors as simply tree cutting. Work progressed slowly from the south to the north and eventually, the contractor understood that the sequence of the work would have to be clearing and grubbing followed by scaling and then assessment and installation of rock dowels.

Figure 7 Massive blocks and severely weathered joint surface in northern portion of project.

As the project neared the north end, several massive blocks became a concern and VTrans wanted the consultant engineer's input on their stabilization or removal (Figure 7). Several approaches to contract Golder were undertaken, but none could be implemented. After roughly 5 weeks of exhausting alternative approaches, the AG's office relented and allowed use of the old contract language to retain Golder for the construction phase of the project.

Figure 8 Photograph showing the excavation of severely weathered rock from joint surface.

Golder requested that the contractor finish grubbing the slope and scale some loose flakes prior to developing a design for the block stabilization. The blocks were reasonably intact and most of the joints were not adversely dilated. Accordingly, the blocks were doweled in place with the dowels space on discrete blocks and the number of dowels established (Figure 8) using doweled joint strength estimated using Spang and Egger's (*4*) approach. Several additional dowels were eventually added to secure discrete blocks in place.

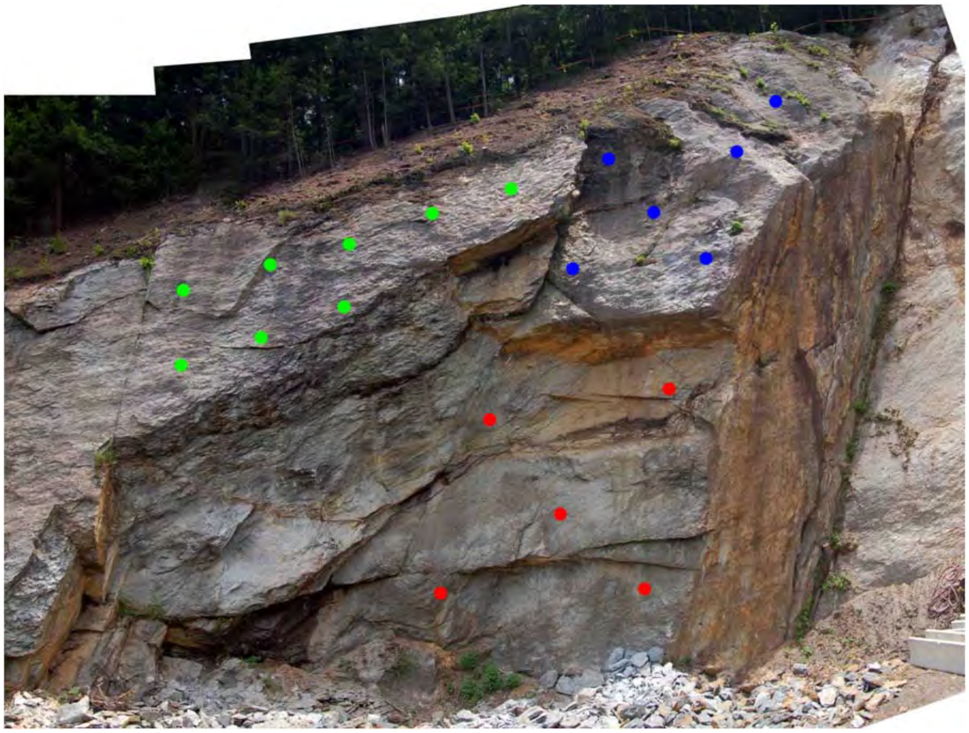


Figure 9 Dowel reinforcement plan for massive blocks in northern portion of project.

LESSONS LEARNED/EPILOGUE

The Chester project was completed in five months, with contract and project sequencing delays on the order of five weeks. As noted previously, several issues served to complicate and delay the construction process. The professional services contract issue has finally been resolved, but only after the legislature started moving toward regulating allowable contract language (as done in 17 states so far) did the AG's office capitulate and adopt reasonable terms and conditions. The following lessons learned probably apply to many types of projects, but in particular to rock stabilization work:

- All rock mitigation projects should require an on-site pre-bid meeting with all prospective bidders (and their sub-contractors). This will assure that they have an appreciation for access issues to the slope and afford them the opportunity to ask questions regarding the expectations are for the project.

- The more heavily vegetated a slope is, the greater chance it may mask potentially hazardous areas. If possible, Agencies should consider letting a clearing and grubbing contract prior to letting the construction contract. We recommend that this be done <u>prior to</u> any investigative activities.

- Understand the division of work, ask how the work will be done and by whom. Your earthmoving contractor should not be doing clearing and grubbing on a high rock slopes without appropriate gear and experience.

- It is important to sequence the work so that the most appropriate mitigation technique can be applied. Scaling of all loose rock should always precede rock reinforcement methods. Ask for a work plan as a deliverable – how does the contractor's team plan to address the work and complete all the tasks necessary in a reasonable time frame.

- It is advisable to have a pay item identified and defined in case the item is needed (even though the need for the item may not be readily apparent). The downside to this is that if the quantities are small, the bidder will most likely put forth a high per unit price (which should be viewed as an unbalanced bid). If during the project, the quantities increase significantly, ultimate costs could be very high.

- For scaling and trim blasting projects, traditional before/after sectioning methods for quantity measurements are not appropriate. For disposal, we recommend measurement either by the hauled weight of the material or a weight/volume conversion. We should note that in the future, the use of terrestrial lidar may prove useful in estimating the amount of material scaled from a slope.

- Flexibility is key on rock remediation projects. The geology will dictate the approach needed for stabilization and the contractor and DOT need to understand that clearing, grubbing, scaling and other site prep will allow rational field-fit of the contract design to the post-scaled conditions uncovered. Rock mitigation projects can change dramatically as removed material reveals unforeseen conditions. Inspectors need to be flexible in considering alternative methods that address these changes.

- If outside design consultants are contracted to perform investigative and design services, that same consultant should be retained during construction to provide construction monitoring and re-design when conditions change. Continuity during the design/construction phases of a project is very important.

- Development of design drawings for rock reinforcement present a conceptual design for anticipated post-scaled conditions that must be field-fit to the final conditions. The design consultant should be retained to provide construction engineering and monitoring and re-design when conditions change.

- It is important to have qualified personnel on-site to help direct the work. Many state DOTs assign resident engineers with no direct rock slope experience. In those cases, constant communication between geologists and geotechnical engineers is paramount.

REFERENCES:

1. Perkins, G.H., *Mineral Resources of Vermont*, in Report of the State Geologist on the Mineral Industries and Geology of Vermont, 1929-1930, p. 258.
2. Eliassen, T.D., Springston, G.E., *Rockfall Hazard Rating of Rock Cuts on U.S. and State Highways in Vermont*, Vermont Agency of Transportation Research Project; Report No. 2007-16, 2007.
3. Pierson, L.A., Davis, S.A., and Van Vickle, R., *Rockfall Hazard Rating System Participant's Manual*, Federal Highway Administration, Department of Transportation, November 1993.
4. Spang and Egger (1990), Action of Fully-Grouted Bolts in Jointed Rock and Factors of Influence, Rock Mechanics and Rock Engineering 23, pp 201 – 229, Springer Verlag, 1990.

Using Tensioned Spiral Rope Net Systems to Mitigate Rockfall Hazards

Dane Wagner, GIT
Hi-Tech Rockfall Construction
2328 Hawthorne Street
Forest Grove, OR 97116
(503)-357-6508
dane@hi-techrockfallconstruction.com

William Kane, PHD, PG, PE
Kane GeoTech Inc.
7400 Shoreline Drive
Stockton, CA 95219
(209) 472-1822
william.kane@kanegeotech.com

Megan Hitchcock, EIT
Kane GeoTech Inc.
7400 Shoreline Drive
Stockton, CA 95219
(209) 472-1822
megan.hitchcock@kanegeotech.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original authors.

USING TENSIONED SPIRAL ROPE NET SYSTEMS TO MITIGATE ROCKFALL HAZARDS

In the past protection from rockfall hazards has been overlooked during construction, which has lead to geologist and engineers needing to find practical solutions to protect the public from existing rockfall hazards.

Various rockfall mitigation systems are available and can be utilized to provide protection. As each rockfall site has unique criteria and circumstances, both in terms of geology and the need for protection, a system must be tailored to provide a realistic and economical solution to the problem.

Solutions for large block rockfall in the past have been largely limited to rock bolting and scaling, depending on the situation both options can become expensive and-or impracticable. As new methods to retain and control rockfall have become available a new approach to mitigating large block rockfall with tensioned spiral rope nets has been proven a successful alternative.

The tensioned spiral rope net system utilizes high-tensile steel wire in combination with tensioned rock bolts and boundary cables of steel wire rope to actively retain blocks on the slope. Dimensioning of the system is completed using a software program based on the rock stresses of the bock and the structural components of the system. The loads of the block are transferred throughout the system from the spiral rope net to the boundary cable and anchors. The transferring of the loads in the system reduces the number and depth of rock bolts compared to a rock bolting pattern. By retaining blocks on the slope and reducing drilling the tensioned spiral rope net system can provide significant cost savings.

INTRODUCTION

When roads and construction activities encroach on slopes today, stability is a common concern. Unfortunately, in the past when soil and rock slopes were constructed, too often the long term stability of the slopes were overlooked. These practices have left many slopes and road cuts throughout the country with adverse stability issues. Even today when stability is a concern current design practice still cannot prevent the need for stabilization.

Many times slopes cannot be avoided and instability features are already in place. Techniques to stabilize soil and rock slopes have proven to be successful when used properly. Common methods include buttressing, mechanically stabilized embankments, soil nailing, retaining walls, shotcrete, rock anchors, and scaling. The proper approach to stabilizing slopes is dependent on the mode of instability and many techniques are used on or near the base of the slope. Large blocks have been problematic to stabilize because many times they are located up slope of the toe and access is difficult. Franklin, 1991, explains the two-dimensional approach in soil mechanics is often inappropriate when applied to rock problems; which assume that a curved path of failure will find a path through the ground and rockslides almost always bound by noncircular surfaces and bound by patterns of jointing.

When stabilizing large blocks, the traditional practice includes concrete buttressing, rock dowels and bolts, scaling, shotcrete, and cable lashing. Traditional methods have proved useful, but at times have been impracticable and/or uneconomical. This can cause no action to be taken which puts public safety at risk. A new method to retain large blocks on slopes has proven to be a solution where many traditional methods have been ineffective. The use of a wide-meshed spiral rope net made of high-tensile, 4 mm in diameter steel wire has been developed for this purpose. This allows the arrangement of nails or anchors in any order and thus perfectly adapts to irregular surfaces that can actively retain large blocks on the slope (Rüegger, 2001).

PREVIOUS WORK

Hoek and Bray, 1981, describe the methods of assessing the stability of rock slopes and techniques to improve stability. In their book *Rock Slope Stability* the basic mechanics of slope failures and the controlling factors of rock slope stability are discussed in detail. The rock slope stability varies depending on the inclination of the slope and the geological structure of the rock. The geological structures or discontinuity features of the slope are the faults, joints, bedding planes, and old failure surfaces in the rock mass. When the discontinuity features interact with the inclination of the slope in a way that the rock mass dips towards the slope face at angles of 30 to 70 degrees, the stability of the slope is significantly decreased. As different slope failures are associated with different geological structures Hoek and Bray outline some of the most common structural patterns. These patterns are displayed in Figure 1. Hoek and Bray, detail the aspects of data collection and graphical representation of data to determine failures and their mode. This is done by stereographic projection and kinematic analysis.

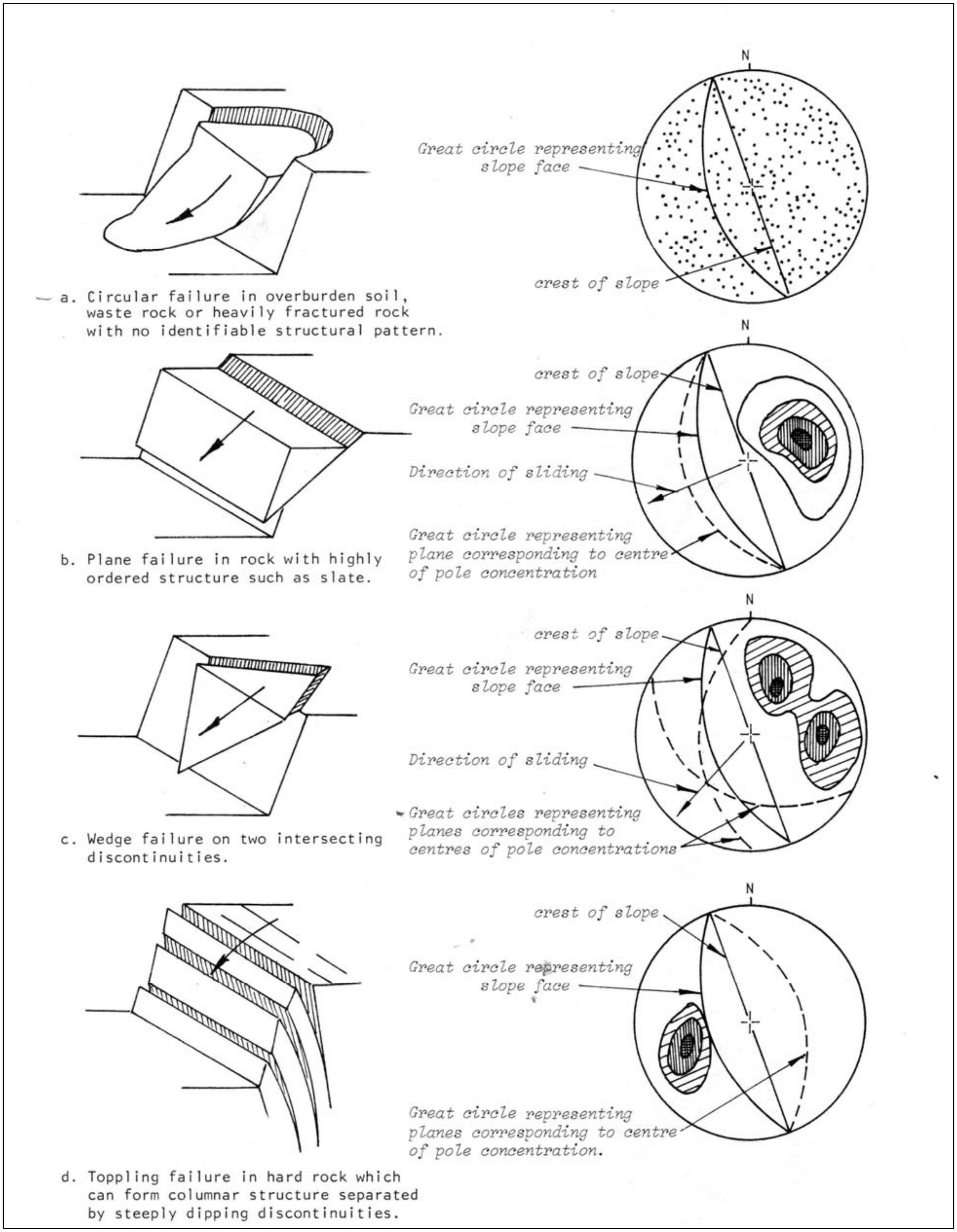


Figure 1 – The main types of slope failures and stereoplots of conditions likely to occur with the associated failures, Hoek and Bray, 1981.

The stabilization of large blocks has relied on the use of rockbolts and wire rope to cable lash the block in place. Brawner, 1994, describes measures to mitigate rockfall hazards. The alternatives include scaling, concrete buttressing, high-capacity soil and rock anchors, or constructing retaining structures in combination with high-capacity soil and rock anchors. When larger blocks are prone to failure, especially when located high on the slope, the alternatives are too expensive to correct the problem.

Brief details of anchored cable nets and cable lashing are described. Brawner describes anchored cable nets as a short term stabilization method of blocks up to a maximum size of 5 to 8 feet. The theory of cable lashing is described as “wrapping unstable rocks with individual cable strands anchored to the slope” (Brawner, 1994). Brawner’s design process of cable lashing includes estimating the weight of the block to be retained and calculating the resistance of the grouted eye bolts into the slope which can carry the weight of the rock plus 20 percent for earthquake areas.

METHODS OF RETAINING LARGE BLOCKS

The controlling factors of rock slope stability are unique to each site and make a universal solution to rock slope stability a difficult task. The factors that control rock slope stability include geologic conditions, shape, height, and steepness of the slope. The complexity of a rock slope requires that an individual evaluation for each potential failure location and mode be done. Evaluations of site can be completed by structural analysis of the slope using the previously mentioned kinematic analysis and limited equilibrium methods (Hoek and Bray, 1981) to locate the potential failure locations. When these locations are determined to be hazardous and in need of stabilization the proper mitigation selection is critical.

Just as there are many controlling factors of slope stability there are also many mitigation options. Mitigation methods are largely dependent on the site and each site has individual characteristics which may include property boundaries, access, constructability, maintenance, design life, and cost. All factors should be considered as each factor can and does influence the others (Wagner, 2010). There are many approaches, including active and passive stabilization. Again for the purposes of this paper only the active retention of large and isolated blocks is focused on.

The difficulty in retaining large blocks on slopes has largely been controlled by the limitations of methods to mitigate the hazard. The methods to determine the controlling discontinuities and mode of failure have been proven effective in the field, but preventative methods have been limited. Use of rock dowels and bolts have long been the standard for retaining large blocks, but drilling activity can disturb the slope, and depending on block size, can be required to reach great depths. Shotcrete and concrete buttressing both can become expensive when used in difficult access areas or on high slopes. The use of anchored cable nets, meshes, and cable lashing have been used but design of these methods is difficult to provide definite design models due to the complexity of the material characteristics. In today’s climate of lawsuits and liability the need for a definite and proven design model is critical.

Wire mesh, such as standard chain link and double twisted wire mesh are not suitable in an active anchored system. This is because the material is of low strength leading to plastic deformation and failure under the loading conditions (Wagner, 2010). Standard chain link and twisted wire meshes are limited in size of material that can be retained, which is also a result of the strength of the material.

Pinned cable nets can be used with certain limitations, wire rope strengths and cable connections need to meet loading requirements. Anchored cable nets have a limited size of retention, 5 to 9 feet, (Franklin, 1991 and Brawner, 1994). When a block of material surpasses the maximum retention ability of cable nets the material will deform, or bulge, a sign that the material is overloaded. When cable nets are overloaded the failure and release of the rock material can occur. The ability of cable nets to deform can also cause the release of the actively applied force that is to be applied to the slope. As the cable nets shift or stretch the release of the active force can allow for failures of the slopes. This can result in overloading the cable nets with rock material, causing failure and release of rock material to be contained. When using a pinned cable net or wire rope system it is critical to calculate the weight of the block and the forces that will be applied to the cables. The strength of the cables will be the determining factor in retaining the rock. When choosing a net type, the net deformation and resistance should be calculated on the performance of the material in laboratory and field testing (Macro, 2006). If cables with clips are used there is the potential that the clips can pop off under loading, causing wire rope openings to increases. The use of wire rope cable nets also increases surface area exposure to corrosion, causing a decreased strength and life span of the product.

The before described traditional methods have created the need for a solution to increase the reliability of stabilization of large blocks on slopes. The tensioned spiral wire rope net system has improved the strengths and filled the gaps of the limitations of past measures. This product combines the use of rock anchors, high strength steel spiral rope nets, spike plates and boundary cables to actively retain the blocks on the slope (Roduner, 2010). High strength materials are made from high tensile strength steel with roughly four times the tensile strength of a standard wire. The high strength steel will not deform plastically, decreasing the likelihood of the material to release retained loads. The components of the systems transfer the loading requirements to stabilize the block creating a system which provides increased contact with the unstable rock mass using a high strength spiral rope net without the maximum size limitations of traditional methods. The interaction of the components to transfer the load through the high strength materials reduces drilling depths and the need to drill through unstable rock mass in comparison to traditional rock bolting methods overall decreasing costs and increasing safety.

CASE STUDIES

Colorado Springs, Colorado

Outside of Colorado Springs, Colorado a tourist attraction was experiencing continuing rockfall that was creating a hazard to the visitors and the park's infrastructure. The main concern of the project was a large fractured outcrop perched on the slope roughly 100 feet above the main entrance and gift shop. The site is located in the Colorado Front Range. The geology of the exposed bedrock at the site consists of steep to nearly vertical slopes of hard and jointed Pikes Peak Granite.

The large outcrop perched on the slope was creating a safety hazard to the visitors and workers at the park. The history of rockfall at that location was evident as the gift shop had to be repaired in the past due to numerous rockfall events including a boulder that crashed through the roof of the gift shop. In another event a boulder in excess of 740 tons fell from the slope and still remains in the parking area of the gift shop. The hazardous outcrop on the slope is in excess of 13,000 tons; dimensions of the outcrop are 50 feet wide by 65 feet high and an average thickness of 40 feet, Figure 2. The outcrop jointed into three main masses and is resting directly above the gift shop at a dip of 32 degrees.

Figure 2 – Tensioned spiral wire rope nets installed on blocks above the gift shop outside Colorado Springs, CO.

The traditional methods of stabilizing the outcrop were determined to be unsatisfactory. Scaling of the boulder by blasting or the use of pillows was not an option because of the near certainty of the scaled outcrop destroying the gift shop. Rock bolting the outcrop posed a concern because of drilling vibrations destabilizing the outcrop while workers were drilling. The solution to stabilizing the outcrop was to use tensioned high strength spiral wire ropes. The use of the system resolved the issues of drilling into the outcrop by anchoring the spiral rope nets to the perimeter of the hazardous area. The tensioned spiral rope net system greatly reduced the drilling efforts and hazards versus drilling rock anchors though the outcrop.

Los Angeles, California

Along the Los Angeles River a commuter railway maintenance facility was experiencing rockfall events. The rockfall was affecting the access road and parking area of the maintenance facility. The project site had an existing flexible rockfall barrier, but the barrier provided inadequate protection as the barrier was being overtopped by rockfall. After an investigation of the site, it was confirmed that the existing mitigation was inadequate in various locations. To protect the facility and access from the rockfall hazards associated with the site, an increase in rockfall protection was necessary.

After an investigation the decision was made to utilize the existing rockfall barrier in combination with supplemental measures. Smaller rockfall events with rock sizes of 3 to 12 inches were overtopping the barrier along the access road where the barrier was installed at the toe of steep exposed bedrock slopes ranging from 50 to 80 feet at approximately 70 degrees. The supplemental mitigation measure installed in this area was a light drapery which prevented overtopping of the barrier. The slopes at the site transition to higher gentler slopes, approximately 300 feet in height and 50 degrees, with isolated exposed sandstone outcrops, Figure 3. The investigation demonstrated these outcrops as a potential hazard to overwhelm the energy capacity of the existing barrier in a rockfall event. The solution was to retain the outcrops on the slope using a tensioned spiral rope net system.

Figure 3 – Existing rockfall barrier and tensioned spiral wire rope nets on sandstone outcrops in Los Angeles, California.

The system was installed on the isolated outcrops only and was a great cost savings compared to the alternatives. The other options included installing a larger capacity barrier (3,000 KJ), shotcrete, high capacity rockbolts, and scaling. Installation of a higher capacity barrier would have required the removal of the existing barrier and an increased cost compared with the tensioned spiral rope net system. Shotcrete and rock bolting would have increased the amount of drilling, and heavy equipment would have been required to create roads and benches on the slope, creating a right of way conflict. Scaling of the outcrops could have damaged the existing barrier and facilities, and left exposed outcrop on the site which over time could become a hazard again. The use of the tensioned spiral rope net system proved to be a solution which prevented multiple construction issues and reduced the overall cost of the project.

Harrodsburg, Kentucky

A rock-fill dam located in Central Kentucky provides flood protection and electricity to the surrounding areas. The largest rock-filled dam in the world at the time it was built, about 300-ft (91-m) high and 1,000-ft (304-m) long. Constructed in the 1920s when the river was dammed, creating a reservoir in a deep gorge with near-vertical exposed limestone cliffs. The limestone bedrock at the site is from the Oregon Formation and exhibits near horizontal bedding with vertical jointing and differential erosion of less resistant bedrock material. Karst features are common. Toppling failures of large to medium limestone blocks created a rockfall hazard.

The hydroelectric infrastructure associated with the dam, including the concrete dam face, intake tower, penstocks, and a high-pressure gas pipeline running across the dam crest, were determined to be at risk of damage from to rockfall. After a hazard assessment was conducted, numerous potential block failures were discovered that could threaten the existing facilities. A plan was established to protect all at-risk infrastructures. The hydroelectric generating station and infrastructure associated with the dam including the concrete dam face, intake tower, penstocks, powerhouse, natural gas pipeline, and access were considered at potential risk of damage due to rockfall. To facilitate design, maintain flexibility, and optimize rockfall protection while controlling cost, the owner divided the project into multiple phases of construction, including extensive grubbing, and multiple phases of rockfall mitigation installation.

Grubbing and the first phase of construction and installation are complete. These measures included scaling of unstable rocks and the installation of a tensioned spiral rope system. The tensioned spiral rope net system was chosen over other methods due to the nature of the site, which included a near vertical exposed limestone rock face with extensive fractured overhanging blocks. A series of five large limestone blocks approximately 45 feet high, 35 feet wide, and 15 feet deep were overhanging the concrete dam face, Figure 4. These blocks created a design challenge as they were all located on the same jointing plane.

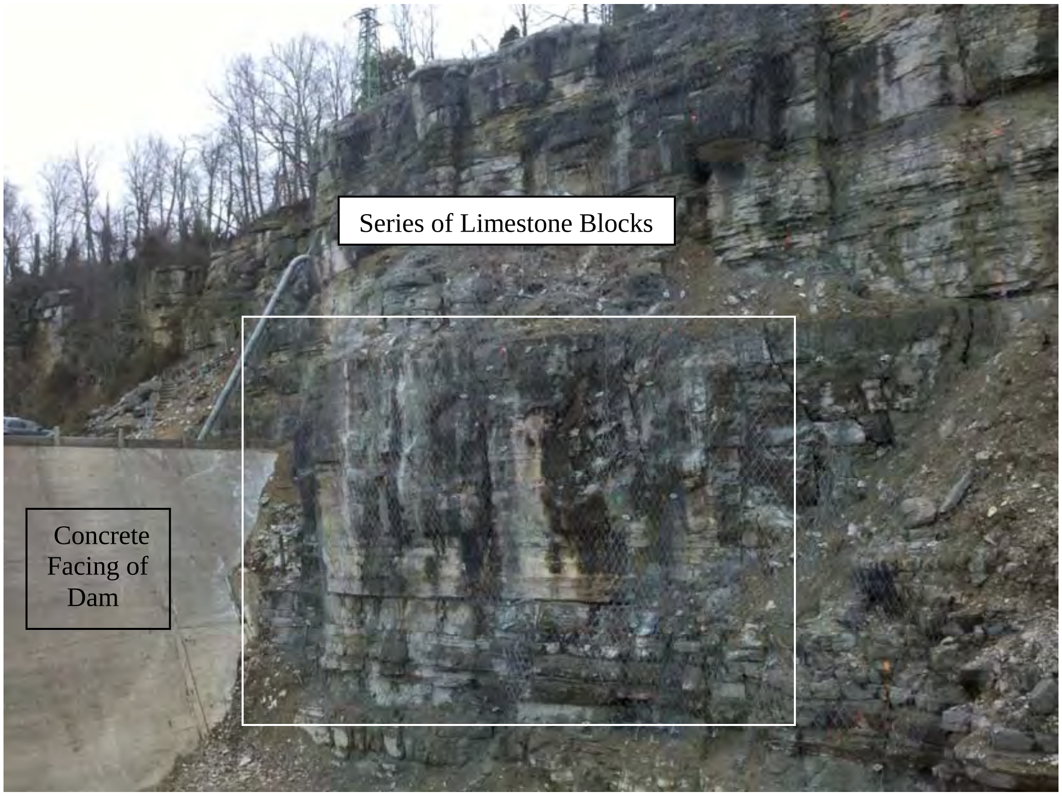


Figure 4 – Tensioned spiral wire rope nets on series of limestone blocks in Kentucky.

To meet this challenge a design to stabilize each block was performed and a careful construction sequence was to be followed to prevent any destabilization of blocks. Based on the tensioned spiral rope net design no block was directly drilled into in order to stabilize the block, only the perimeter of the blocks were drilled for the installation of the anchors. The construction sequence of the installation was based on stabilizing one block to the adjacent blocks or slope face, were the furthest protruding block was stabilized first and progressed toward the limits of the potential hazard area. The use of the tensioned spiral rope net system reduced the drilling depths and total linear footage compared with a rock bolting pattern. The use of shallower anchors allowed the contractor to use hand drills versus heavy equipment, which would have been difficult with the limited access above the slope.

CONCLUSIONS

Compared to the conventional system of retaining large blocks, the new design provides a flexible nail pattern and effective protection from corrosion, thus ensuring that a perfectly adaptable system would reliably and safely satisfy the requirements for the planned duration of use (Rüegger, 2001). The use of these systems has proven to be successful both in the United States and Europe. The development of the spiral rope net system improves the limited design control of the traditional systems. These improvements provide an increased reliability of calculations, thus limiting liability issues during the design as laboratory and field tested method has proven successful.

The tensioned spiral rope net system increases the ability of anchors and restraining material (spiral rope nets) to interact with each other for more reliable load transmission to the rock to be secured on the slope. The decrease in deformation of the high tensile strength spiral rope net utilizes the strength of the material to retain the block and transfer loads to supporting anchors and boundary ropes. The transferring of loads through the system enables the components of the system to retain and prevent movement of the block.

The wire element of the spiral rope net system provides an increased corrosion protection due to reduced exposed surface area versus wire rope, increasing the life span of the system. The spiral rope net uses a 4 mm diameter wire compared to the 0.9 mm diameter of twisted wire used in cable nets or wire ropes. The cost saving during construction is evident as compared with traditional methods as drilling locations can be tailored to an individual block rather than the mesh panel. Drilling depths required to secure the block decrease as the loads are transferred throughout the system, and the use of the system plate ensures the spiral rope net is braced against the slope as securely as possible. The high strength of the spiral net allows for lacing of the boundary ropes rather than requiring a shackle connection further reducing material cost.

The improvements to the conventional methods by the tensioned spiral rope net system have provided decisive advantages in design, strength, longevity, cost, and overall reliability of the system components. The added benefits and cost savings of the tensioned spiral rope net system provide a solution to many hazardous situations that were once impracticable and/or uneconomical to solve.

REFERENCES:

- Conference Proceedings
  1. Wagner, D. A. and W. F. Kane. Rockfall Mitigation Field Techniques and Design. In *Conference Proceedings 3: Slope Stabilization and Mitigation*, 61st Annual HGS, Highway Geology Symposium, Oklahoma City, OK. 2010.
  2. Rüegger, R. & D. Flum. Slope Stabilization with High-performance Steel Wire Meshes in Combination with Nails and Anchors. *International Symposium, Earth Reinforcement.* IS Kyushu, Fukuoka, Japan. 2001.
- Federal Government Reports
  1. Brawner, C. Rockfall Hazard Mitigation Methods, Participants Workbook. *Federal Highway Administration*, Washington, D.C., 1994.
- Books
  1. Hoek, E., and J. Bray. Rock Slope Engineering. *Institution of Mining and Metallurgy*, London, 1974.
  2. Franklin J. A. and M. B. Dusseault. Rock Engineering Applications. *McGraw-Hill Inc.*, New York, NY, 1991.
- Manuals
  1. Rouduner, A. Ruvolum Rock to Dimension the Spider Rock Protection System. *Geobrugg AG Protection Systems*, Romanshorn, Switzerland. December, 2010.
  1. Maccaferri MAC.RO. 1 User Guide. Maccaferri Inc., Williamsport, MD. 2006.

Influence of Weak Pennsylvanian System Shales in Ohio and Kentucky on Transportation Projects

Richard Williams, Ph.D., P.E.
Stantec Consulting Services Inc.
1500 Lake Shore Drive, Suite 100
Columbus, OH 43204
614-844-4004
rich.williams@stantec.com

Eric Kistner, P.E.
Stantec Consulting Services Inc.
11687 Lebanon Road
Cincinnati, OH 45241
513-842-8213
eric.kistner@stantec.com

Luis Arduz, P.E.
Stantec Consulting Services Inc.
1409 North Forbes Road
Lexington, KY 40511
859-422-3051
luis.arduz@stantec.com

Prepared for the 62nd Highway Geology Symposium, July, 2011
Lexington, KY

Acknowledgements

The authors would like to thank the Office of Geotechnical Engineering of the Ohio Department of Transportation as well as the Kentucky Transportation Cabinet Division of Structural Design – Geotechnical Branch for their permission to share project information and for their contributions and assistance in the preparation of this document. We also extend our appreciation to Mr. Mark Litkenhus and Mr. Stan Harris of the Lexington, KY and Cincinnati, OH offices, respectively, of Stantec Consulting Services Inc. for their support of our time and efforts to prepare this document. Also special thanks to Ms. Mary Murphy of the Stantec Columbus, OH office for her efforts in preparing the figures for this document.

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original authors.

ABSTRACT

The increased incidence of slope stability problems in Ohio and Kentucky within the excavated and embankment slopes constructed within or from selected shales of the Pennsylvanian System is widely recognized. Not all Pennsylvanian System bedrock units, even some shales, are considered less competent with regard to slope stability; however, the Pennsylvanian redbed formation shales and resultant colluvium, in particular, within the upper series of formations within the Pennsylvanian system are considered notoriously unstable. Understanding the differences in shale structure and the resultant impact of those structure differences on mass shale strength is a crucial part of geotechnical design and construction of transportation infrastructure within the areas of these states underlain by Pennsylvanian bedrock.

The structure of "typical" shales as compared with the structure of weak Pennsylvanian shales will be discussed. The impact of the structure differences on strength will be detailed, with particular attention given to the strain-softening behavior of the redbed shales. The suggested strength parameters for use in design of slopes excavated within selected redbed shales and/or colluviums will be discussed as well as the parameters for embankment design incorporating redbed shales and/or colluviums. Suggested methods for measurement and/or estimation of redbed shale strength will be presented.

The presentation will also include a discussion of a few roadway projects which dissected Pennsylvanian redbed formation shales and colluviums. Projects include SR 7 in Lawrence County, OH; SR 124 in Meigs County, OH; and, I-64 (Industrial Parkway) in Boyd/Carter/Greenup Counties, KY. The analyses undertaken as well as the lessons learned (good and bad) from these respective projects will be presented.

INTRODUCTION

Within the unglaciated physiography of southeastern Ohio and eastern Kentucky lie hilly to steeply-sloped terrain covered with residual soils and colluvium derived from the parent sedimentary rocks. The predominant sedimentary system within this area is the Pennsylvanian system. Within the Pennsylvanian system, several geologic members, notably the “red bed shales”, have been linked with an increased occurrence of landslides, both within the member itself as well as a more frequent occurrence within the colluvial and residual soils derived from those red beds. The presence of red bed shales is mostly concentrated within the Upper Pennsylvanian Epoch, and within the Conemaugh Group. There exist notable differences between the structure of red bed shales and the structure of other shale members, even within the Upper Pennsylvanian Epoch. The structural differences of the red versus “other” shales will be explained. The impact of those structural differences as related to overall strength of the intact member as well as the residual and colluvial soils derived from the shales will be described. Finally, the incorporation of applicable design parameters, which accurately reflect the red bed shale properties, into the design and construction of several completed projects, will be detailed.

PENNSYLVANIAN SYSTEM GEOLOGY

The Pennsylvanian System members form the near-surface bedrock within the area extending from a location where Ohio/Pennsylvania/West Virginia state boundaries meet, through a majority of southeastern Ohio, continuing into eastern Kentucky near Ashland, encompassing much of eastern Kentucky down toward Middlesboro. Fig. 1 shows the near-surface geologic members of Ohio (Ohio, DNR, www.dnr.state.oh.us/tabid/9526/), while Fig. 2 depicts the geology of Kentucky (Kentucky Geologic Survey, www.uky.edu/KGS/geoky/). The red bed shale members, more prevalent within the upper epochs of the Pennsylvanian System, tend to be exposed near-surface within the eastern portions of these delineated areas, more toward the southeast of Cambridge, Ohio, and east of Pikeville, Kentucky.

PENNSYLVANIAN SHALE STRUCTURE DIFFERENCES

Field **s**tudies of exposures of some Pennsylvanian shales reveal that the structure and color of shale members, even within the Upper Pennsylvanian Epoch, vary significantly (Condit, 1912; Fisher, et. al, 1968; Williams, 1982). The shale members exhibiting more stable (i.e., less weathering, deterioration) exposures had a generally gray color and tended toward a compact, tight, plate-like appearance with more prominent, parallel bedding orientation. The exposed bedding plane surfaces appeared dull and dry, with fractures and partings noted as having a

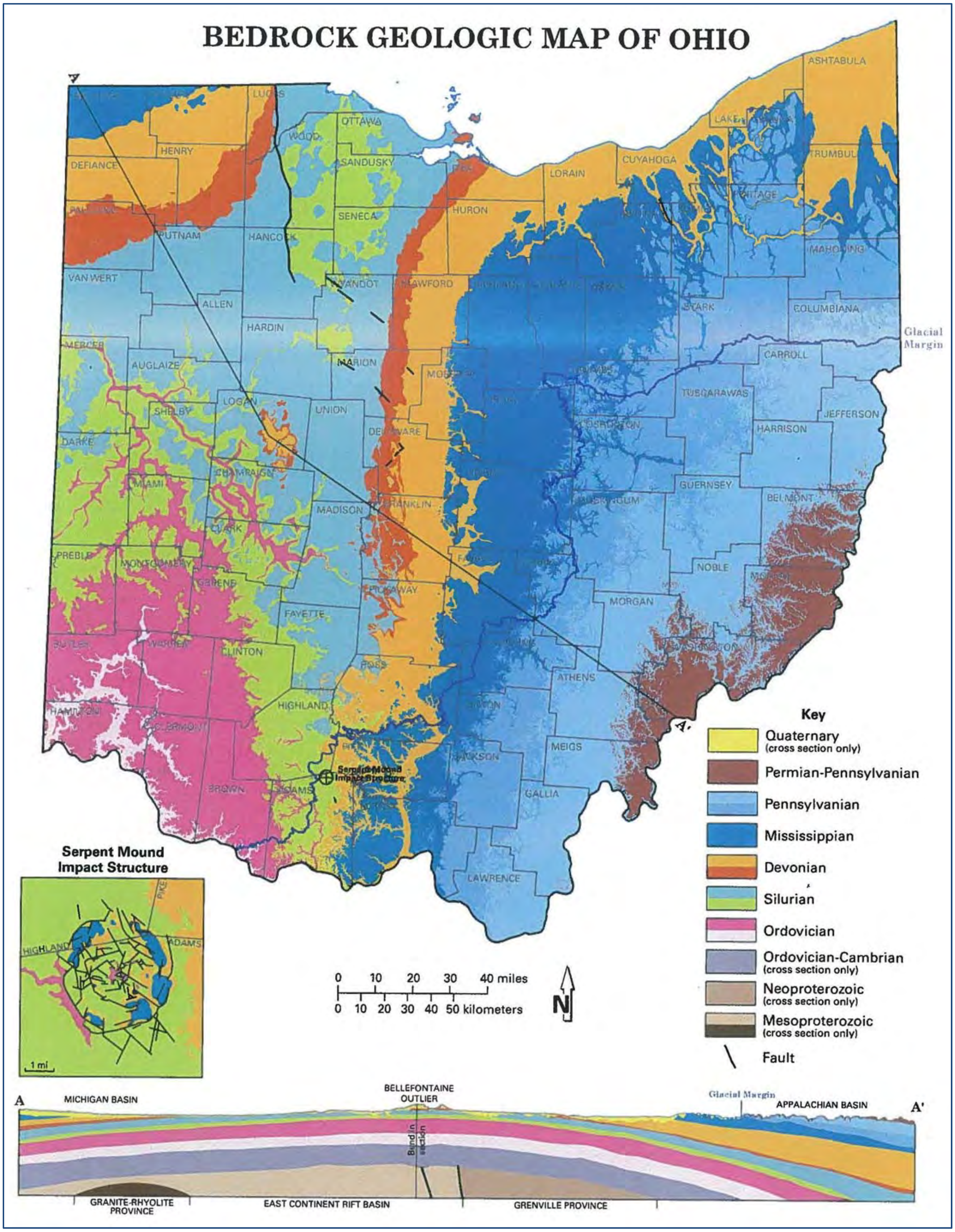

BEDROCK GEOLOGIC MAP OF OHIO
Key
Quaternary (cross section only)
Permian-Pennsylvanian
Pennsylvanian
Mississippian
Devonian
Silurian
Ordovician
Ordovician-Cambrian (cross section only)
Neoproterozoic (cross section only)
Mesoproterozoic (cross section only)
Fault
Serpent Mound Impact Structure
Glacial Margin
0 10 20 30 40 miles
0 10 20 30 40 50 kilometers
N
A
A'
MICHIGAN BASIN
BELLEFONTAINE OUTLIER
APPALACHIAN BASIN
GRANITE-RHYOLITE PROVINCE
EAST CONTINENT RIFT BASIN
GRENVILLE PROVINCE
ASHTABULA
LUCAS
OTTAWA
WOOD
HENRY
DEFIANCE
SANDUSKY
LORAIN
CUYAHOGA
TRUMBULL
SENECA
HURON
PUTNAM
HANCOCK
MAHONING
VAN WERT
CRAWFORD
STARK
COLUMBIANA
ALLEN
HARDIN
CARROLL
MERCER
MARION
AUGLAIZE
TUSCARAWAS
JEFFERSON
LOGAN
UNION
SHELBY
HARRISON
DARKE
COSHOCTON
CHAMPAIGN
GUERNSEY
MIAMI
MADISON
FRANKLIN
BELMONT
CLARK
PREBLE
NOBLE
GREENE
MORGAN
FAYETTE
BUTLER
WARREN
CLINTON
ROSS
ATHENS
HIGHLAND
HAMILTON
CLERMONT
MEIGS
JACKSON
BROWN
ADAMS
GALLIA
LAWRENCE
PIKE
1 mi

Figure 1

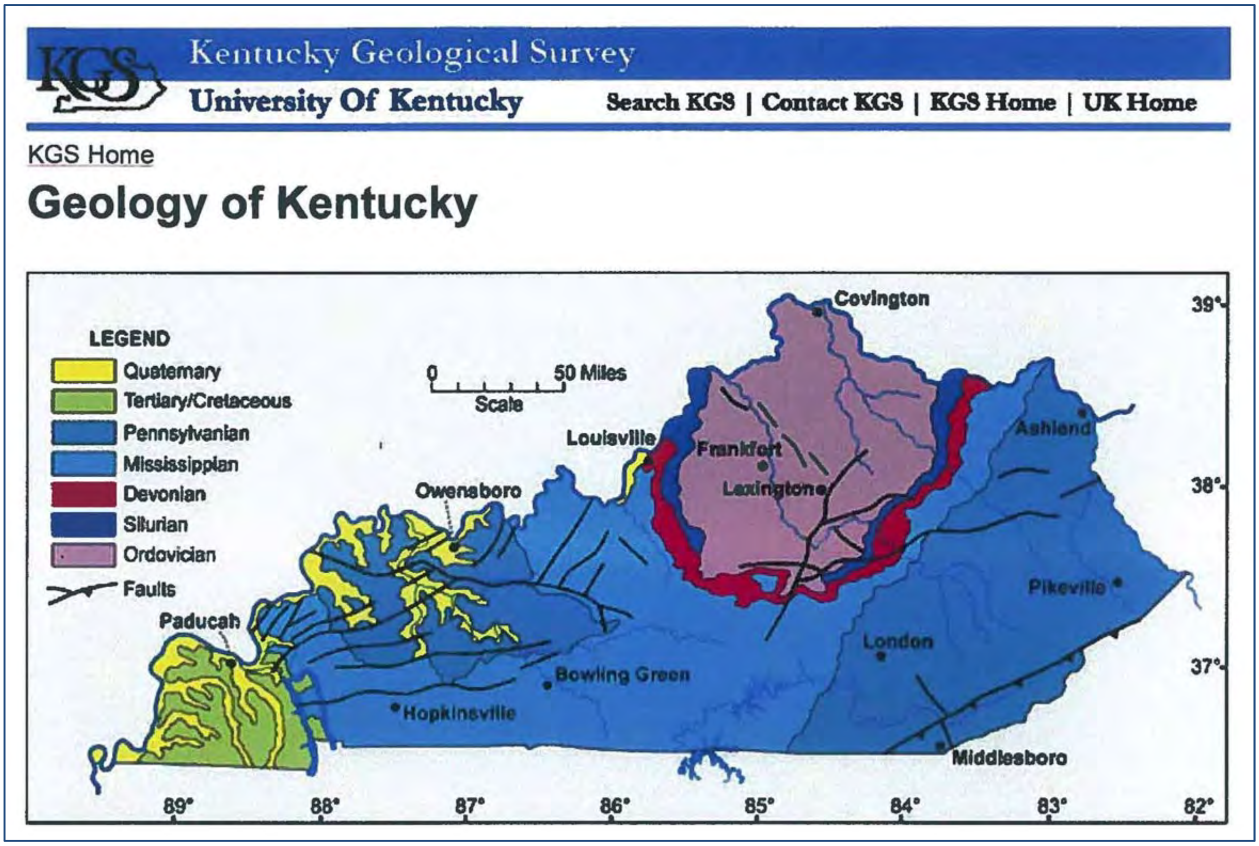


Figure 2

rough surface. Any accumulated detritus at the base of the exposure were comprised of small, plate-like particles, resembling a gravel-size. An exposure of the Morgantown Shale within the Conemaugh Group is shown in Figure 3 and is typical of a shale of more stable structure.

Figure 3 Exposure of Morgantown Shale

Conversely, the shale exposures noted to be associated with greater weathering and landslides generally had a reddish coloration, with a blocky appearance and no apparent bedding plane orientation, similar to a mudstone. The exposures of these weak shales revealed significant planes of weakness, such as smooth discontinuities and slickensides. Most of the observed discontinuities and slickensides possessed smooth, almost shiny surfaces, with a higher moisture content in the vicinity of such discontinuities (Williams, 1982). Figure 4 is an exposure of the Round Knob Shale within the Conemaugh Group, revealing the lack of bedding plane or discontinuity orientation. Figure 5 is the same exposure of Round Knob Shale with the discontinuities revealed more prominently, showing the general absence of orientation of discontinuities. Additionally, the weak red shales were observed to have accumulated detritus at the base of exposures that resembled moist soil with no "rock-like" structure.

Figure 4 Exposure of Round Knob Shale

Figure 5 Discontinuity Surfaces Exposed – Round Knob Shale

From the foregoing descriptions, an understanding of the impacts of shale structure on shale member strength, for engineering purposes, may be gleaned. The shale of more stable structure generally exhibits strength derived from shearing across or along the predominantly-observed bedding planes. The resultant relationship of strength versus displacement resembles a more classic elastic-perfectly plastic material, such as that shown in Fig. 6b, where once attained, strength tends to remain constant with continued displacement. However, the redbed shales tend to exhibit a strength-displacement relationship typical of a strain-softening material, such as that shown in Fig. 6a, wherein the initial shearing of the more intact shale between the numerous discontinuities results in a peak shear strength at small strains. With continued displacement, the sheared intact material and the smooth discontinuities interconnect, resulting in a reduction of overall shale strength (see zone 2 of Fig. 6a), with attainment of an ultimate shear strength, termed residual strength, with further displacement (see zone 3 of Fig. 6a). The residual strength is consistent with the strength exhibited along planes of failure, such as landslide surfaces, within these types of redbed shales.

Understanding of these structural differences in shale types becomes critical when assigning strength values for use in engineering analyses, such as global slope stability analyses. It should be recognized as unrealistic to design or analyze slopes with redbed shale strengths typified by a peak strength value, such as an elastic-perfectly plastic material (Fig. 6b), unless overall displacements can be significantly limited – a difficult condition to maintain. Since the formation of shales involved the significant vertical compression of deposits, the *in-situ* and unexposed shales possess large kinetic energy which becomes released laterally (due to a loss of confinement) within a cut or exposure. The release of such energy results in differential lateral strains sufficient to displace the shale (particularly near the base of a cut or redbed member) beyond the peak strength and the movements along a slip surface continue until stability is attained with respect to the driving forces of the landslip. Since such large displacements are consistent with those associated with the attainment of residual strength, it is usually prudent to incorporate such limiting strength values for the redbed shales for analyses purposes.

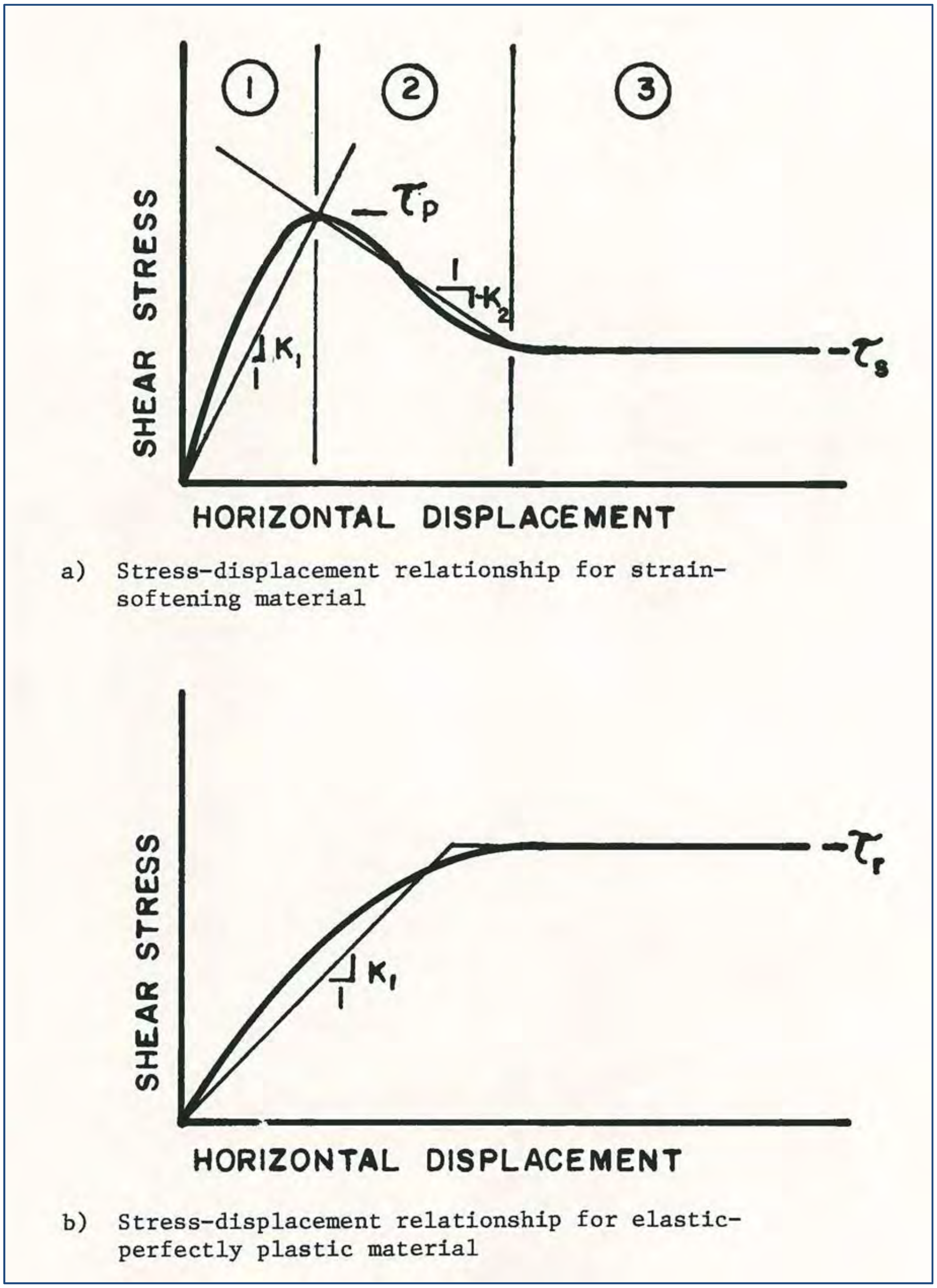


Figure 6 Typical Stress – Displacement Relationships

The residual strength is also symbolic of the overall strength of the colluvial material which accumulates within the lower reaches of shale exposures. Since the colluvium represents the weathered shale which has been transported to the base of the exposure by various natural means, the colluvium has already been exposed to significant displacement, with minimal to no innate strength derived from the original intact shale mass. Although exposed shales are known to weather due to the influences of wetting and drying, the redbed shales have been observed to weather much more rapidly than the more stable shales, such as the Morgantown shale.

REDBED SHALE STRENGTH

A better understanding of appropriate representation of redbed shale strength may be illustrated through the results of strength testing of shale samples. Figure 7 displays a summary of tests performed by the author (Williams, 1982) on samples of a Southeastern Ohio redbed shale that were observed to be, either, intact (no observed discontinuities) or contain notable discontinuities/slickensides. In those tests performed on small specimens (i.e., 2.0 in. X 2.0 in.), peak strengths were typically achieved at lateral strains of approximately 2%, while residual strengths were achieved at lateral strains of 4% to 5%. For comparison, additional tests on larger samples (i.e., 11.0 in. X 11.0 in.) of the same redbed shale revealed peak strengths achieved at lateral strains of approximately 2%, while residual strengths were achieved at lateral strains of approximately 5%. The summary of tests on those larger samples is shown in Figure 8. Further, the residual strength values measured in all of those tests could be represented as: effective angle of internal friction (φ') = 14^0 and effective cohesion (c') = 0. This is generally consistent with Stantec geotechnical engineering practice, where measured and back calculated residual strengths of weaker Pennsylvanian shales have varied from φ' = 13^0 to 18^0.

Alternatively, in absence of such shear strength test results for a particular site, some approximations of applicable strengths may be developed using generally accepted approaches such as that put forth by Stark et. al. (2005). This approach utilizes generally available (or readily developable) engineering properties (i.e., liquid limit, clay fraction) for a subject stratum as the basis for estimation of an applicable residual shear strength value.

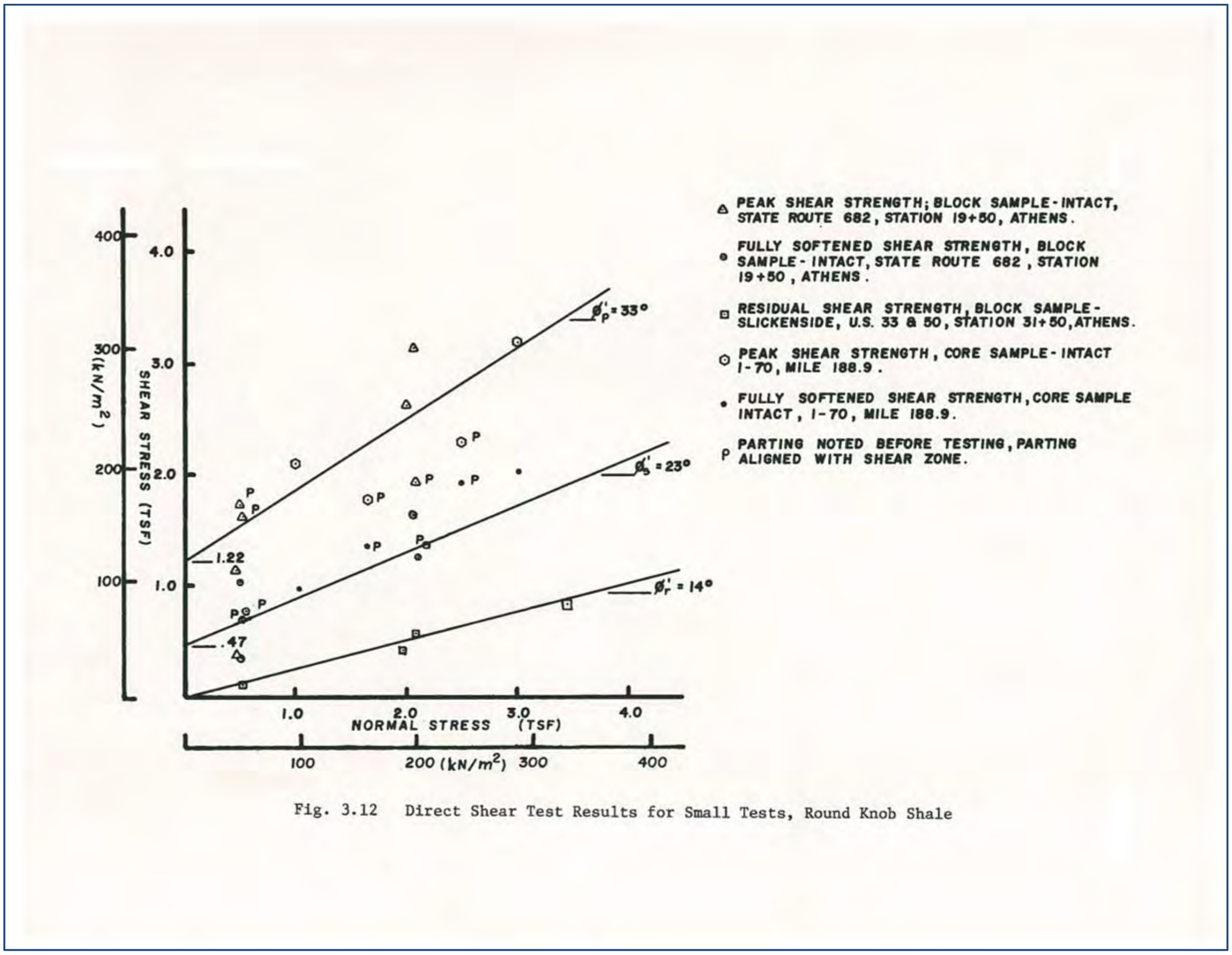


Figure 7 Direct Shear Test Results; Round Knob Shale – Small Specimens

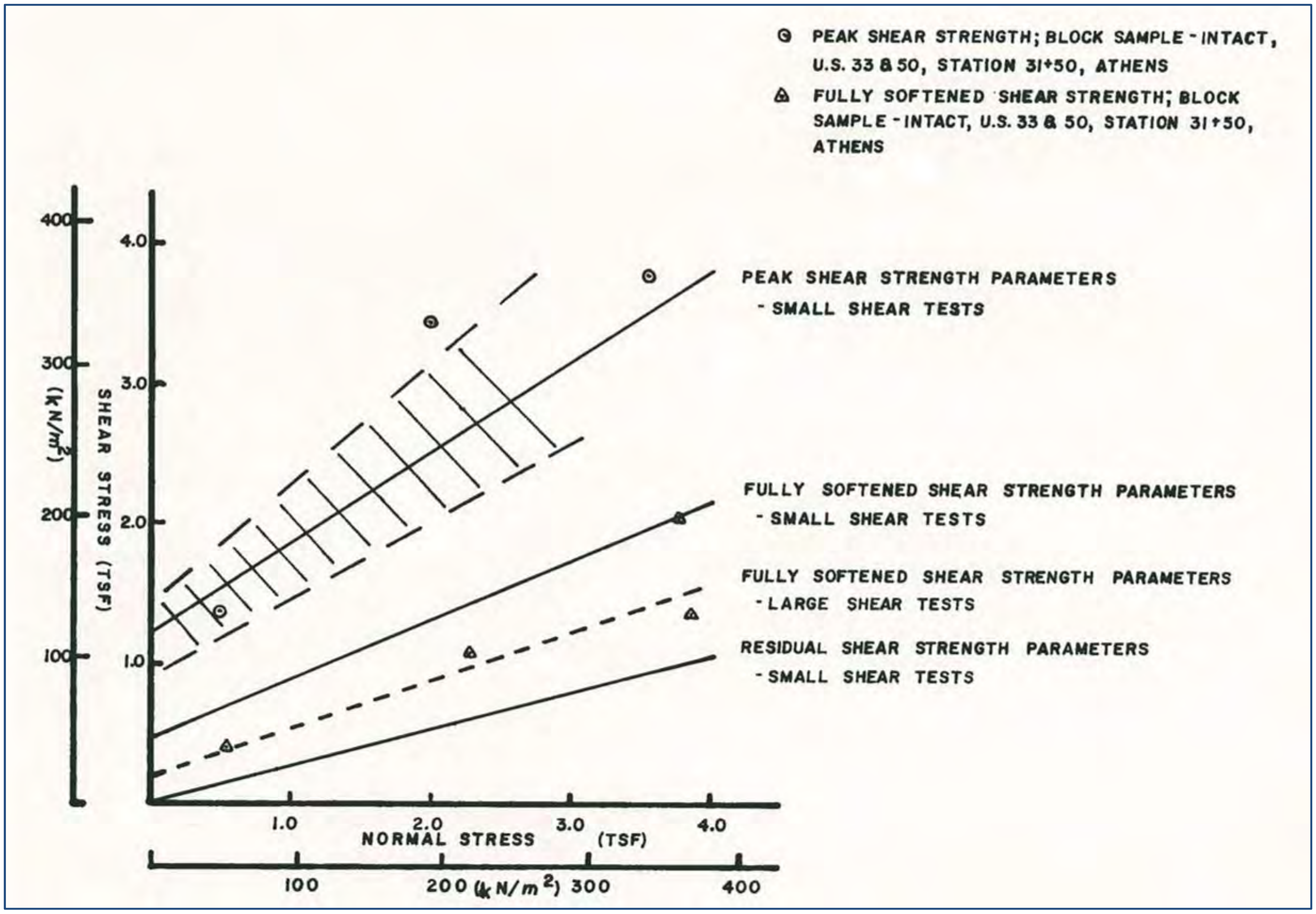


Figure 8 **Direct Shear Test Results; Round Knob Shale**

GENERAL CONSTRUCTION GUIDELINES

For portions of projects dissecting Pennsylvanian stratigraphy, initial observations of the existing landscape are primary indicators of likely future behavior of excavated slopes and embankment performance. At the inception of design, a reconnaissance of the prospective roadway alignment should be performed to delineate any indicators of past landslide movement (as well as the limits), exposures of bedrock and/or soil slopes (and the durability and inclinations of those exposures) and any of the sources for any prominent erosional features within the prospective corridor.

The general inclination of the naturally occurring topography within the roadway corridor is also a good indication of the overall strength of underlying deposits, as overburden typically seeks an inclination near equilibrium. Within the Pennsylvanian redbed strata, the naturally occurring topography is often sloped with an inclination between 3:1 and 4:1 (H:V). This inclination corresponds closely with the range of residual strength values discussed within the previous section of this report. Consequently, design and construction of engineered embankments and cut slopes within these strata should respect these strength parameters. To attempt otherwise may be incorporating a degree of risk inconsistent with overall project requirements or intent. This does not discount the incorporation of overall soil strength enhancement approaches such as internally-reinforced soil slopes or soil stabilization methods with chemical admixtures as a means to construct embankments with steeper inclinations than previously listed here. However, such enhancements must be appropriately accounted for within the engineering design process and the costs of such enhancements weighed against the costs of larger right-of-way requirements and embankment fill quantities using native material without enhancements.

Additionally, it should be recognized that construction of embankments using the redbed shales requires extra effort by a contractor to appropriately breakdown the shale and properly moisture-condition the shale prior to placement as controlled fill material. Since the redbed shales will ultimately degrade within an embankment to a soil-like material, it should be prepared and compacted during initial construction in a manner consistent with soil-type placement/compaction procedures, not in a manner consistent with a rock-type material. This preparation requires considerable effort on the part of the contractor to apply larger-than-normal amounts of water to the shale material to be used as fill while turning and breaking down the shale as the material degrades with the absorption of the added water. Once the mixture attains a soil-like appearance at the optimum moisture content, only then should it be considered suitable for placement as embankment material.

Within excavated slopes which dissect the redbeds, the previously mentioned slope inclinations of 3:1 (H:V) are generally appropriate for design, unless evidence of previous movement has been documented; wherein, it is more appropriate to design slopes with a 4:1 (H:V) inclination – but only within that delineated zone. When the redbeds are included within a cut slope exposure which is bounded vertically by more competent bedrock, it is common to design the slope with steeper inclinations within the exposures of the more competent bedrock and limit the recommended inclinations to the redbed exposures. Additionally, a bench is often excavated within the slope configuration at the base of the redbeds to minimize movement of degraded redbed material downslope.

PROJECT DISCUSSIONS

SR 7, Lawrence Co., OH

The State Route 7 Chesapeake Bypass in a project that has been fully designed but only partially constructed. The reason for the partial construction is that, during the construction on one section of the alignment, numerous landslides developed on constructed slopes that necessitated ODOT to halt construction to consider alignment shifts and changes in design assumptions. Stantec is currently performing additional geotechnical borings in the area as a result of alignment shifting that reduced the amount of sidehill cut and fill sections.

The bedrock geology underlying the Chesapeake Bypass alignment consists of soft multicolored shales of the Conemaugh and Monongahela formations. During the original design, the long term shear strengths assumed for these materials ranged from 18^0 to 19^0 for the internal angle of friction (φ-angle) and from 0 to 300 psf for cohesion. A lower φ-angle of 14^0 was assumed for material considered to have been formed in residuum of the shale bedrock or for colluvium (residual material or bedrock that has moved from its original position during the past). Based on these assumptions, cut slopes of 3:1 (H:V) were initially recommended for the shale bedrock and 4:1 (H:V) for residual or colluvial material. Due to right-of-way restrictions, cut slopes of 2.5:1 (H:V) for cuts in shale and 3:1 (H:V) for cuts in residual or colluvial material were designed and constructed. As stated earlier, dish-shaped failures developed in the cut slopes during construction, indicating that the cut slopes were designed with an inclination that was too steep in relation to the low shear strength of the naturally occurring material.

The currently completed section of the alignment was designed as a four-lane divided section; however, due to budget concerns, only the eastbound lanes were constructed and the road currently functions as a “super-two lane” roadway. As may be noted in Figures 9 to 11, the cut slopes have been inclined at a maximum 3:1 (H:V) with no instabilities observed since completion of construction over 5 years ago. Figure 12 shows the typical erosion and degradation of the surface of exposed redbed strata, even when cut within design parameters, often resulting in difficulty in establishment of meaningful vegetative cover. It may be noted in Figure 13 that steeper cut slope inclinations have been constructed within the more competent bedrock exposures that were present within the upper regions of this cut.

Figure 9 LAW-7; Cut Slope

Figure 10 LAW-7; Cut Slope

Figure 11 LAW-7; Cut Slope

Figure 12 LAW-7; Redbed Exposure

Figure 13 LAW-7; Constructed Cut Slope

MEG-124-31.57, Meigs Co., OH

The Ohio Department of Transportation (ODOT) constructed a new roadway (State Route 124), which is referred to as the Ravenswood Connector, in Meigs County, Ohio. Stantec performed the geotechnical exploration on a 6.6-mile segment of this roadway that connects Ravenswood, West Virginia and Pomeroy, Ohio.

The existing topography of a 4-mile section of this alignment consisted of steep hills and deep hollows with a maximum vertical relief of approximately 260 feet. Bedrock geology mapping (Reconnaissance Bedrock Geological Map of the Ravenswood, West Virginia-Ohio Quadrangle, Ohio Department of Natural Resources [ODNR], 1997) indicated that overburden material are underlain by sedimentary bedrock of the Dunkard Group of the Permian or Pennsylvanian geologic age and the Monongahela Group of the Pennsylvanian geologic age. Descriptions of these bedrock units are as follows according to ODNR:

- The Dunkard Group (Permian/Pennsylvanian) is comprised of sandstone, siltstone and shale with relatively thin seams of limestone and coal. This formation is described as brown, gray, red and green in color and thin to massive bedded. It is also characterized by rapid horizontal and vertical changes in rock type. The Washington and Waynesburg coal seam is present within these formations.

- The Monongahela Group (Pennsylvanian) consists of shale, siltstone, limestone, sandstone and coal. The bedrock is typically gray and green and infrequently red in color and non-bedded to massive bedded. The Pittsburgh coal bed is present within this formation.

Bedrock encountered within the core borings drilled along the project corridor correlated well with referenced geologic mapping (although coal was not present in the area). The rock core (see Figure 14) obtained from the proposed cuts consisted predominantly of non-durable shale and, to a lesser degree, durable shale, sandstone and siltstone. Rock Quality Designation (RQD) values were frequently less than 20 and rock cores contained numerous slickensides (see Figure 15). Slake Durability Index (SDI) testing on the non-durable shale yielded results that were predominantly less than 20 percent, and often 0 percent. These conditions resulted in the recommended use of 2:1 (H:V) rock cut slopes with 20-foot wide intermediate benches positioned roughly every 100 feet along the face of the cut (see Figure 16). The benches were sloped to promote drainage (see Figure 17). The use of serrated slopes consisting of 2-foot horizontal and vertical steps was recommended to promote vegetation growth. ODOT decided against constructing these steeped slopes due to unfavorable performance on past projects.

Although not ideal, the non-durable shales were also the most available material for embankment construction (see Figure 18). Embankment slope stability analyses were performed modeling both 2:1 (H:V) and 3:1 (H:V) slopes, with embankment shear strength assumptions of 1,500 psf for undrained cohesion, 200 psf for drained cohesion, and 20^0 for drained internal angle of friction. Generally, results of the stability analyses determined that for embankments over 20 feet high, 3:1 (H:V) slopes were necessary for adequate factors of safety to be achieved. The primary controlling factor was the low anticipated long-term shear strength of the embankment materials. Where construction of the 3:1 (H:V) slopes were not feasible due to right-of-way restrictions, geogrid-reinforced slopes (see Figure 19) were constructed with 1.5:1 (H:V) slopes using the non-durable shale as the primary embankment component.

Approximately 10 years after construction, ODOT reports that the performance of the cut and embankment slopes on the project have been favorable. During construction, some of the slopes had to be regraded and reseeded due to some minor sloughing and erosion. Long term, some areas have eroded into a washboard-shaped pattern on the slope, but have not affected the ditch or roadway.

Figure 14 MEG-124; Core Box Containing Redbed Shale Sample

Figure 15 MEG-124; Example of Slickensided Bedrock Surface

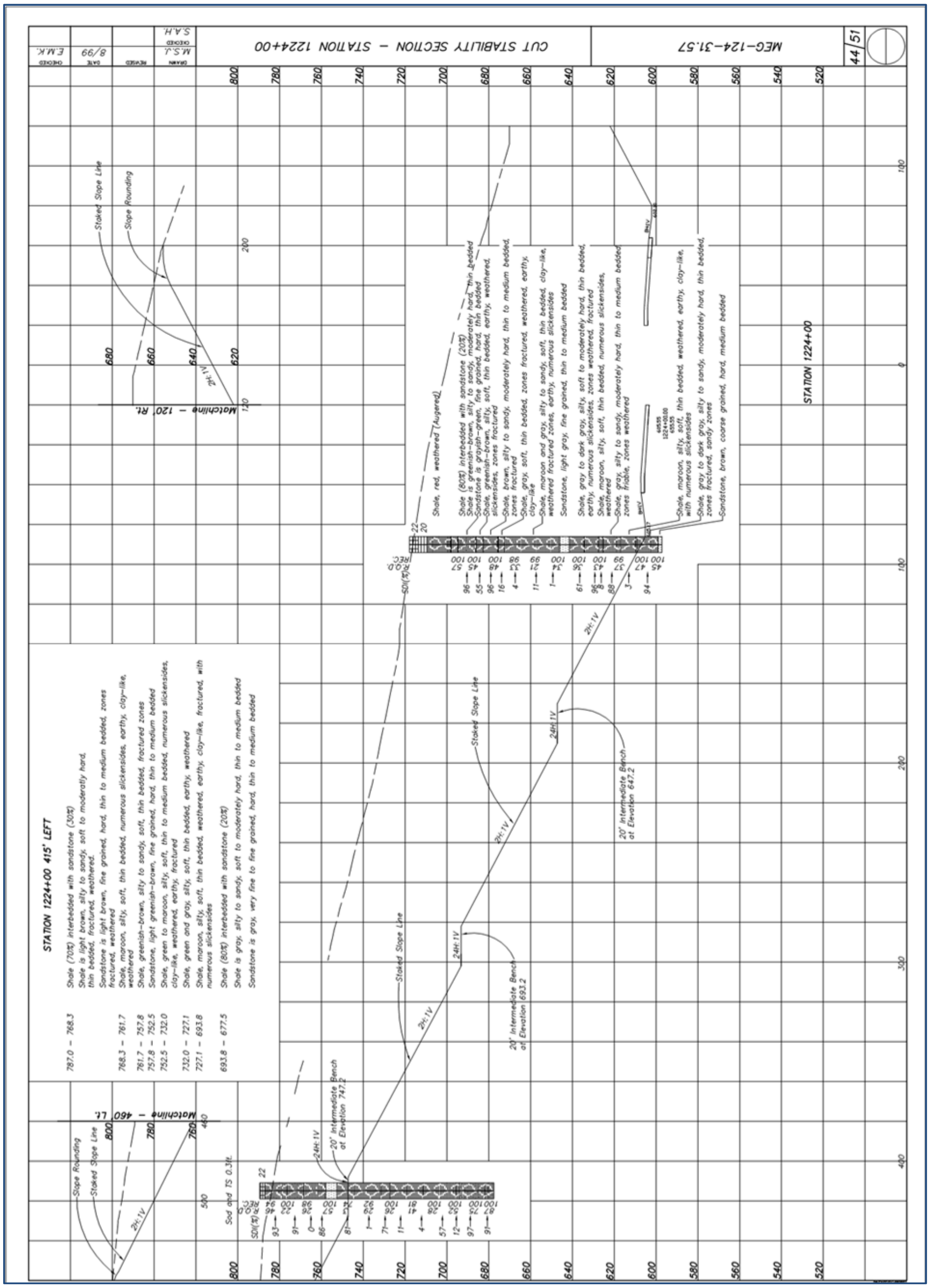


Figure 16 MEG-124; Cut Section Design

Figure 17 MEG-124; Constructed Cut Slope

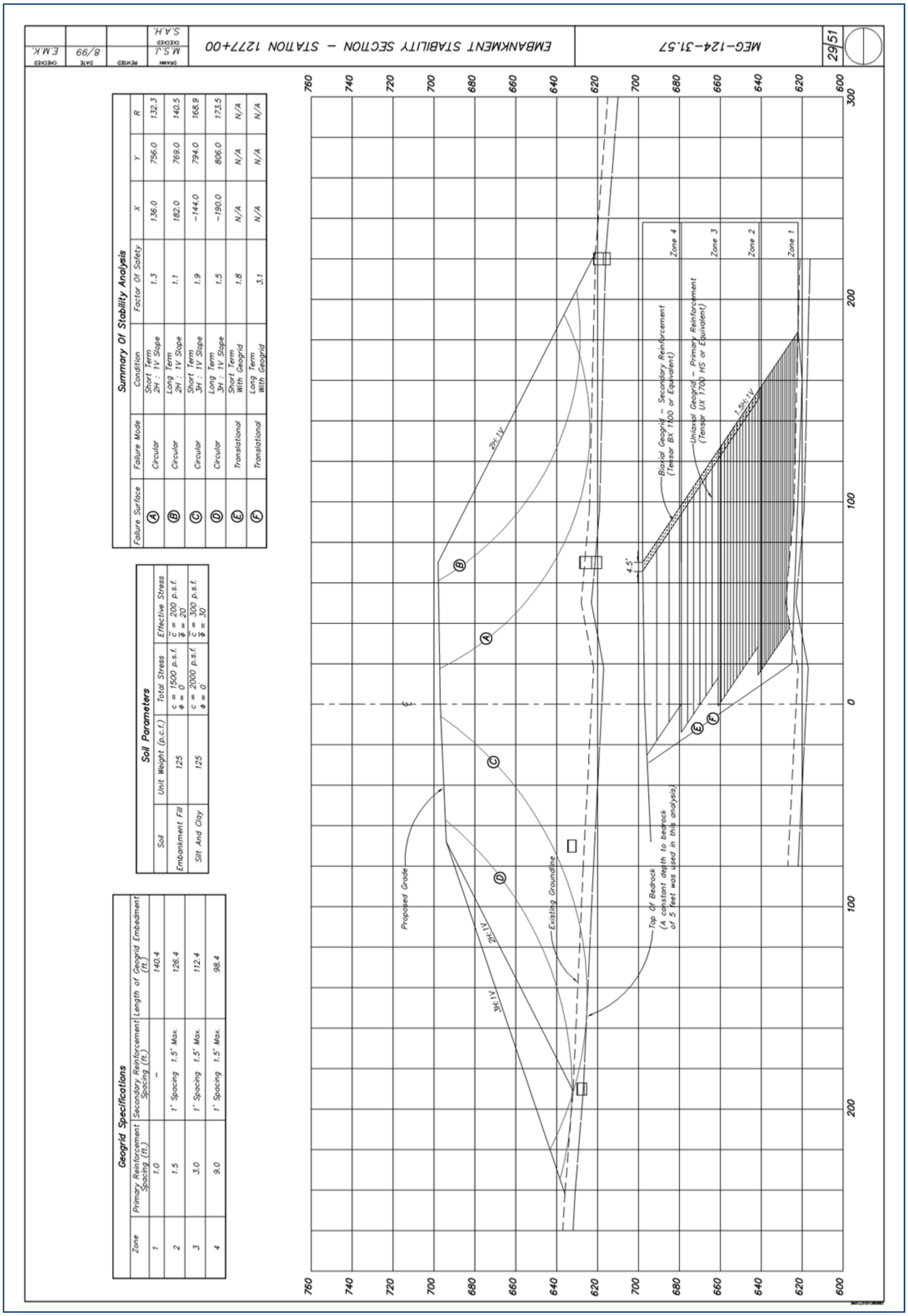


Figure 18 MEG-124; Embankment Design Section Example

Figure 19 MEG-124; Constructed Reinforced Soil Slope

I-64, Industrial Parkway, Boyd/Carter/Greenup Cos., KY

The Kentucky Transportation Cabinet (KYTC) constructed a $90 million roadway in Boyd, Carter and Greenup Counties named the Industrial Parkway from 1998 - 2003. The Industrial Parkway begins with an intersection at I-64 about six miles east of Grayson, extends 14.5 miles northward and ends at-grade with US 23 near Wurtland. In 1995, the KYTC selected a design team which included Stantec Consulting Services Inc. (formerly FMSM Engineers) to design and oversee construction of the Industrial Parkway. Stantec was the geotechnical engineering consultant responsible for all geological studies and geotechnical exploration efforts associated with this corridor.

The Industrial Parkway is situated within the East Kentucky Coal Field physiographic province. Although naturally characterized by narrow ridge tops and steep-sided, “V” shaped valleys formed by erosional dissection of regional sedimentary rocks, the terrain over much of the corridor has been significantly altered by coal mining, petroleum exploration and landfill activities. Such operations have created areas of flat to gently rolling topography, exposed and buried rock highwalls, mine adits, deep mine spoil storage areas, sediment ponds and numerous other disturbances. Maximum topographic relief within the roadway corridor is on the order of 400 feet.

The Argillite (1962) and Greenup (1966) USGS geologic quadrangle maps indicate the region is underlain by bedrock belonging to the Breathitt and Conemaugh formations. The Breathitt formation consists of cyclic sequences of interbedded sandstone, siltstone, shale and coal (including the Princess Nos. 3, 4, 5, 6, 7 and 8 seams), formed from sediments deposited during the Middle Pennsylvanian geologic period. The Conemaugh formation is primarily situated along the tops of ridges above the Breathitt formation, and consists mostly of interbedded shale, siltstone, and sandstone formed from sediments deposited during the Upper Pennsylvanian period. Shale within these formations in this region of the state is commonly very non-durable and degrades quickly when exposed to weathering elements.

The conditions left behind that resulted from past mining activities presented some of the most significant challenges in designing and building the Industrial Parkway. Figures 20 through 22 illustrate some of the conditions present at the site at the beginning of the geotechnical field exploration efforts. The geotechnical explorations conducted for this roadway project included testing of the rock core samples obtained primarily within planned roadway cuts. Slake Durability Index (SDI) testing resulted in ninety-seven percent that yielded SDI values less than 95 percent (classification of Non-Durable, Class I), including almost forty percent of the shale samples classified as Non-Durable, Class III which correspond to materials most prone to weathering and slope degradation. Additional considerations identified included potential acid-producing rock strata. Figure 23 shows construction of a roadway cut interval.

In conjunction with rock core testing, extensive soil testing was performed and this included testing on soil and shale materials derived from the Conemaugh formation. Table 1 presents a summary of undrained shear strength values obtained from this soil testing, and which were used in performing slope stability analyses for this roadway project.

Table 1. Results of CU Triaxial Tests – Industrial Pkwy.

Sample Description and USCS Classification	Range of Values Obtained	
	Cohesion, $\bar{c}$ (psf)	PHI, $\bar{\phi}$ (degrees)
Lean Clay – CL	210 - 240	25 - 28
Silty Clay, Sandy Lean Clay, or Clayey Sand – CL, SC	0 - 70	31 - 32
Mine Spoils – CL	0	26

Recommended slope geometries were based upon field conditions, subsurface data, selected roadway cross-sections, regional and local geology, engineering analyses, and experience gained from design of cut and embankment slopes in similar geologic conditions. Typically, slope grades of ½:1(H:V) were recommended for cut slopes in rock, with the exception of those zones exhibiting low SDI values. Within these types of strata, slopes of 2:1 (H:V) were recommended.

Table 2 summarizes typical slope geometries applied to cuts in mine spoil materials and fills constructed of non-durable shale/mine spoils. Figures 24 and 25 show a model used during the stability analysis and the constructed approach embankment for one of the several crossings along the Industrial Parkway. Figure 24 depicts an approach embankment constructed over a buried highwall which, in turn, was filled in with mine spoils after coal extraction during mine activities. The approach slopes were designed using 3:1 (H:V) side slopes, given that the materials used consisted primarily of non-durable shales or mine spoils. Figure 26 exhibits the Industrial Parkway-Interstate 64 interchange after construction was complete**,** shown on this photograph are some of the crushed limestone lined ditches used to mitigate acid runoff.

Table 2. Slope Geometries for Soil/Mine Spoil Cuts – Industrial Pkwy.

Approximate Depth of Cut	Recommended Slope Grade (H:V)
Less than 10 feet	2:1
10 feet to 30 feet	2.5:1
30 feet to 120 feet	3:1

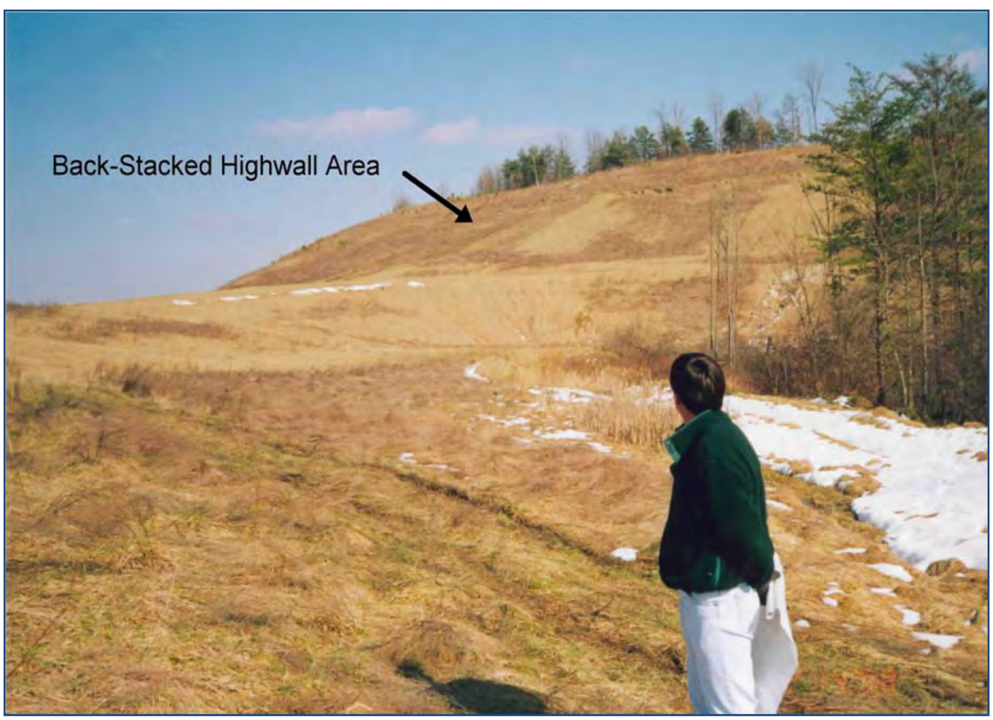


Figure 20 Back-Stacked Highwall Area

Figure 21 Silt Pond and Mine Bench Area Encountered at Project Site

Figure 22 Typical Hollow Fill Encountered Along the Project

Figure 23 Roadway Cut being Constructed within Non-Durable Shales

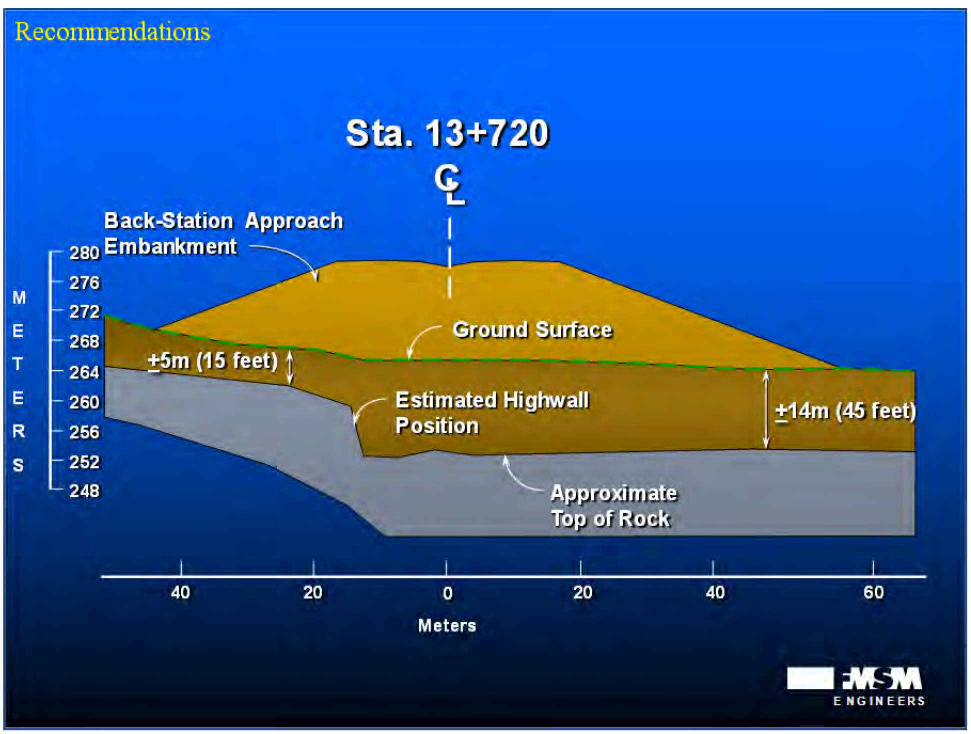


Figure 24 Approach Embankment Modeled for Stability Analysis

Figure 25 View of Approach Embankment after Construction

Figure 26 Industrial Parkway/I-64 Interchange

REFERENCES

Condit, D. D., Conemaugh Formation in Ohio, 4th series, *Bulletin 17*, Geological Survey of Ohio, 1912.

Fisher, S. P., Fanaff, A. S. and Picking, L. W., “Landslides in Southeastern Ohio”, The Ohio Journal of Science, vol. 68, no. 2, March, 1968.

Stark, T. D., Choi, H. and McCone, S., “Drained Shear Strength Parameters for Analysis of Landslides”, Journal of Geotechnical and Environmental Engineering, American Society of Civil Engineers, May 2005.

Williams, R. L., “Progressive Failure of Red Conemaugh Shale”. Ph.D. Thesis, The Ohio State University, Columbus, Ohio, March, 1982.

Discussion of the Roadway Cut−Section in the Hartshorne Formation
Gowen, Oklahoma

By

Vincent G. Reidenbach[1] and James B. Nevels, Jr.[2]

1. Vincent G. Reidenbach, Ph.D., P.E., Geotechnical Engineer Materials Division, Oklahoma Department of Transportation, 200 N.E. 21st Street, Oklahoma City, OK 73105; PH (405) 522-4998; FAX (405) 521-0522; email: vreidenbach@odot.org and 2. James B. Nevels, Jr., Ph.D., P.E., Geotechnical Engineering Consultant, PH (405)-818-2897, email: jnevels1@cox.net

Abstract. This paper presents the preliminary field work required to access site for a cut−section investigation (a very contemptuous case), the surface soils and site geology, variation in the site geology along the cut−section extent, laboratory analysis of the cut−section core data, and cut−section slope design. Extraordinary steps were made in an effort to accommodate the property owner namely in the redesigns of the alignment and the minimization of tree cutting and access road building. The surface soils were very shallow residual soils. The underlying geology was first anticipated to be a potential thick sandstone of the Hartshorne formation as written up in the geologic descriptions; however, this did not prove to be the case. The sandstone as described in the geologic publications was found at the beginning of the project and where the sandstone was exposed on the surface near the east end of the project turned out to be a shallow cap rock. Thick shale sequences were encountered with only minor sandstone layers found. The laboratory testing of the rock core samples included the following: unit weight, point load strength index, unconfined compressive strength, and slake durability. The shale was observed to air slake when exposed to the atmosphere. Test results on the shale proved that the slake durability was a critical issue. The investigation included 20 core borings and refraction seismic surveys. The cut−section slope design was eventually decided on a 3:1 slope ratio because of the slake durability of the shale.

Introduction. The Oklahoma Department of transportation (ODOT) had planned for a roadway alignment improvement on a section of US 270 just east of Gowen, Oklahoma to straighten out an S shaped curve which had a accident rate, see Figure 1. The project was given to the ODOT Materials Division Geotechnical Branch to proceed with a geotechnical investigation in March 2011 with an expected date for the completion of the preliminary geotechnical investigation of June 2010.

At the start of the geotechnical investigation, the preliminary surface soil survey, walk−outs of the alignment, and layout of the cut−section borings were all initiated. However, as the start of the drilling of the planned cut−section borings, the property owner refused to let the Materials Division drill crew proceed with the borings. The property owner demanded to be paid damages following the completion of each boring. Following the property owner's denial of right of entry with regard to the borings, months of negotiation ensued which involved the central office Materials, Right of Way, and Roadway Design Divisions; and the Division 2 Engineer and Maintenance staff. Finally under threat by the Division 2 Engineer invoking a seldom used state statute allowing legal entry by bringing in the State Highway Patrol and Latimer County Sheriff, the property owner relented and allowed the Materials Division Geotechnical Branch drill crew to enter the property. The drilling started in early December 2010 and finally completed in late May 2011.

This was only the beginning of a very tense and contemptuous arrangement. The property owner insisted on controlling the access of the drill rig and water truck at each drill hole location. The Latimer County maintenance brought in prison crews to clear cut the necessary pine timber and lesser scrub timber, stack and stump grind as much as possible and the rest hauled off. Numerous site grading and the hauling in of bridging material (aggregate materials) were made at selected locations to access the drill rig and water trucks. The property owner also wanted the alignment moved further south from his home on the west end of the project and further north of the pond dam on the east end of the project. The Roadway Design Division made two changes in the alignment trying to appease him before he was satisfied.

Site Description. The site location as seen in Figure 1 is just inside the western boundary of Latimer County between the McAlester in Pittsburg County and Wilburton in Latimer County on US 270 highway. An aerial map of the project extent is presented in Figure 2. As can be seen from Figure 2 the site is heavily timbered in a mix of pine, oak, hickory, and various scrub vegetation. Also of note in the eastern part of alignment is a large pond south of centerline of survey. The alignment crosses a northeast projecting cuesta as seen in Figure 3.

The site lies in the McAlester Marginal Hills Belt geomorphic province which consists of resistant Pennsylvanian sandstones capped broad hills and mountains rising 300 to 2000 feet above wide hilly plains consisting mostly of shale.

Soils and Geology. The surface soils are mapped along the alignment extent starting from the beginning of the project according to the Natural Resources and Conservation Service (NRCS) Web Soil Survey 2.2 include the following soil series: Clebit−Pirum complex, Carnisaw−Clebit association, and the Carnesaw−Pirum−Clibit association. These soils are all very shallow to shallow stiff, residual soils containing sandstone fragments.

According to the (ODOT) Division Two Red Book the underlying geology for this project alignment is the Hartshorne unit (Phs). The Hartshorne unit consists mostly of sandstones, with some siltstone, shale, and a few thin coal beds. The lower portion of the unit consists of dense, massive though thin−bedded, gray sandstones which are finely conglomeritic. The interbedded shales shales are thin, silty, and gray. The middle portion of the unit consists of about 50 feet of gray platy shales. The upper 75 feet of the unit consists of thick, generally massive, gray to brown sandstones. The total thickness of the Hartshorne unit varies from 100 to 400 feet, but is usually about 225 feet. The alignment is shown crossing the Hartshorne unit in Figure 4.

According to the Oklahoma Geological Survey Hydrological Atlas 9, the geology is recorded as the Hartshore Formation (Pha). The Hartshorne formation consists of sandstone tan to gray to white sandstone with some interbedded gray shale.

The Oklahoma Geological Survey has a definitive geologic statement of the site geology in the Gowen Quadraingle by LeRoy Hemish , 1992. The Hartshorne formation of Pennsylvanian geologic age consists of the following: grayish−orange (10YR7/4) to moderate−reddish−orange (10YR6/6) to very light−gray (N8), very fine−grained, ripple−marked, bioturbated, thin−bedded to massive sandstone interbedded with silty, medium−gray (N5) shale. Thicknesses of the formation are approximately 250 to 300 feet.

Cut−Section Borings. A total of twelve borings were planned for this geotechnical investigation placed at two hundred feet apart. In addition at approximately stations 422+00 and 435+00 cross−sections were developed by adding four additional borings at each station. The rock in all 20 borings were cored with continuously cored with an NQ wire line coring The total number of borings made covering the plan cut−section extent was 20, see Figure 5. Note that the cut depth is close to 50 feet. The plan layout of the borings is presented in Figure 6. The borings were supplemented with five seismic refraction shot lines. Typical borings are presented in Appendix A.

Three drill and log borings were made directly along the pond dam extent to check groundwater seepage. The result of these borings indicate no seepage into the bore holes.

Laboratory Testing. The laboratory testing schedule for the rock samples collected from the core runs included the unit weight, the point load strength index, unconfined compressive strength, and the slake durability being tested according to the current AASHTO T 233 and ASTM D 5731, D 7012, and D4644 test standards respectively. These test results and detailed rock sample descriptions are still under evaluation.

Assessment of Findings. There were limited exposures of the in−place bedrock between stations 434+00 and 436+00 on the ground surface. A total of 50 dip and dip direction measurements was made with a clar compass resulting in an average dip of 8.7^o and dip direction of $SE187^o$. Around the area shown in Figure 2 off the alignment, similar clar compass readings were seen.

The borings indicate the following sequence: a) a shallow yellowish−red residual clay soil, b) an olive brown to olive gray shale, and c) underlain by alternating beds of light gray to gray sandstone and dark gray shale. The shale with depth from boring 9 forward appears to become softer with depth, see Figure 4.

Design Cross−Sections. Typical cross−sections are presented at stations 422+00 and 435+00 in Appendix B. The cross−sections presented cover the standard ODOT 3: 1 slope ratio design and the inclusion of right of way retaining walls. A design decision is pending on the height of the retaining walls to place along alignment extent. The Roadway Design Division is limited to 15 foot high retaining walls.

Summary and Conclusions. The preliminary review of the site geology indicated the possibility of massive sandstone in the Hartshorne formation; however, this was found not to be the case. The sandstone was more thin−bedded than massive. There were a significant number of large sandstone float rock on the ground surface.

The traditional ODOT slope design is a 3:1 slope ratio for roadway slopes in earth materials and shales. For this project with varying sandstone beds predominately near the top of cut section but at lower depths as well, the following two slope designs are being considered.

1. The traditional ODOT slope design is a 3:1 slope ratio.
2. A flatter approximately 2.5:1 slope ratio with a 7 foot high retaining wall near the toe of slope and a wider ditch section (8−foot). The height of the right of way retaining walls is still pending.

The results of the five seismic refraction shot lines indicate no problem with the rippability of the sandstone and shale beds. The core logs indicate that the sandstone and shale are predominantly thin−bedded and moderately hard.

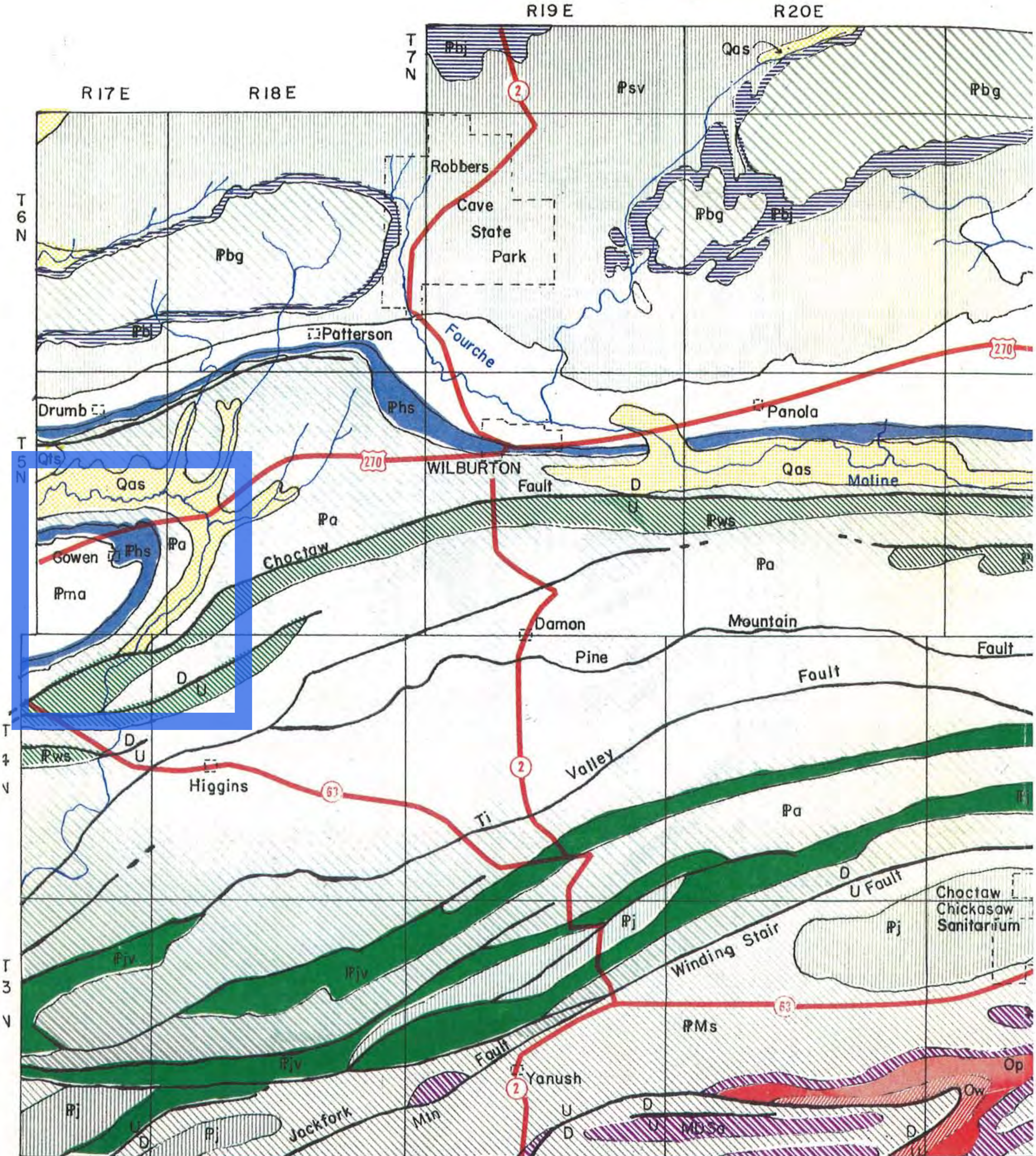
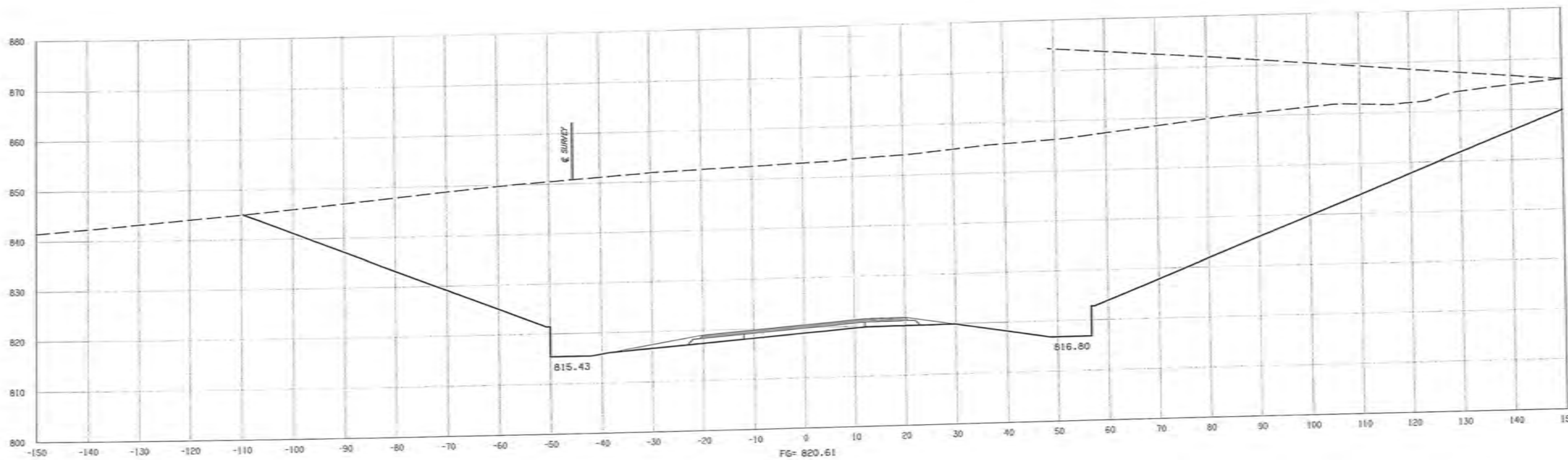
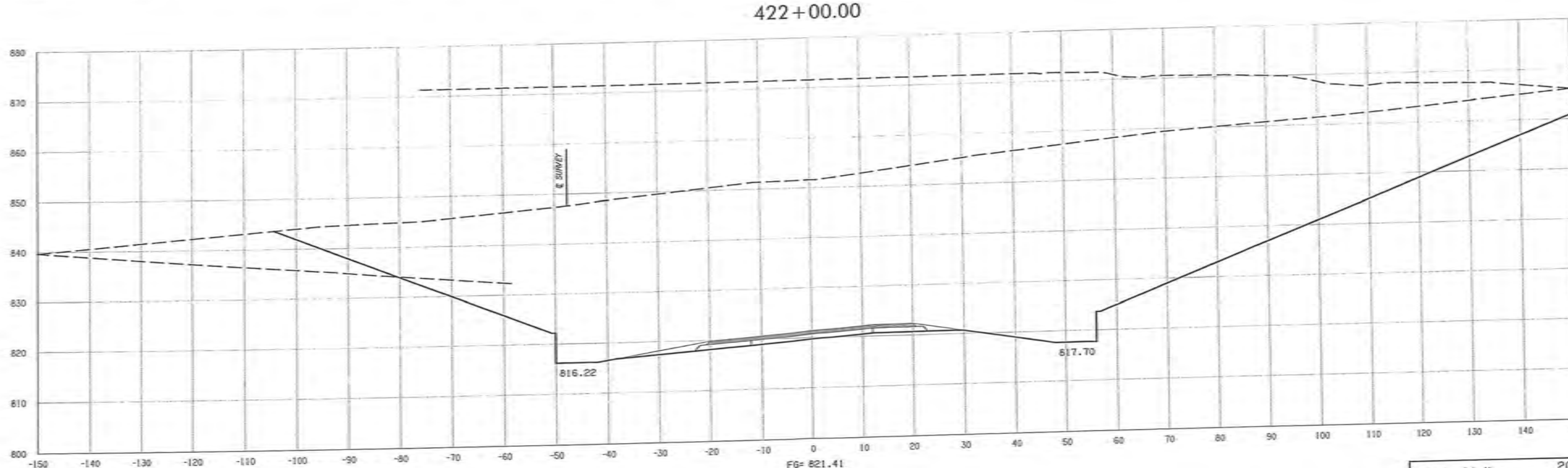


Figure 6. Site Geology

OKLAHOMA DEPARTMENT OF TRANSPORTATION
PRELIMINARY PLANS
13-MAY-2011



422+00.00



421+00.00

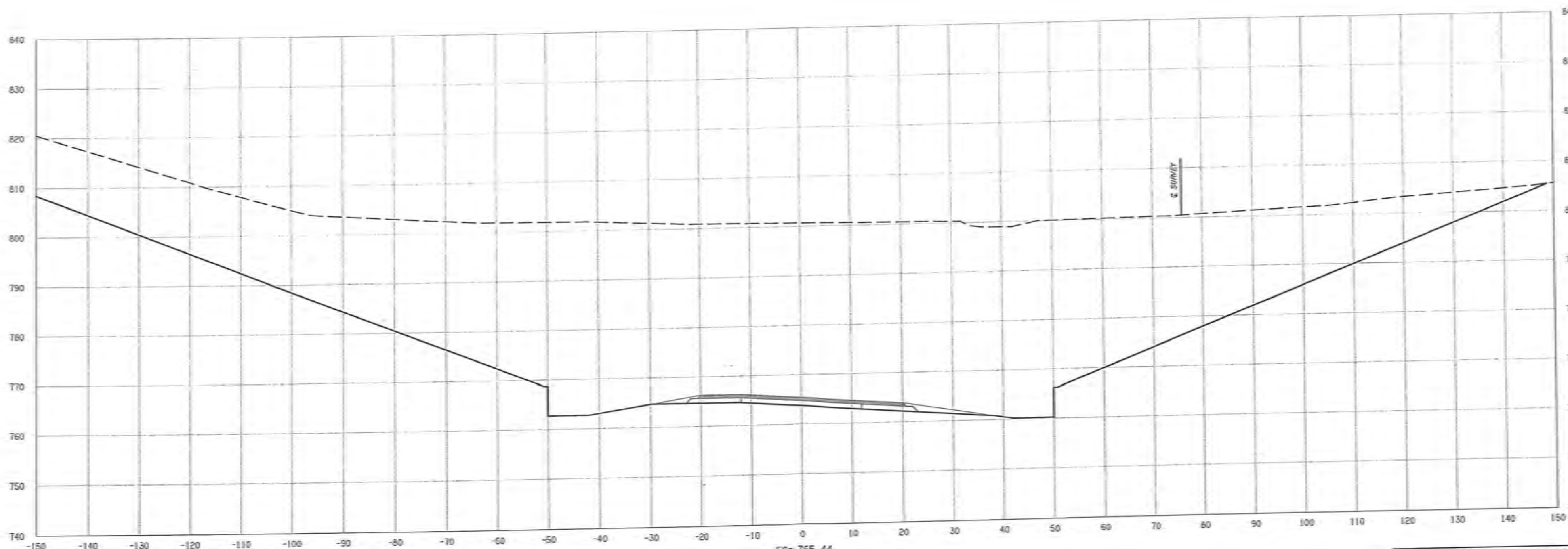
State Job No. 20922(04)
LATIMER County Sheet No. 16)

RDYPC72 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 2:46:08 PM 5/13/2011

OKLAHOMA DEPARTMENT OF TRANSPORTATION

PRELIMINARY PLANS

13-MAY-2011



435+00.00

State Job No. 20922(04)

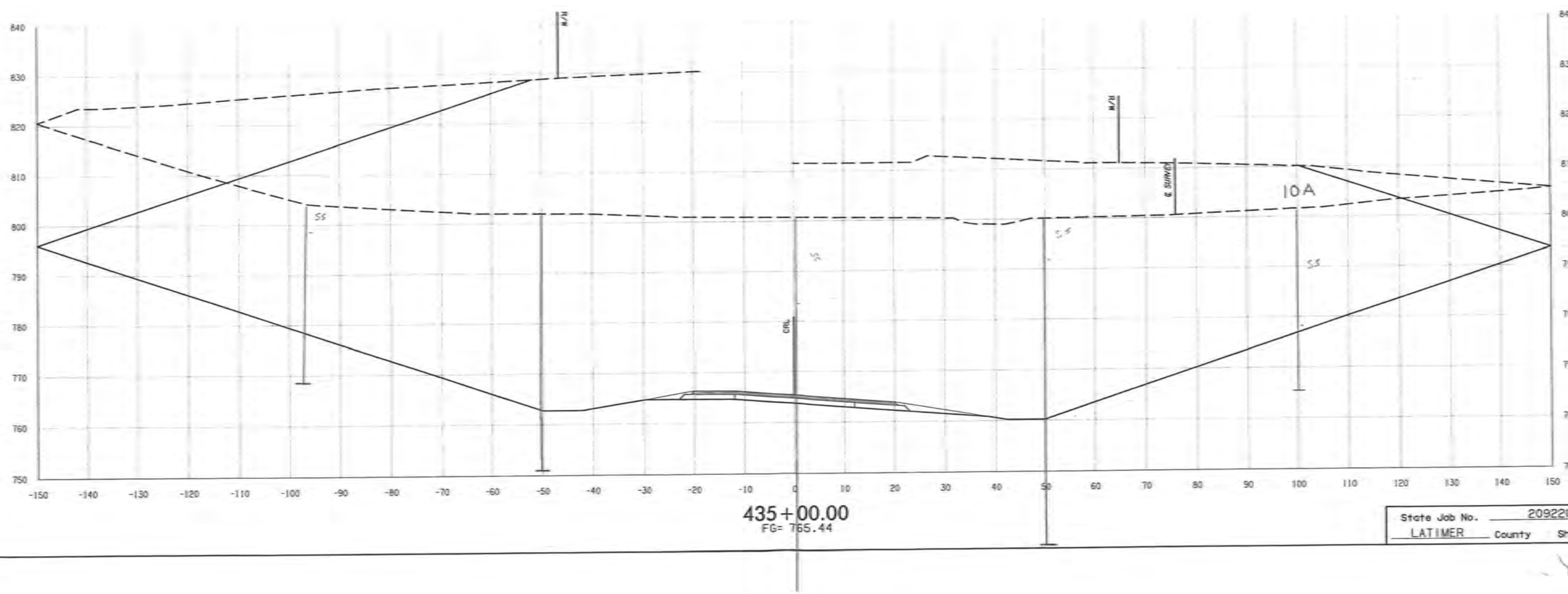
LATIMER County Sheet No. 27X

RDYPC72 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 2:46:15 PM 5/13/2011

OKLAHOMA DEPARTMENT OF TRANSPORTATION

PRELIMINARY PLANS

03-JAN-2011



435+00.00
FG= 765.44

State Job No. 20922(04)

LATIMER County Sheet No. 23X

RDYPC51 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 9:52:27 AM 1/3/2011

BORING 6

STATION 428+42.9

ELEVATION 818.07 FEET

Gravel, red, with lean clay and sandstone fragments yellowish red, moist, dense (old road fill)

3.0

Lean clay, yellowish red, moist, stiff

6.0

Shale, weathered, gray lensed yellowish brown, moist, moderately hard

9.3

Shale dark gray, with sandstone stringers yellowish brown, thin bedded, moderately hard

20.0

Shale, dark gray, with calcium partings, moist, thin bedded, moderately hard

25.0

Shale, dark gray, with calcium parting, moist, thick bedded, moderately hard

60.0

60 SCALE

BORING 2

STATION 419 + 64.1, 5.5 FT. LT.

ELEVATION 845.91 FEET

Lean clay with sandstone fragments, yellowish red, moist, stiff

2.8

Shale, weathered, olive brown lensed yellowish brown, moist, moderately hard

5.0

Shale, olive brown, moist, thin bedded, moderately hard

9.5

Shale, olive brown, lensed with sandstone stringers yellowish brown, moist, moderately hard

14.5

Sandstone, yellowish brown, lensed shale stringers dark gray, moist, moderately hard

19.5

Sandstone, gray, moist, thin bedded, moderately hard

22.1

Sandstone, gray, lensed with shale stringers, dark gray, moist, moderately hard

24.5

Sandstone, gray, moist, thin bedded, moderately hard

29.5

Shale, dark gray, lensed with sandstone stringers, light gray, moist, thin bedded, moderately hard

44.5

Shale, dark gray, lensed with sandstone stringers light gray, moist, thin bedded, moderately hard

49.5

50 scale

BORING 11

STATION 435+92.7, 14.9 FT. RT.

ELEVATION 791.00 FEET

Lean clay with sandstone fragments, yellowish brown, moist, stiff

1.5

Sandstone, brown, moist,

2.0

Shale, gray, moist, thin bedded, moderately hard

5.2

Shale, dark gray, lensed with sandstone stringers, gray, moist
thin bedded, moderately hard

35.2

Shale, dark gray, moist, thin bedded, moderately hard

40.2

Shale, dark gray, lensed with sandstone stringers, gray, moist,
thin bedded, moderately hard

45.2

50 SCALE

BORING 6

STATION 428+42.9

ELEVATION 818.07 FEET

Gravel, red, with lean clay and sandstone fragments yellowish red, moist, dense (old road fill)

3.0

Lean clay, yellowish red, moist, stiff

6.0

Shale, weathered, gray lensed yellowish brown, moist, moderately hard

9.3

Shale dark gray, with sandstone stringers yellowish brown, thin bedded, moderately hard

20.0

Shale, dark gray, with calcium partings, moist, thin bedded, moderately hard

25.0

Shale, dark gray, with calcium parting, moist, thick bedded, moderately hard

60.0

60 SCALE

BORING 3A

STATION 422+ 39.6, 83.05 FT. RT.

ELEVATION 856.94 FEET

Lean clay, yellowish brown lensed red, moist, stiff

1.9

Sandstone, yellowish brown, moist, hard

2.5

Lean clay, yellowish red, moist, stiff

3.7

Shale, weathered, olive brown, moist, moderately hard

4.5

Sandstone, yellowish brown, moist, hard

4.8

Shale, olive gray, lensed with sandstone stringers, yellowish brown, thin bedded,

10.0

Sandstone, yellowish brown, moist, thin bedded, moderately hard to hard

15.0

Shale, dark gray, with sandstone stringers, gray, moist, thin bedded, moderately hard

35.0

Sandstone, gray, with shale stringers, dark gray, moist, thin bedded, moderately hard to hard

40.0

40 Scale

BORING 2

STATION 419 + 64.1, 5.5 FT. LT.

ELEVATION 845.91 FEET

Depth	Description
	Lean clay with sandstone fragments, yellowish red, moist, stiff
2.8	
	Shale, weathered olive brown lensed yellowish brown, moist, moderately hard
5.0	
	Shale, olive brown, moist, thin bedded, moderately hard
9.5	
	Shale, olive brown, lensed with sandstone stringers yellowish brown, moist, moderately hard
14.5	
	Sandstone, yellowish brown, lensed shale stringers dark gray, moist, moderately hard
19.5	
	Sandstone, gray, moist, thin bedded, moderately hard
22.1	
	Sandstone, gray, lensed with shale stringers, dark gray, moist, moderately hard
24.5	
	Sandstone, gray, moist, thin bedded, moderately hard
29.5	
	Shale, dark gray, lensed with sandstone stringers, light gray, moist, thin bedded, moderately hard
44.5	
	Shale, dark gray, lensed with sandstone stringers light gray, moist, thin bedded, moderately hard
49.5	

50 scale

Boring	Station	Depth
1	418+00	25 Feet
2	420+00	40 "
3	422+50	45 "
3A	"	40 "
3B	"	55 "
3C	"	45 "
3D	"	25 "
4	424+00	45 "
5	426+00	40 "
6	428+00	25 "
7	430+00	20 "
8	432+00	25 "
9	434+00	45 "
10	435+00	45 Feet
10A	"	35 "
10B	"	50 "
10C	"	50 "
10D	"	35 "
11	436+00	45 "
12	438+00	20 "

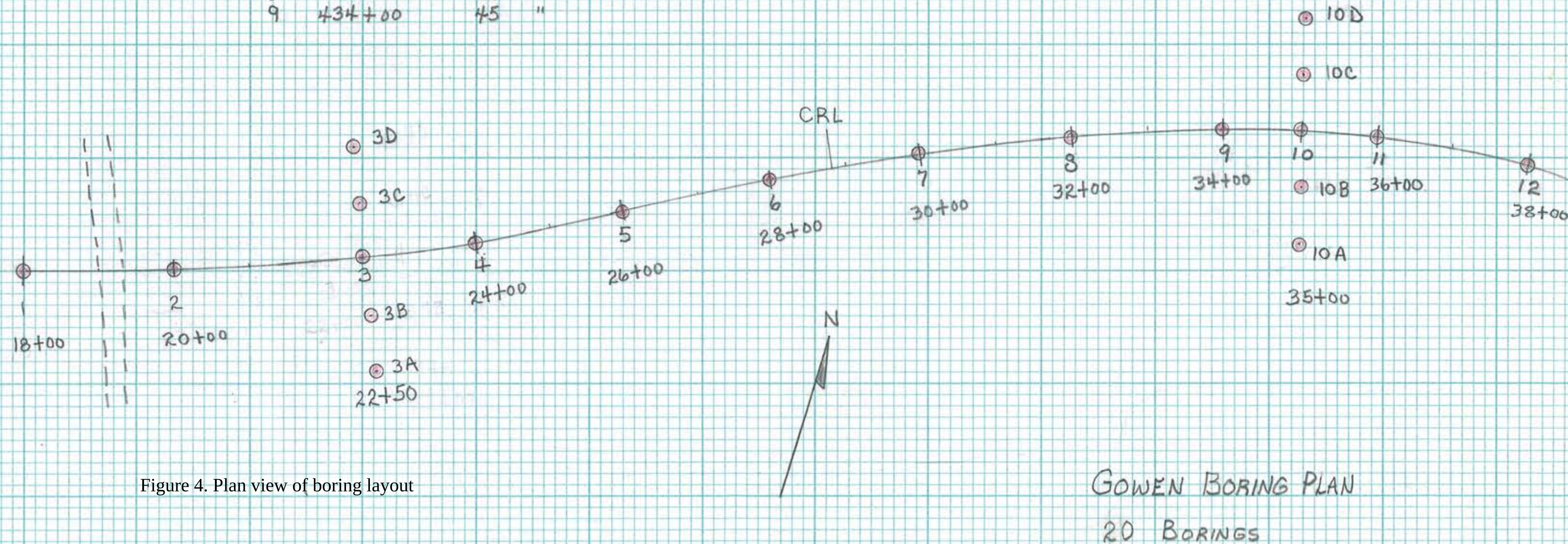


Figure 4. Plan view of boring layout

DATA,
IEETS

STATE OF OKLAHOMA
DEPARTMENT OF TRANSPORTATION

PLAN OF PROPOSED

STATE HIGHWAY

FEDERAL AID PROJECT NO. XXXX-XXXX(XXX)
TYPE HERE JOB TYPE DESCRIPTION
STATE HIGHWAY XXXX

LATIMER COUNTY

CONTROL SECTION NO. 270-39-02
STATE JOB NO. 20922(04)
BRIDGE "X" LOCATION NO. XXXX-XXXXX
EXISTING NBIS NO. XXXXX ;NEW NBI NO. XXXXX

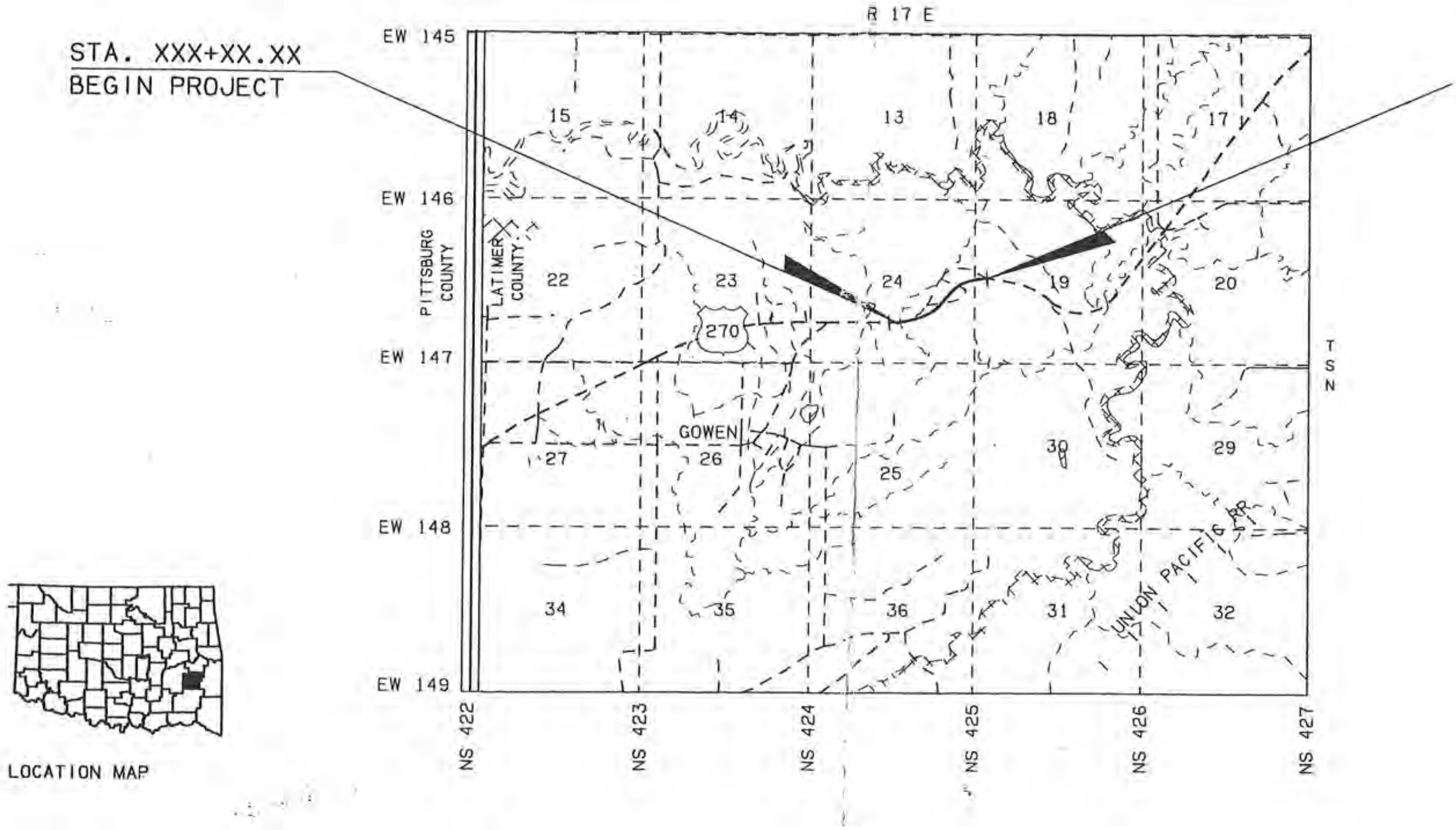


Figure 1. Site Location

Figure 2. Aerial map of the site

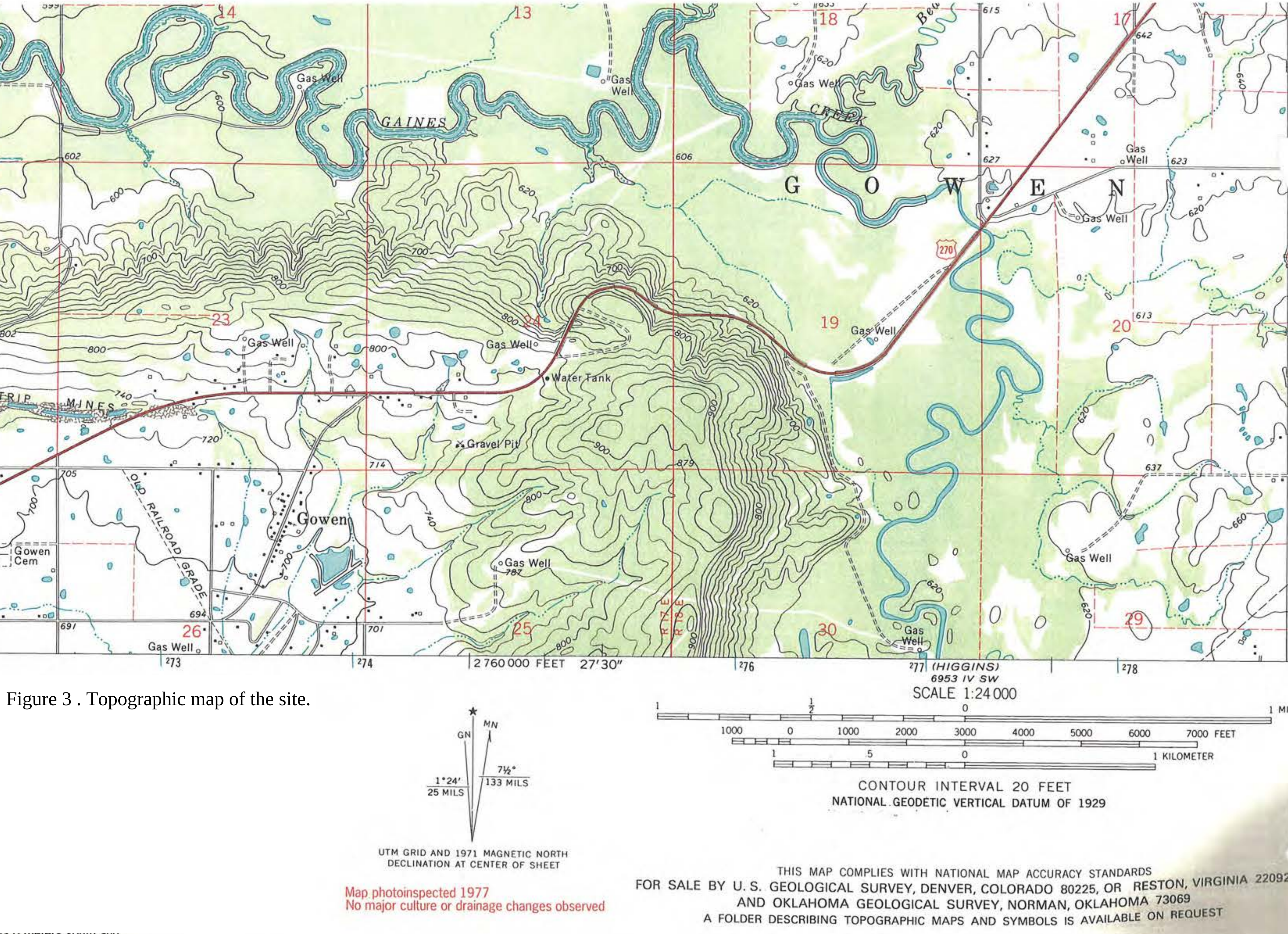
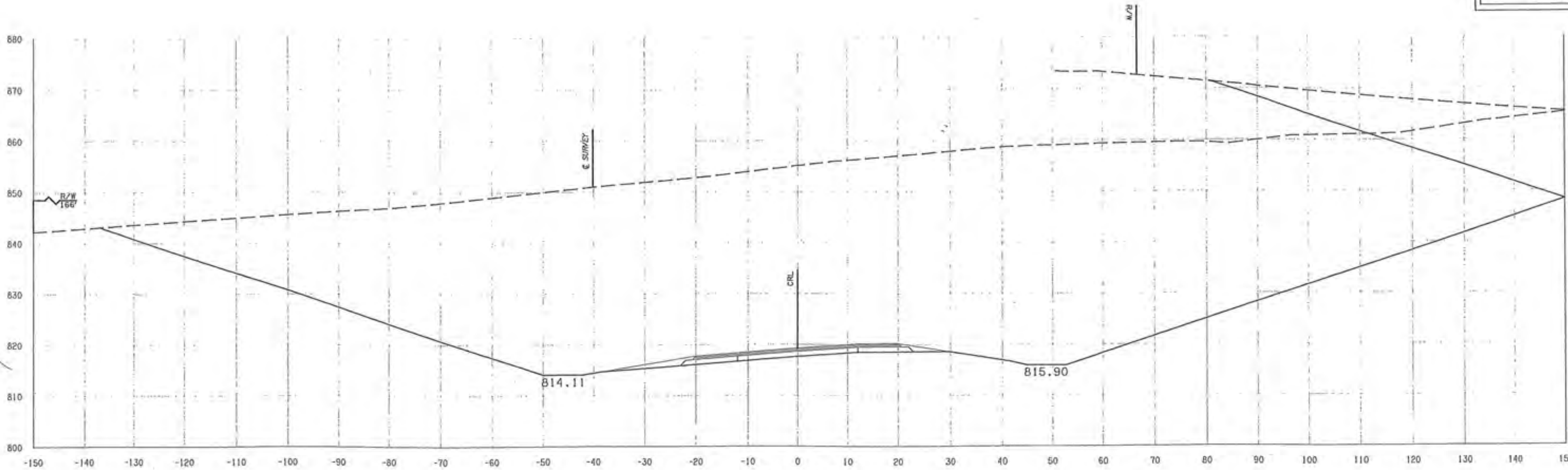
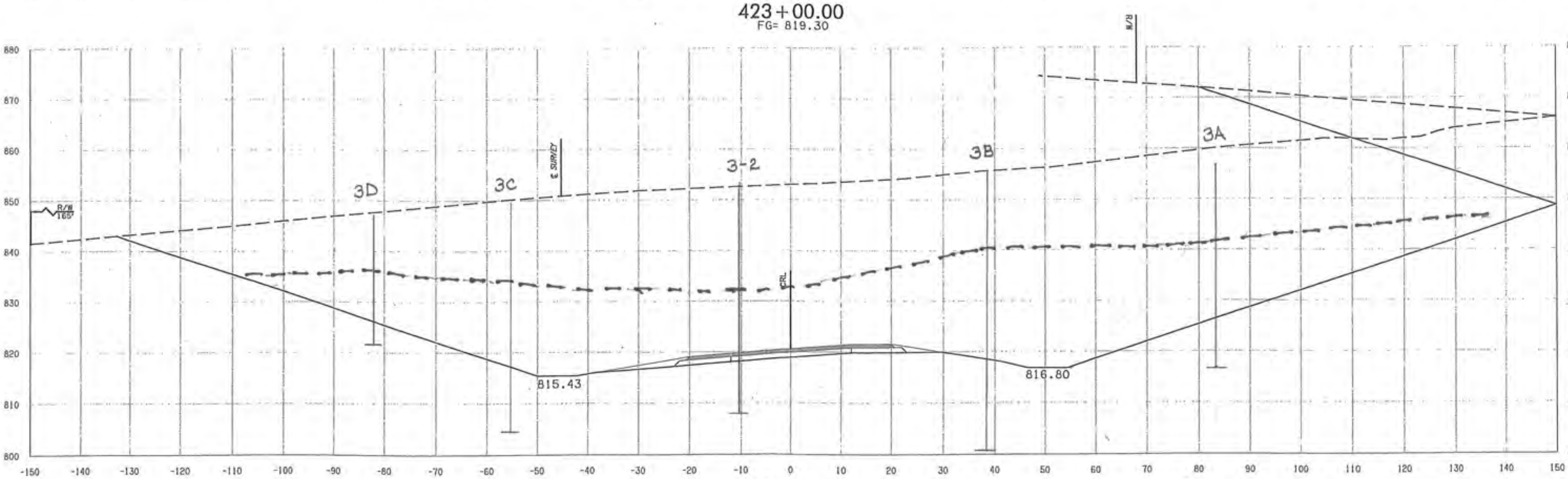


Figure 3 . Topographic map of the site.

OKLAHOMA DEPARTMENT OF TRANSPORTATION
PRELIMINARY
PLANS
03-JAN-2011



423+00.00
FG= 819.30

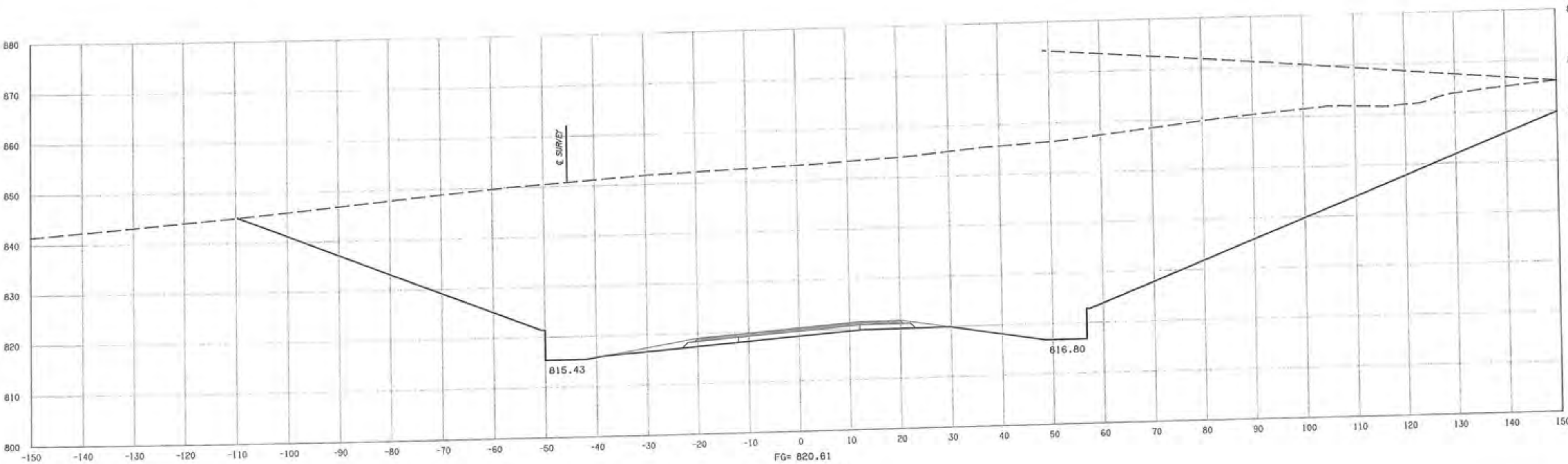


422+00.00
FG= 820.61

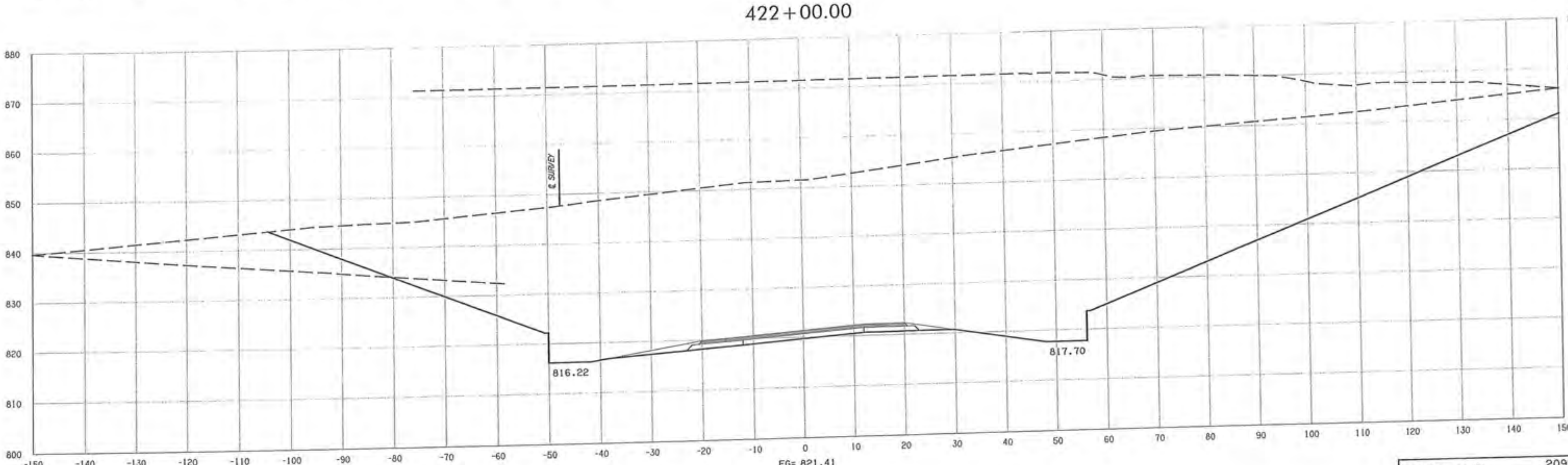
RDYPC51 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 9:52:15 AM 1/3/2011

State Job No.	20922(04)
LATIMER County	Sheet No. 15X

OKLAHOMA DEPARTMENT OF TRANSPORTATION
PRELIMINARY PLANS
13-MAY-2011



422+00.00

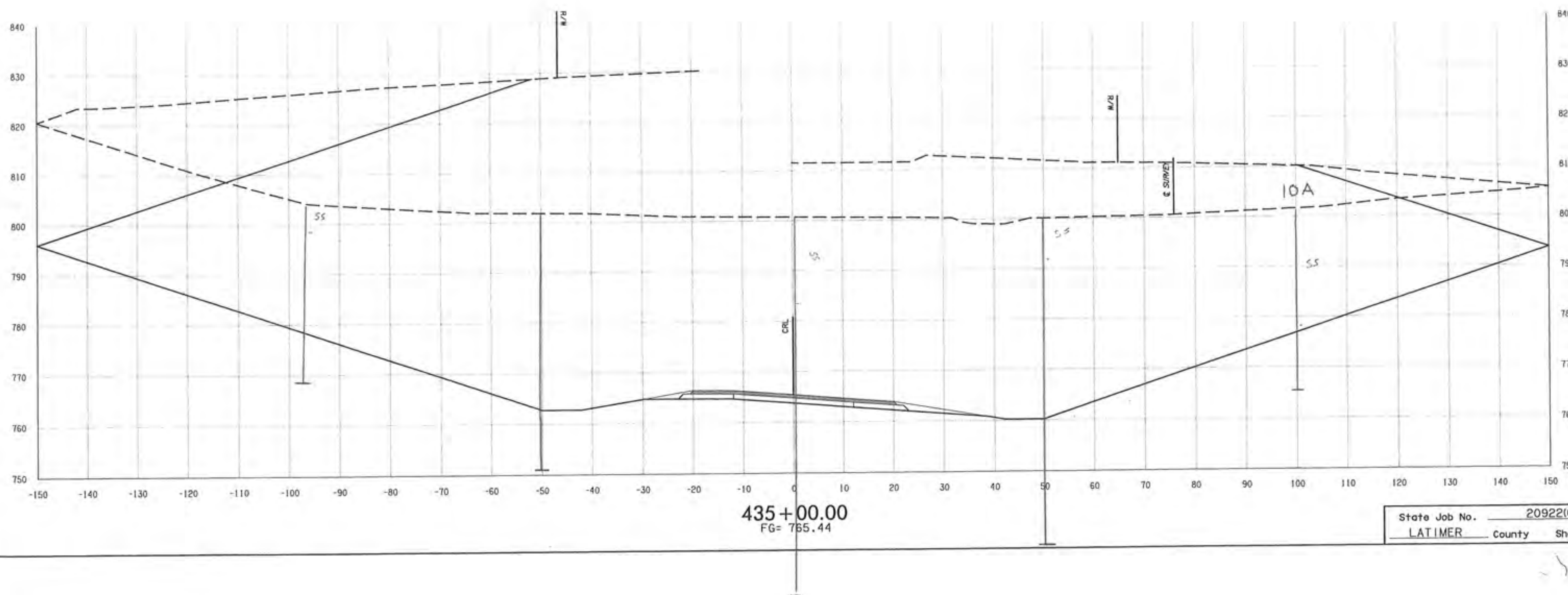


421+00.00

State Job No. 20922(04)
LATIMER County Sheet No. 16?

RDYPC72 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 2:46:08 PM 5/13/2011

OKLAHOMA DEPARTMENT OF TRANSPORTATION
PRELIMINARY PLANS
03-JAN-2011



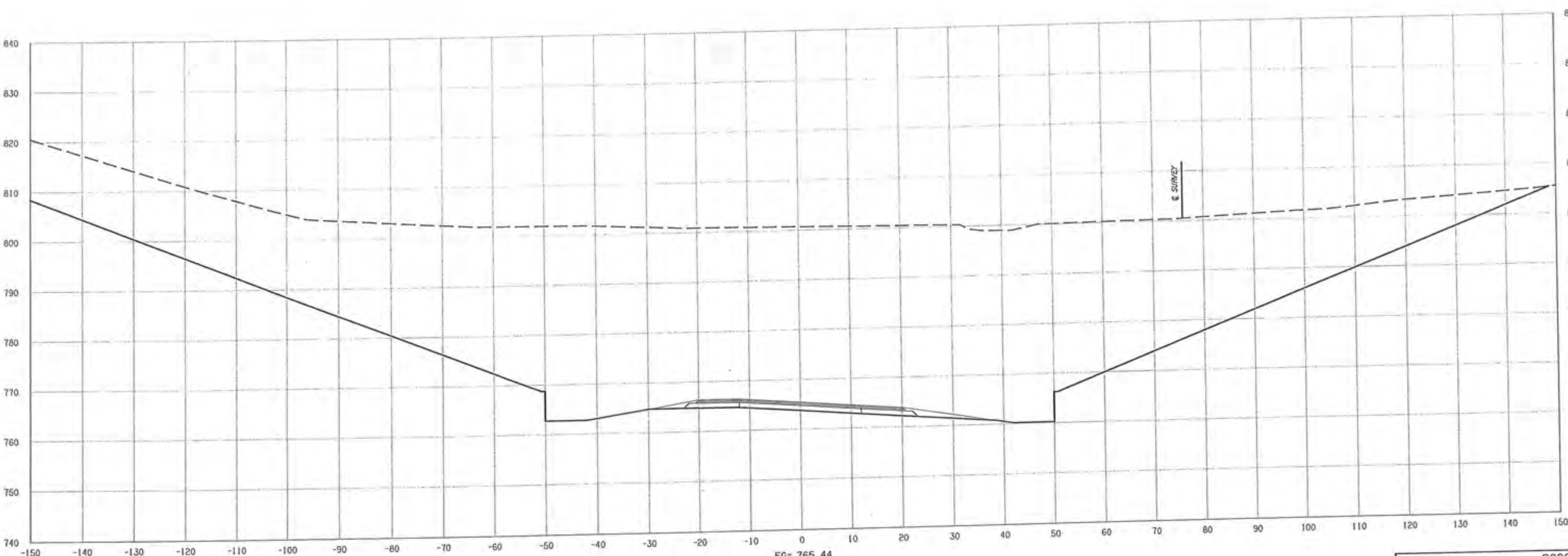
RDYPC51 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 9:52:27 AM 1/3/2011

State Job No. 20922(04)
LATIMER County Sheet No. 23X

OKLAHOMA DEPARTMENT OF TRANSPORTATION

PRELIMINARY PLANS

13-MAY-2011



FG= 765.44

435+00.00

State Job No. 20922(04)	
LATIMER County	Sheet No. 27X

RDYPC72 Y:\Division 2\20922(04) Latimer\P&P Sheets\Working x-sections.dgn 2:46:15 PM 5/13/2011

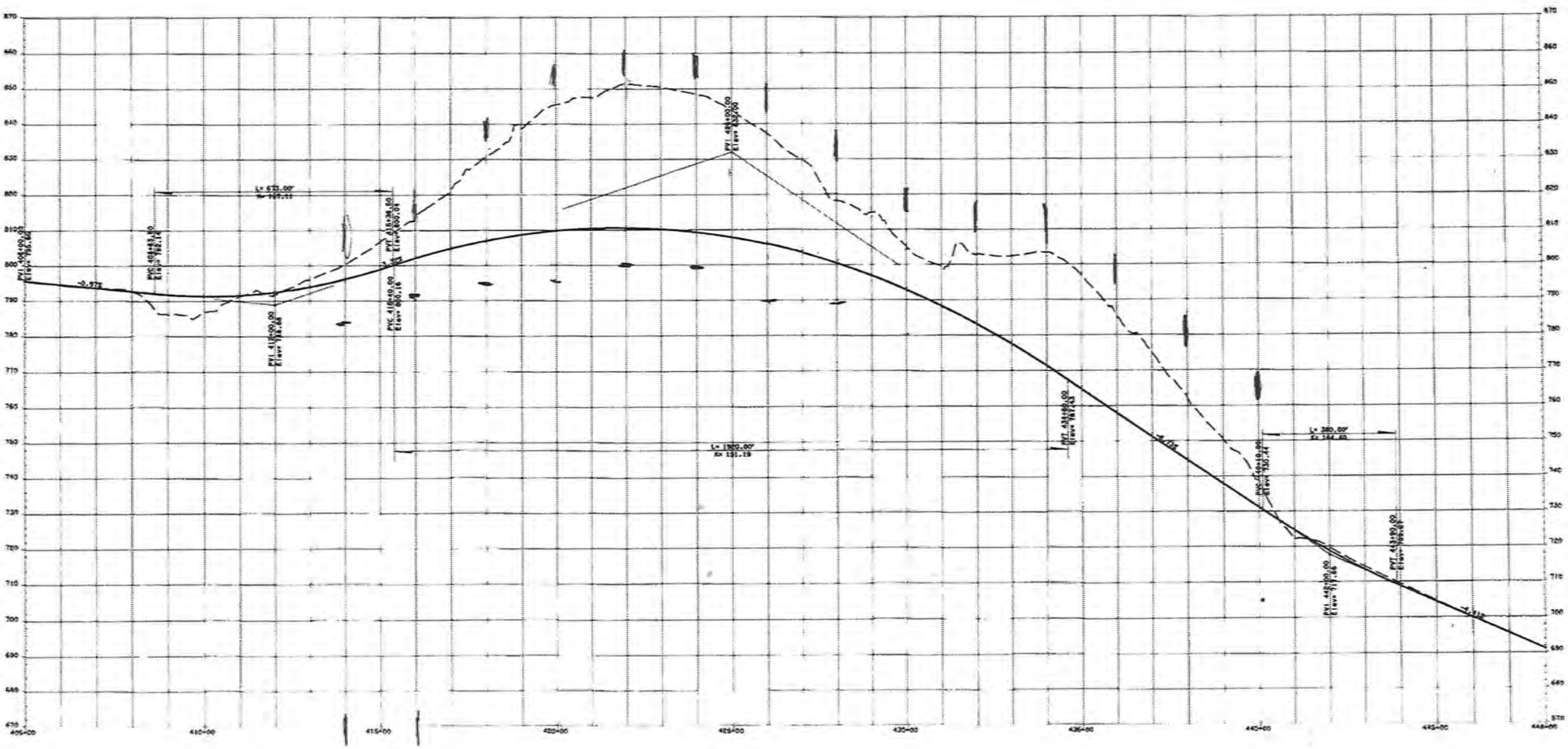


Figure 1. Site Location

Karst Geohazards

Along Highways

Identification and Mitigation

By

Harry Moore

Golder Associates

3730 Chamblee Tucker Rd.

Atlanta, GA 30341

865-603-7652

HLMoore@golder.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The author wishes to thank the Tennessee Department of Transportation, Geotechnical Engineering Section for their help with the historical information and details regarding project information presented in this paper. In addition, the author expresses his sincere thanks to Tony Sak, Deana Sneyd, Abbie Faust, and Courtney Tisius with Golder Associates in Atlanta for their help with the geotechnical information regarding the I-26 Welcome Center project in Sullivan County, Tennessee.

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s)

ABSTRACT

Geohazards in general continue to cause multimillion dollar damages to not only infrastructure but to private businesses, homes, and can result in loss of life. Earthquakes, flooding, tsunamis, and sinkhole collapse tend to be the major geohazard issues to be confronted.

Karst geohazards that include sinkhole collapse, sinkhole flooding, groundwater pollution, and sensitive environmental populations associated with karst are the major areas of concern when planning, locating, designing, constructing, and maintaining highway systems in karst areas. The collapse of sinkholes along roadways and highway structures continues to be a major safety issue resulting from karst. Encroachment onto land containing geohazards is becoming more common as the geologically better ground is fast being used-up. Approaching karst geohazards should include concepts such as avoidance, minimizing impacts and implementing mitigation measures. Examples of each concept are discussed and illustrated.

INTRODUCTION

The dramatic scenes of sinkhole collapses with roads, bridges or even houses crumpled in the throat of the collapse is always sensational and easily grabs ones attention. Repairing such calamities can, however, be a daunting task.

Karst geohazards typically include sinkholes, caves, sinking streams, groundwater issues, microenvironments and wildlife associated with sinkholes and caves. In addition, subsidence and collapse of sinkholes and cave passages and sinkhole flooding are also included with karst geohazards.

A karst landscape can be composed of gently rolling hills and valleys textured with sinkholes, cave entrances (Figure 1), sinking streams and outcroppings of weathered limestone as is typically found in Kentucky and Tennessee and the Virginia/West Virginia/ Pennsylvania sections of karst. Dramatic topography that exhibits tall “haystack” type karst terrane as found in the Li River section of southwest China (Figure 2) can be impressive and a daunting landscape for highway construction.

Existing roads in karst areas often experience the sudden collapse of a sinkhole (Figure 3), perhaps damaging a vehicle or even causing bodily injury. The flooding of a sinkhole basin crossed by a road is a scene often found in karst areas of the Appalachian region (Figure 4). In addition, issues of groundwater pollution often go unchecked or addressed by highway agencies as well as developers and city planners. Today, more recognition of the sensitive karst (cave) environment requires more detailed attention from specialists than ever before.

To the geologist and geotechnical engineer, caves are more than just curious features of the natural world because they reveal information about the groundwater movement and solution development processes that act on carbonate rocks. The patterns in which the solution cavities and resulting caves develop generally indicate the attitude of the bedrock, fracture density, and groundwater movement. Cave passage patterns, sinkholes and springs are important features that may contribute to an understanding of the local groundwater flow and where sinkholes may form. The recognition of areas of active karst subsidence and collapse is of considerable importance to those engaged in the design and implementation of highways, especially the construction of infrastructure.

AVOIDANCE

In most situations, avoiding the karst area or feature is the preferred action. This reduces impacts on the road facility, the karst environment, and the pocketbook. The key in avoidance is identifying the karst area or feature during the planning phase of highway development. The production of karst maps, sinkhole maps, cave maps, and sensitive karst area maps can all aid in the avoidance of karst.

One of the most usable types of map is the “area of karst” map. These types of maps usually identify broad areas of karst which include existing sinkholes and areas of potential sinkhole development and may cover square miles of area. Broad bands or large areas of karst can be easily identified and displayed in this type format for the planner or developer use so as to avoid problematic areas. However, this is often done after the planners have made the critical decisions and the geotechnical investigations have been initiated.

These types of karst maps can show general patterns of karst development, such as Valley and Ridge Province karst or Plateau or Highland Rim geology. Figures 5 and 6 show examples of general karst maps that have been developed by the Tennessee DOT, Geotechnical Engineering Section.

The geohazard areas can be expressed as outlined patterns on topographic maps to better illustrate the geohazard relative to the surrounding landscape. In addition, the proposed corridor route is overlain on the geohazard map. This map is then used by the roadway planners to better locate the final roadway centerline.

In some cases, maps with contour intervals as small as one to three feet are available which greatly enhances the sinkhole identification (Moore and McDowell, 2008). David A. Hubbard (2003) researched this issue in the karst regions of the Valley and Ridge of Virginia where he consistently identified more sinkholes on the ground than were depicted on selected 7.5 minute quadrangle maps. In one instance Hubbard describes a 7.5 minute quadrangle with a 6 meter (20 foot) contour interval that revealed 55 sinkhole features, while ground field mapping identified 533 sinkhole features.

In another case that relates this concept to East Tennessee, Moore (2004) studied a proposed location of an interchange along I-181 in Sullivan County, Tennessee situated in the Valley and Ridge province and underlain by carbonates. Within an approximately 100 acre site, the standard 7.5 minute maps with 20 foot contour intervals revealed some 30 or so sinkhole features. After aerial mapping using a 1 meter contour interval, a total of 177 sinkholes and 7 cave entrances were identified at the proposed site (Figures 7 and 8), a site that was later rejected by the Tennessee Department of Transportation for the interchange location.

In addition to the surface mapping of sinkholes, it is becoming more important to map the caves where they may exist in close proximity to the proposed roadway. By knowing spatially where the cave passages are located, a more accurate design of proposed roadway cut slopes can be made. This would prevent the unnecessary opening of a cave system to the surface, and benefit the cave biota, such as bat colonies and salamanders to name a few.

In 2008 the Tennessee Department of Transportation (TDOT) was in the process of planning a new roadway alignment (SR 71) that crossed portions of South Knox County in East Tennessee. The proposed corridor was located in a section of the Valley and Ridge Province of East Tennessee where several ridges and valleys will be crossed, as well as creeks, roads, subdivisions and rural lands. The proposed corridor is planned to connect the current terminus of SR 71 at Moody Ave. in South Knoxville to John Sevier Highway (SR 168).

In an attempt to properly evaluate the potential for geologic hazards along the project, an effort was made to locate all sinkholes and caves within the project area, karst being the primary geologic hazard identified (Moore and McDowell, 2009). To this effort, a karst map was completed (Figure 9).

In some instances, it is important to identify cave entrances and cave passages in order to avoid these features. As a result, the planners or designers can see the cave map overlay on the proposed roadway plans and make the appropriate changes as needed (Figure 10). Detailed cave maps can be obtained from local caving organizations or groups, or can be developed by DOT survey offices or Geotechnical office personnel.

MINIMIZATION OF IMPACT

It is often the case that decisions have already been made that impact karst areas and the geotechnical engineer or geologist has to adapt the proposed roadway plan to reflect the karst issue. In these situations, minimizing the impact of the proposed roadway design is the course of action needed. Avoidance is usually not an option at this stage of the roadway design.

Minimizing the roadway's impact on the karst environment is a proactive approach that will lessen future consequences of building in karst terrain. Such actions as reducing cut and fill sections to a minimum (often requiring grade changes during design), using graded rock embankments in sinkhole areas (as opposed to using soil or common excavation), adjusting roadway alignments to minimize encroachments into sinkhole basins and caves, and avoiding the discharge of highway runoff into karst basins and sinkhole swallets and cave entrances will enable a proactive design that is constructible. The construction of bridges across a sinkhole and/or sinkhole terrain can greatly reduce the impact of a roadway project. Although costly, the bridging concept is very effect in reducing flooding and the impact on the karst environment.

One of the more widely used concepts for constructing roadways across karst terrain and minimizes the roadway's impact on the karst area is the use of graded rock pads and embankments (Figure 11). The graded nature of the shot rock material removes most of the fines and allows the larger rock pieces to have interlock with each other providing stability for the embankment. When used in thin soil areas where bedrock is exposed at the surface the rock pads and embankments can serve as a "bridging" element over the karst feature.

In 2010 the Tennessee Department of Transportation contracted with Golder Associates to investigate a proposed site for a Welcome Center along I-26 in Sullivan County, just south of Kingsport and north of the I-26 and I-81 interchange. During this study, which consisted of field mapping, numerous soil and rock borings, sample testing and design recommendations, a number of sinkholes were found along the proposed study area (Sak, Sneyd, and others, 2010).

As a result of the geotechnical investigation by Golder Associates, several proactive design concepts were proposed to minimize the impact of the welcome center on the local karst environment. These concepts included the use of graded rock embankments, geomembrane-lined drainage ditches, and the use of filtration systems for the roadway run off.

This discussion does not address the problem with soil voids developed in the residual soil over cavitose bedrock. These soil voids will typically collapse when the soil arch looses sufficient thickness to arch the open space in the soil. Drumm and Yang (2005) discusses the arching ability of soils and the residual soil stability in karst terrain.

In addition, the graded rock pads and embankments allow surface water to continue to flow into the existing sinkhole area thereby continuing to recharge the local groundwater regime. The graded rock also serves as a filter for larger debris such as trash, tree limbs and leaves.

The graded rock specification used in Tennessee by the Tennessee Department of Transportation is as follow:

Graded Solid Rock shall consist of sound, non-degradable rock with a maximum size of 1 meter (3.3 feet). At least 50 percent of the rock shall be uniformly distributed between 300 millimeters (1 foot) and 1 meter (3.3 feet) in diameter and no greater than 10 percent shall be less than 50 millimeters (2 inches) in diameter. The material shall be roughly equi-dimensional in shape. Thin "slabby" material will not be accepted.

The contractor shall be required to process the material with an acceptable mechanical screening process that produces the required gradation. When the material is subjected to five (5) alternations

of the sodium sulfate soundness test (AASHTO T 104), the weighted percentage of loss shall be not more than 12. The material shall be approved by the Engineer before use.

Another design concept that reduces impact on karst terrain is to provide a lining for the roadway ditchlines (Figure 12). Karst problems along Tennessee highways have previously been described by Royster (1984), and Moore (1980, 1984, 1987, and 2003). Moore's study (1987) involved the analysis of 72 karst related subsidence and collapse problems experienced along highways in East Tennessee over a ten year period (1976-1986). The data collected in the study indicated that of the 72 sinkholes researched, 85% were "induced", while 15% were considered "natural". The most important result of the study was the revelation that 74% of the karst problems occurred in roadway ditchlines. The remaining 26% occurred in roadway subgrades and in areas unrelated to highway facilities (fields, yards, woods) (11% and 15% respectively).

The majority (93%) of the ditchline problems studied occurred along untreated roadway ditches. Untreated ditches are defined in this study as being standard roadway drainage ditches which are constructed without the benefit of pavement or other impervious materials.

In 2003 Moore updated the 1987 study by analyzing 163 cases of sinkhole collapse incidents in east Tennessee between 1969 and 2002. Of the 163 sinkhole incidents studied, 86.5% of the sinkhole occurrences were located in highway ditch lines (Moore, 2003). The 2003 study also supported the findings of the 1987 study by showing that of the ditch line collapse incidents analyzed, 93% also involved unlined ditches (the same amount disclosed in the 1987 study).

The most efficient method of "lining" a roadway ditchline is using a geomembrane (not a geotextile which is permeable). Construction practice has shown that 40 to 60 mil HDPE geomembranes tend to survive the best during construction activity. After placement of the geomembrane the ditch can then be covered with soil and sod, rip-rap or even concrete paving. It is important to attend to the details of overlapping the geomembrane during placement, and making sure that the sides of the geomembrane are "feathered" into the adjacent soil medium to prevent run off from flowing down the sides of the geomembrane and undermining the ditch.

MITIGATION MEASURES

When avoidance of karst is not possible and minimizing the roadway impact has been achieved, then it is most likely that some measure of mitigation will be required. Using graded rock fills and lined ditches are ancillary to other mitigation techniques. Primarily, the use of filtration systems for highway runoff in karst areas is a rather new and innovative concept.

The use of sinkholes for drainage is often considered in roadway design. In a proactive methodology for environmentally sensitive roadway design in karst, the best approach would be to avoid placing drainage into a sinkhole, particularly using the sinkhole as a wastewater disposal feature. The use of sinkholes as drainage features is basically groundwater contamination by design. Eventually, hazardous waste will be spilled and the karst aquifer will become contaminated.

Concerns about groundwater contamination must be addressed by either a filtration system, retention/sediment basin or both. A study by Stephenson and others (1997) showed that using a pilot researched peat-moss filtration system on highway runoff on the I-40/I-640 interchange in Knoxville, Tennessee (Figure 13) effectively reduced the highway runoff contaminants by 90 to 99 % (Beck and others, 1996). This type of treatment of highway runoff is highly recommended for high volume highways where the roadway runoff recharges karst aquifers via sinkhole features.

This type of filtration has been effectively employed by the Tennessee Department of Transportation (TDOT) on a demonstration project as described by Beck and others, 1996 in east Knoxville at the intersection of I-40 and I-640. Constructed by TDOT maintenance forces in the late 1990's the filtration system has been in operation over ten years, successfully filtering first-flush runoff from the I-40/I-640 interchange.

Another "sinkhole filter" was constructed by TDOT in Morristown in 2008 along a section of Jarnigan Road (Figure 14) in an effort to mitigate any future runoff pollution from the roadway (Sutton, 2005). The filter was constructed in the format of the researched demonstration filter constructed in Knoxville at the I-40/I-640 interchange in the late 1990's. To date, the filter is operating as designed and appears to be a successful filtration project.

Another mitigative and proactive approach to karst is related to drainage and requires attention to several design related items, some of which have been discussed above. These include lined ditches, rock pads, overflow channels, sinkhole opening improvement/protection, curbs for embankment sections, and drainage wells. The following summarizes the treatment of karst related drainage problems used to mitigate the impact of roadway construction on the karst environment (Moore, 2004):

- **<u>Lined ditches</u> - The single most important item that can be implemented to prevent future sinkhole collapse occurrence is the use of lined drainage ditches. Types of liners that tend to function the best include 60 mil PVC and/or HDPE geomembrane, and concrete and asphalt materials.**
- **<u>Rock Pads</u> - Rock pads beneath embankments using clean rip rap limestone may be used for bridging depressions and sinkholes.**
- **<u>Overflow Channels</u> - This concept involves the construction of a lined channel or pipe from a negative drainage area (sinkhole) to a positive draining system.**
- **<u>Sinkhole Opening Improvement/Protection</u> - This concept involves improving the runoff flowing into subsurface cavities by removing debris and trees from around the**

throat of a sinkhole and protection of the cavity opening from siltation and debris using such methods as siltation barriers, debris catchment fences, rip-rap, gabion siltation barriers, and concrete structures.

- **Drainage Wells - An additional concept that has been implemented in some areas is the use of injection drainage wells (Crawford and Groves 1984, 1995; Reeder and Crawford, 1989; Royster, 1984; and Moore, 1981). These types of storm water drainage wells are required to be permitted by the State and are known as Class V injection wells.**

When implementing any of the above drainage concepts, it is imperative that a maintenance program be established and monitored in order to reduce future problems.

SUMMARY

The collapse of highway surfaces, drainage ditches, and bridge foundations, as well as numerous instances of flooding are karst related problems triggered by human construction activity. The greatest number of karst problems that develop along highways in East Tennessee involves subsidence and collapse of the drainage ditch.

Avoidance measures and some combination of drainage and bridging methods used to minimize the roadway project’s impact on the karst environment are usually the best direction to take in a proactive approach to designing and constructing highways in karst areas. Minimizing the impact on karst is best achieved by using graded solid rock fills (embankments) instead of common fill material, bridging over sinkholes and caves, and using geomembrane-lined ditches.

Mitigating impacts on sensitive karst areas is often the result of constructing roads in karst terrain. The most common mitigation measure (in addition to the above mentioned avoidance and minimizing concepts) is using solid rock backfill for collapse incidents and filtration systems to improve highway runoff that drains into adjacent sinkholes or cave systems. An often overlooked issue is the impact of road construction on the very fragile and sensitive environments found in large “Ice-Age” developed ecosystems located in sinkholes and cave entrances. Once destroyed, these sinkhole and cave environments, which often harbor rare plants and animals, cannot be replaced.

Innovative and cost effective remedial mitigative concepts for solving karst related geotechnical problems require modifications and refinement of the standard design to insure proper results to site-specific conditions. Proactive involvement by the geologic and engineering profession is necessary to insure the success of karst related remedial design concepts proposed for highways.

REFERENCES

Beck, B.F., Stephenson, J.B., Wanfang, Z., Smoot, J.L., and Turpin, A.M., 1996. Design and evaluation of a cost effective method to improve the water quality of highway runoff prior to discharge into sinkholes: in Proceedings of the 1996 Florida Environmental Expo, Tampa, Florida, October 1-3, p.155-164.

Crawford, N.C., and Groves, C.G., 1984. Storm water drainage wells in the karst areas of Kentucky and Tennessee. U.S. Environmental Protection Agency and Center For Cave and Karst Studies, Bowling Green, Kentucky, 52 p.

Crawford, N.C. and Groves, C.G., 1995. Sinkhole collapse and groundwater contamination problems resulting from storm water drainage wells on karst terrain. In Karst Geohazards, Proceedings of the 5th Multidisc. Conf. On Sinkholes, ed. B.Beck, Gatlinburg, Tenn., pp. 257-264.

Drumm, Eric C., and Yang, Michael Z., 2005. Preliminary Screening of Residual soil stability in karst terrain. Environ. And Engineering Geoscience, Vol. XI, No. 1, Feb., pp.29-42.

Hubbard, David A., 2003. Use of regional sinkhole mapping for sinkhole susceptibility maps. ASCE Geotech. Special Publ. No. 122: Sinkholes and the Engineering and Environmental Impacts of Karst, proceedings of the 9th Multidisciplinary Conference on Sinkholes, ed. Barry Beck, Huntsville, Ala., Sept. 6-10,p. 61-71.

Moore, H.L., 1980. Karst problems along Tennessee highways: an overview. In Proceedings of the 31st Annual Highway Geology Symposium, Austin, Texas, August, pp 1-28.

Moore, H.L., 1984. Geotechnical considerations in the location, design, and construction of highways in karst terrain - ‘The Pellissippi Parkway Extension’, Knox-Blount Counties, Tennessee. In Sinkholes: Their Geology, Engineering, and Environmental Impact, Proceedings of the 1st Multidisc. Conf. On Sinkholes, Orlando, Fla., pp 385-389.

Moore, H.L., 1987. Sinkhole development along ‘untreated’ highway ditchlines in East Tennessee. InKarst Hydrogeology: Engineering and Environmental Applications, Proceedings of the 2nd Multidisc. Conf. On Sinkholes, Orlando, Fla., pp 115-119.

Moore, H.L., 2003. Recent sinkhole occurrences along highways in East Tennessee, A historical perspective. Proceedings of the 54th Annual Highway Geology Symposium, Burlington, Vermont, Sept. 24-26, pp 46-55.

Moore, H.L., 2004. Preliminary geotechnical investigation of proposed interchange, I-181 with ext of SR 126, Sullivan, Co., unpublished report files, Geotechnical Engineering Section, TDOT Region One, Knoxville Office.

Moore, Harry L., and Lori McDowell, 2008. The Development and Use of Karst Maps in the Location of Highways in East Tennessee. In Beck, B. F., ed., Sinkholes and the Engineering and the Environmental Impacts of Karst, Proceedings of the 11th Multidisciplinary Conference on Sinkholes, September 22-26, 2008, Tallahassee, Fla., ASCE Geotechnical Special Publication No. 183, P. 680-693.

Moore, Harry L., and Lori McDowell, 2009. Mapping Surface and Subsurface Karst:SR71 South Knoxville Blvd., Knox County, Tennessee. In the Proceedings of the 41st STGEC meeting in Wilmington, NC, November 2-5,2009, paper No. 1056.

Reeder, P.P., and Crawford, N.C., 1989. Potential groundwater contamination of an urban karst aquifer - Bowling Green, Kentucky. In Beck, B.F., ed., Engineering and Environmental Impacts of Sinkholes and Karst , Proceedings of the 3rd Mjultidisc. Conf. On Sinkholes and the Engineeringand Environmental Impacts of Karst, Orlando, Fla. October 2-4, p. 197-206.

Royster, D.L., 1984. Use of sinkholes for drainage. In Construction and Difficult Geology: Karstic limestone, permafrost, wetlands, and peat deposits; Trans. Research Board Record 978, Wash..D.C., pp. 18-25.

Sak, Tony, Deana Sneyd, Abbie Faust, Courtney Tisus, and Harry Moore, 2010. Draft Geotechnical Exploration and Recommendations Report: I-26 Welcome Center and Interchange, Sullivan County, Project NO. 82084-1129-44, PIN NO. 040028.01. Tenn. DOT Geotechnical Engineering Section Project Files.

Stephenson, J. B., W.F. Zhou, B. Beck, and T. Green, 1997. Management of highway storm water runoff in karst areas – baseline monitoring and design of a treatment system for a sinkhole at the I-40/I-640 interchange in eastern Knoxville, Tennessee. In Proceedings of the 48th Annual Highway Geology Symposium, Knoxville, Tenn., May 8-10, p. 24-34.

Sutton, George, 2005. Addendum to Geologic Report Concerning Sinkhole Storm-water Runoff Treatment System Design. Tenn. Dept. Transportation, Geotechnical Engineering Section File No. 3208503, Jan. 6, 2005.

Figures

Figure 1. Karst landscapes often include cave entrances and rock outcroppings as well as sinkhole terrain.

Figure 2. Karst landscapes in China are often very dramatic as seen in this photo along the Li River near Gulin, China.

Figure 3. The sudden collapse of sinkholes cause the most dangerous and dramatic problems along roads in karst areas.

Figure 4. This flooded road in Knoxville, Tennessee is the result of sinkhole flooding. The sinkhole throat is along the left side of the road. A flooded out car is barely visible.

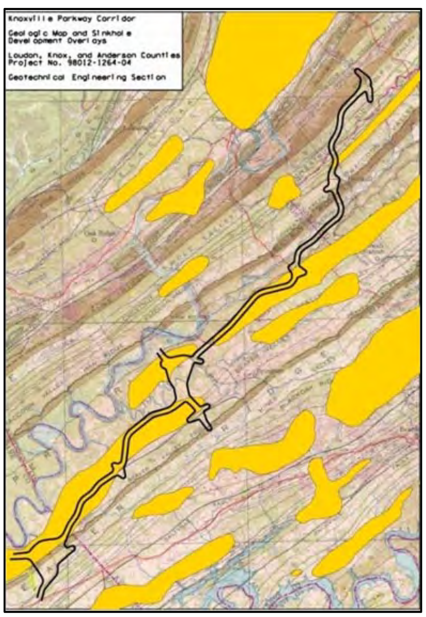


Figure 5. This karst map of an area just west of Knoxville, Tennessee, shows areas of sinkhole development that was mapped during a study for a proposed Knoxville by-pass. The pattern shows a parallel to the Valley and Ridge geology and topography.

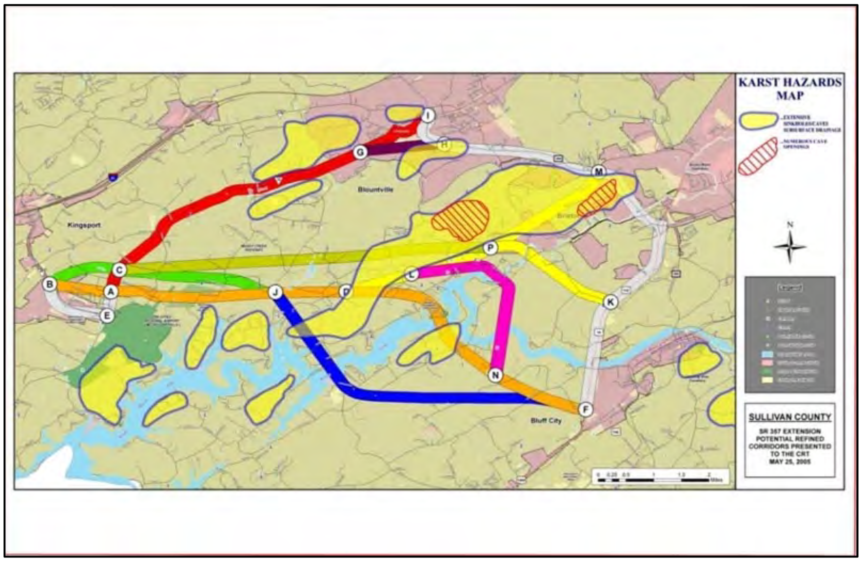


Figure 6. This map shows proposed routes for a road extension in upper East Tennessee. Karst areas are shown as irregular patterns. Maps like these aid planners and road designers in siting new infrastructure routes.

Figure 7. This section of a 7.5 minute topographic map indicates approximately 15 sinkholes at a 20-foot contour interval. This location for a welcome center was located in extreme karst.

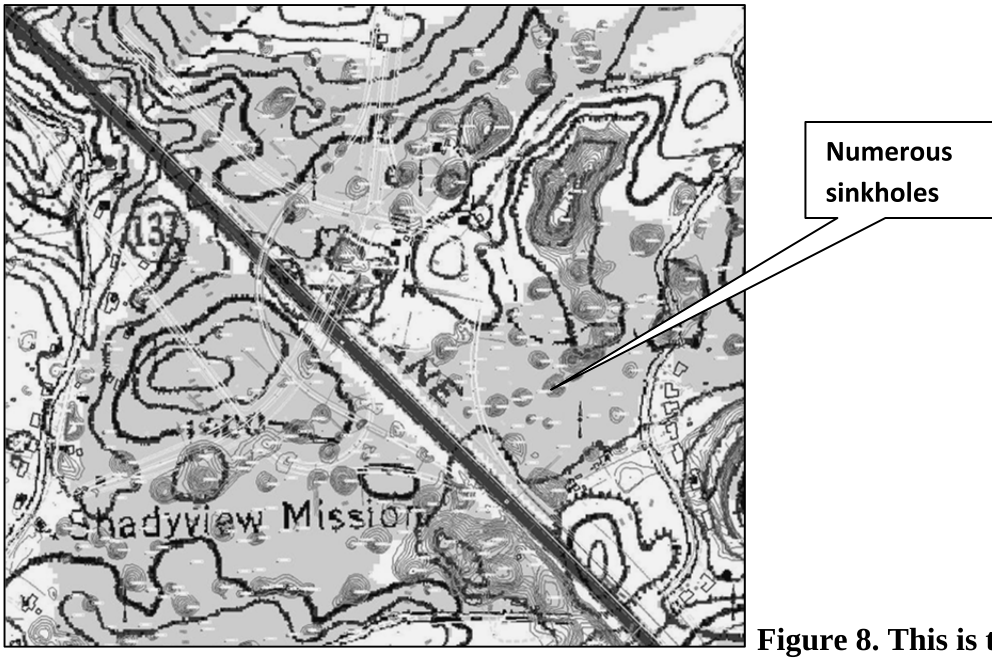


Figure 8. This is the same section of topography as shown in Figure 7, except this map shows a contour interval of 3 feet. As a result, more than 60 sinkholes (shown as darker roundish areas) are indicated instead of the 15 shown in Figure 5.

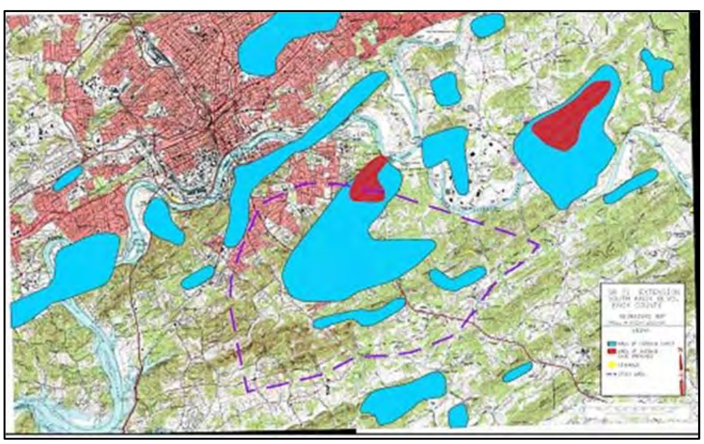


Figure 9. This karst map shows areas of sinkholes and cave entrances (blue or gray patterns) in a study area for a proposed SR 71 route extension in South Knoxville.

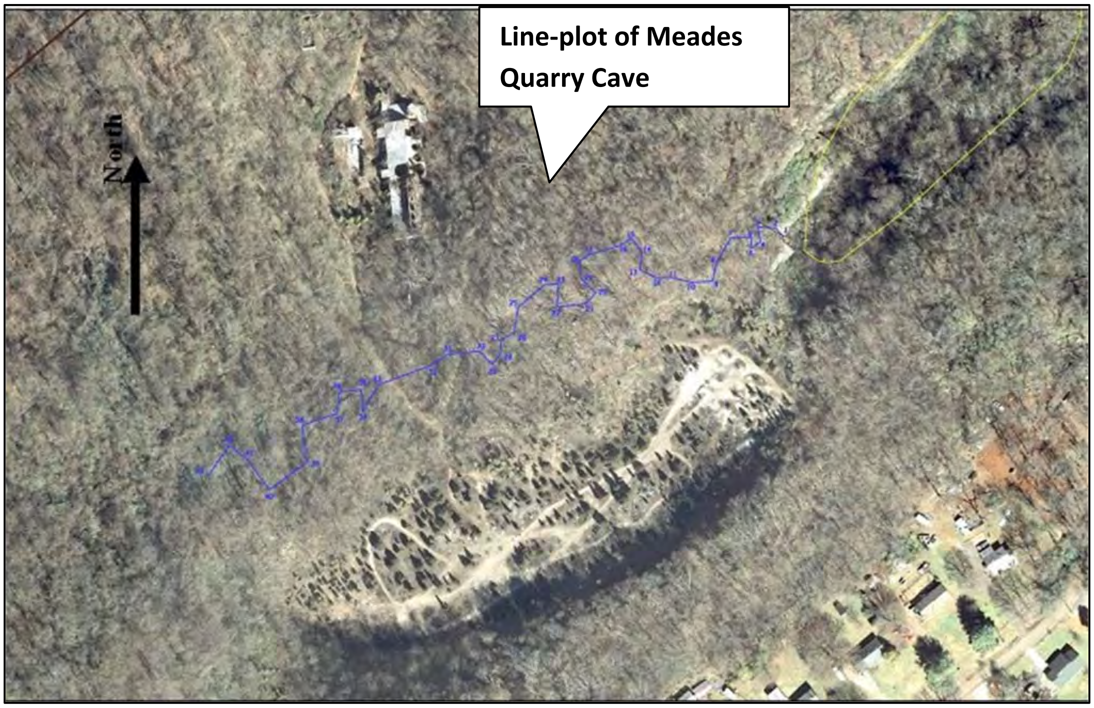


Figure 10. This figure shows an aerial view of a karst area located in the karst map in Figure 9. Plotted on the aerial map is a line-plot of a cave that was mapped during the geotechnical investigation of the project by the Tenn. DOT Geotechnical Engineering Section.

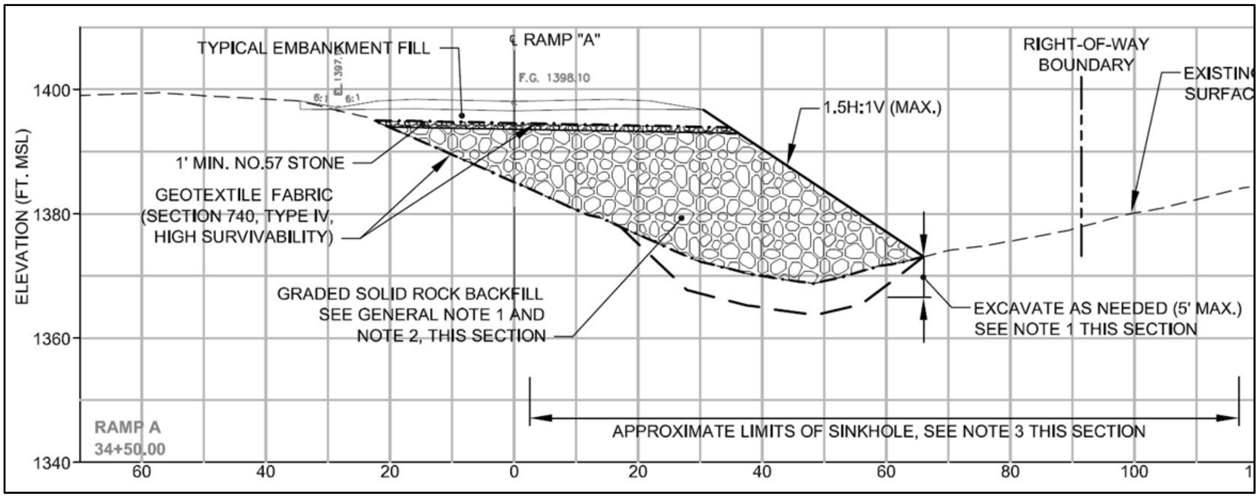


Figure 11. This drawing shows the recommended graded rock embankment for a ramp proposed for the I-26 Welcome Center in Sullivan County, Tennessee (Sak and others, 2010; Tennessee DOT Geotechnical Engineering Section project files).

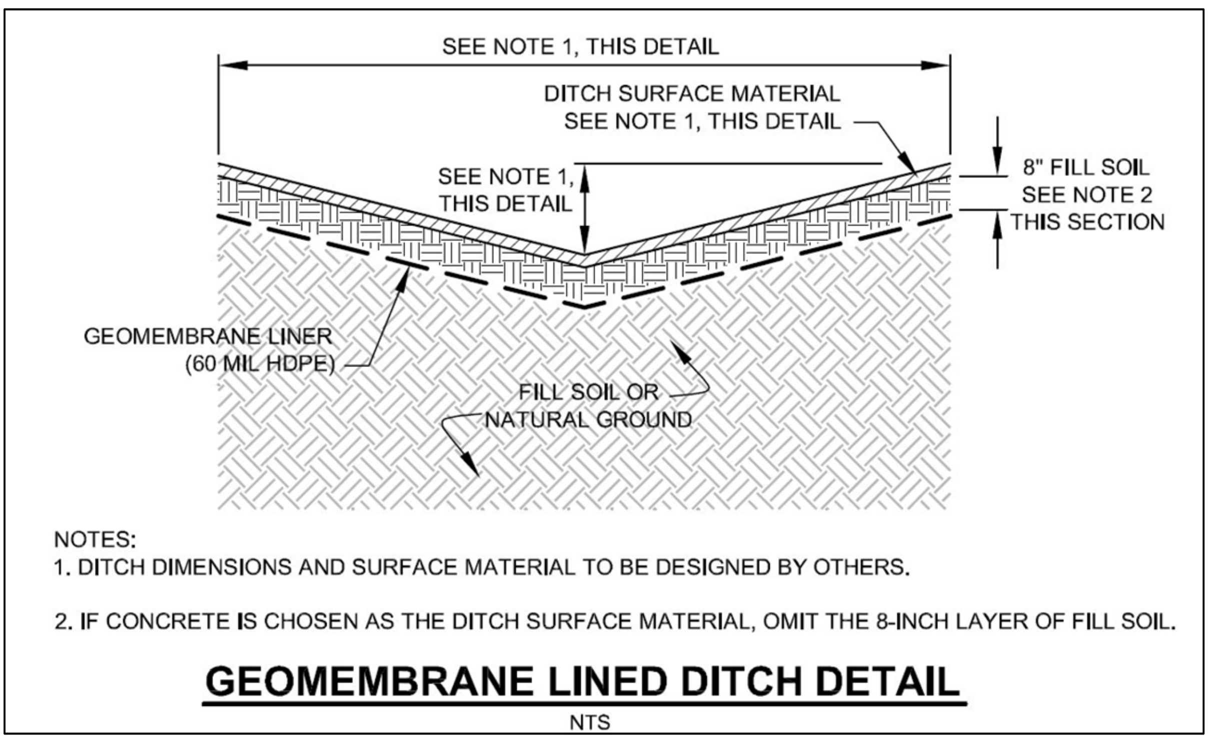


Figure 12. This detail drawing shows the recommended geomembrane lined ditch detail for the proposed I-26 Welcome Center in Sullivan County, Tennessee (Sak and others, 2010; Tennessee DOT Geotechnical Engineering Section files).

Figure 13. This photo shows a completed filtration system implemented at the interchange of I-40 and I-640 in east Knoxville, Tenn. The roadway runoff filters thru this horizontal peat filter before discharging into a sinkhole located just to the left in this photo.

Figure 14. This photo shows the filtration system constructed adjacent to Jarnigan Rd. in Morristown, Tennessee to filter runoff from the roadway. The sinkhole receiving the discharge is located to the extreme right.

Route I-70 & 435 Interchange Mine Remediation, Kansas City, Missouri

Jamie Martens
HNTB Corporation
715 Kirk Drive
Kansas City, MO 64105
(816) 527-2348
jamartens@hntb.com

John Szturo, R.G.
HNTB Corporation
715 Kirk Drive
Kansas City, MO 64105
(816) 527-2275
jszturo@hntb.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

The Missouri Department of Transportation (MoDOT) is planning modifications to the existing Interstate 70 (I-70) and I-435 Interchange located in Kansas City, Missouri. The modifications include the addition of two ramps at US Highway 40. Both ramps include new collector distributor lanes extending along the east and west sides of I-435. The existing roadways and proposed improvements are in the vicinity of existing underground limestone mines present on both sides of I-435. The detailed site investigation included a review of available maps and aerial imagery; exploratory borings to identify the presence of the mine space and to characterize the subsurface materials; laser scanning to create a three-dimensional model of the mine space; and mine and surface visits to map existing site features. Mine structure stability was evaluated with respect to overstressing the limestone pillars, overstressing the tensile capacity of the mine roof beam, bearing capacity failure, and kinematic instability. Both numerical modeling and theoretical calculations indicated areas of unstable to marginally stable mine roof conditions. Based on the results of the mine remediation study, plans and specifications were developed for ground improvement grouting and tunnel improvements. Grouting included placement of low slump grout to form a barrier at the perimeter of the remediation area and infilling with high slump grout. Tunnel improvements consisted of installation of rock bolts, welded-wire fabric, and fiber-reinforced shotcrete. Following completion of the rail tunnel improvements from within the mine space, the ground improvement grouting was conducted from the ground surface. Close monitoring of the grouting activities resulted in over $450,000 of savings in grout volume.

PROJECT DESCRIPTION

The Missouri Department of Transportation (MoDOT) is completing modifications to the existing Interstate 70 (I-70) and I-435 Interchange located in Kansas City, Missouri (Figure 1). This interchange is one of the busiest and most congested interchanges in the Kansas City metropolitan area. The project was a selected 2009 ARRA project with $39 million budgeted for fast-track improvements.

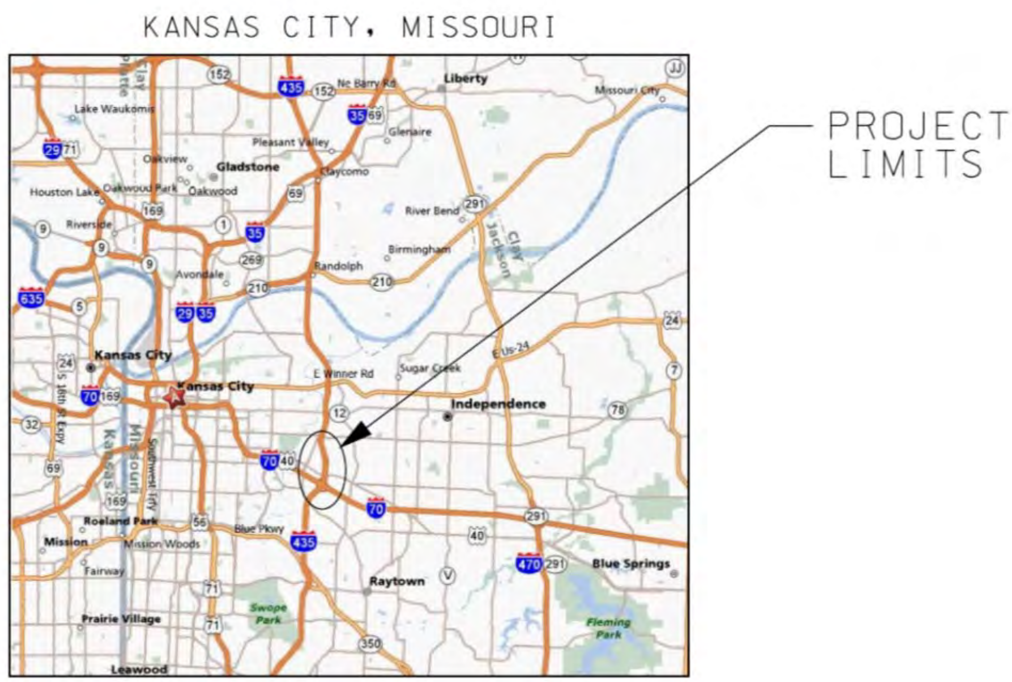


Figure 1 – Project Location Plan

Part of the project includes the addition of two ramps at US Highway 40 (US 40) and I-435 (Figure 2). One ramp will convey traffic from US 40 to Northbound (NB) I-435 at the northeast quadrant of the existing intersection adjacent to Marsh Avenue. This ramp will be included in a new collector distributor lane extending along I-435 northward to approximately 23rd Street. The other ramp will take Southbound (SB) I-435 traffic from a new collector distributor ramp along the west side of I-435, under a widened US 40 overpass, and into a loop to meet US 40 several hundred feet west of the existing overpass.

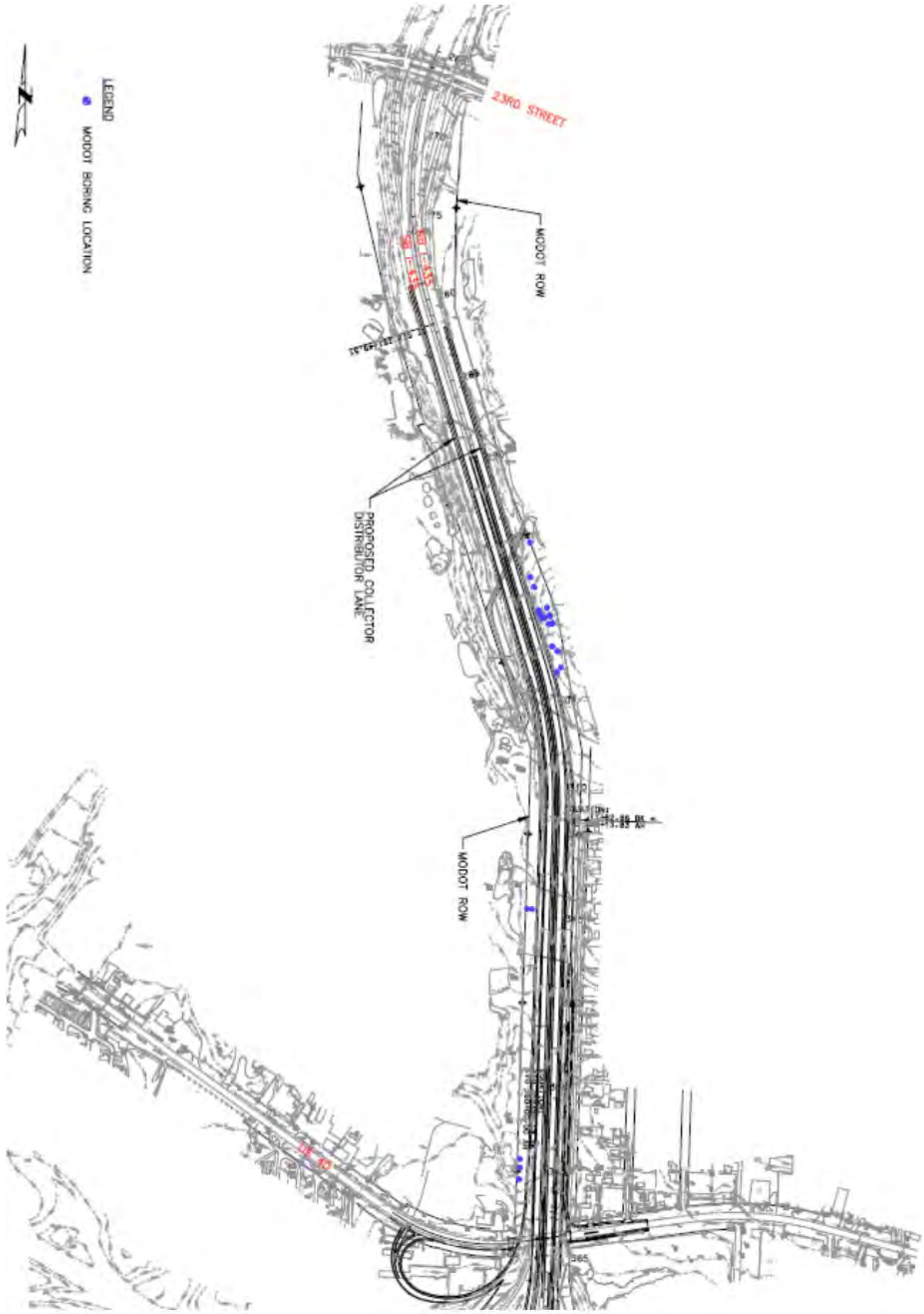


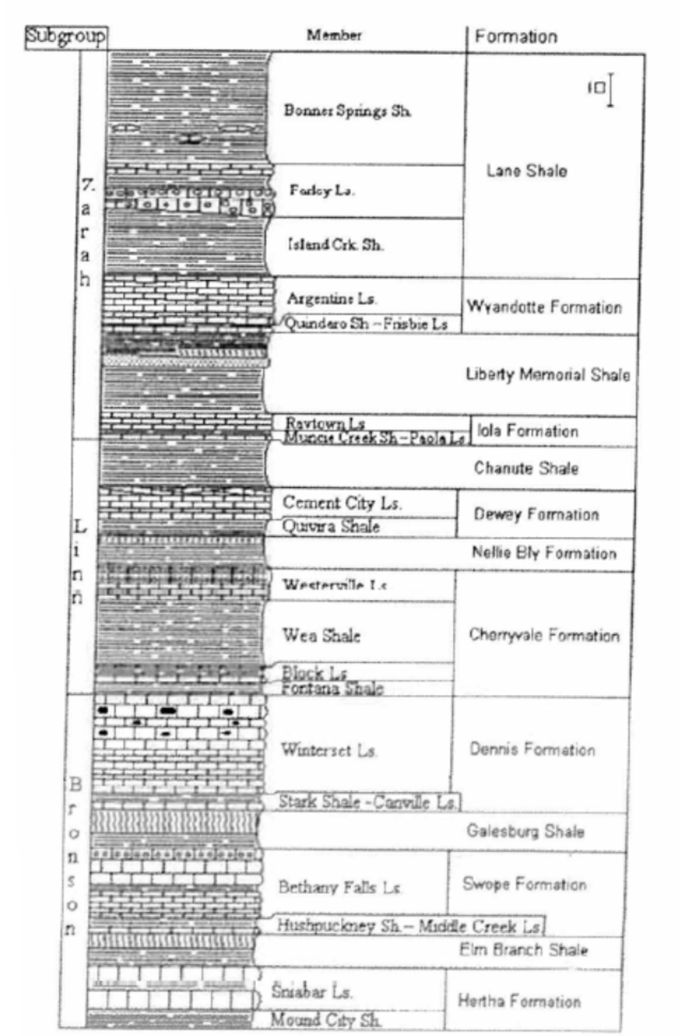
Figure 2 – Proposed Improvements

The existing roadways and proposed improvements are in the vicinity of existing underground limestone mines. Subsurface limestone mining was conducted during the 1940's and 1950's. US 40 was relocated to its present position during the 1950's. I-435 was then constructed during the late 1960's, about 10 to 15 years after mining ceased in the area. Underground limestone mines are present on both sides of I-435.

GENERAL GEOLOGIC SETTING

Subsurface Stratigraphy

The subsurface bedrock in the overall project area consists of alternating beds of Pennsylvanian-aged limestone and shale from the Kansas City Group. The Kansas City Group includes a succession of beds that extend from the base of the Hertha Formation to the top of the Lane Shale Formation. Refer to Figure 3 for a generalized stratigraphic column. This succession is divided into three subgroups (from top to bottom): Zarah, Linn, and Bronson. Of particular interest are three formations of the Bronson Subgroup (from top to bottom): Dennis Formation, Galesburg Formation, and Swope Formation.



SOURCE: PALEOZOIC SUCCESSION IN MISSOURI. PART 5: PENNSYLVANIAN SUBSYSTEM. R. GENTILE AND T. THOMPSON, 2004.

Figure 3 – Stratigraphic Succession

Dennis Formation. The Dennis Formation is the uppermost (youngest) formation of the Bronson Subgroup and consists of three members – Winterset Limestone, Stark Shale, and

Canville Limestone. The Winterset Limestone is one of the thickest limestone units in the area with an average thickness of about 30 feet. In this project area, the Winterset consists of two distinct limestone layers separated by a shale bed. The Stark Shale is medium dark gray in the upper part and dark gray to black fissile shale in the lower part. This shale can be water bearing and typically ranges in thickness from 2 to 3 feet. The Canville Limestone is typically absent in the Kansas City area and was not observed at the project site.

Galesburg Formation. The Galesburg Formation is an erodible, gray to dark gray shale to clayey shale. It is generally non-fossiliferous with a thickness ranging from 3 to 6 feet. Its basal contact with the underlying Bethany Falls is irregular and reflects the uneven or nodular development in the top of the Bethany Falls. Generally, this unit is considered impermeable with respect to water migration and forms a water tight seal above the underlying Bethany Falls Limestone.

Swope Formation. The Swope Formation contains three members (from top to bottom) – Bethany Falls Limestone, Hushpuckney Shale, and Middle Creek Limestone. The Bethany Falls Limestone constitutes the uppermost unit of the Swope Formation. The thickness of the member varies from 15 to 24 feet with a typical thickness of 20 feet. The uppermost 12 to 36 inches of the member consists of poorly cemented, clay-rich limestone nodules. Upon weathering this material turns into a residuum that resembles gravel and is locally referred to as the "Peanut Rock." The Bethany Falls Limestone generally consists of two distinct layers separated by a thin bed of calcareous shale. The upper layer, which averages 10 to 12 feet thick, is characterized as thin, irregular to wavy bedded, medium to light gray, and mostly fine to medium crystalline texture. The Bethany Falls Limestone is extensively quarried in the Kansas City area and the lower layer of the Bethany Falls is commonly mined.

The Hushpuckney Shale is typically 4 to 5 feet thick, consisting of a dark gray to black fissile shale in the lower half to two-thirds and dark gray to buff calcareous shale above.
The Middle Creek Limestone is the basal member of the Swope Formation. The Middle Creek is a thin but persistent bluish gray, finely crystalline, fossiliferous limestone with associated shale parting that ranges in thickness from 1 to 2 feet. The Middle Creek Limestone can be represented by a single massive bed at some localities or is represented by two or three limestone layers separated by shale at other localities.

Structural Geology

In general, the bedrock in the Kansas City area dips to the west-northwest at 10 to 20 feet per mile (0.2 to 0.4 percent) toward the Forest City Basin. In the vicinity of the project site, the localized bedrock dip may be altered due to the presence of the Centropolis Dome and the Centerview-Kansas City Anticline. Based on exploratory borings in the area, the apparent dip toward the north is about 1.4 percent.

Joints are generally well developed in the Bethany Falls Limestone, and two sets are common in the Kansas City area. One set strikes northwest-southeast and the other set is approximately perpendicular to the first set. Both have high angle to near vertical dips. General spacing between the major vertical joints is on the order of 20 to 25 feet. The joints are usually tight but may exhibit solutioning effects wherever the overlying cover of shale has been removed, exposing the Bethany Falls Limestone at the ground surface. This pronounced jointing, along with weathering of the Bethany Falls Limestone, results in large blocks ranging from 10 to 20 feet in length and breadth commonly observed as slump blocks along slopes in outcrops.

SITE AND SUBSURFACE INVESTIGATIONS

Literature Review

Mining began on the east side of present-day I-435 circa 1952 and advanced toward the south and east. In the vicinity of the project area, mining continued into the late 1950's (Figure 4). An overlay of the mining progression map with a plan view of I-435 indicates that mining activities originating on the east side of I-435 advanced beneath both lanes of the Interstate and within the right-of-way at several locations. A rail tunnel connects the east and west side of the mine beneath I-435 providing private service for mining activity and subsequent commercial development. Based on the mining progression map, the rail tunnel was constructed sometime after 1956 and prior to construction of I-435.

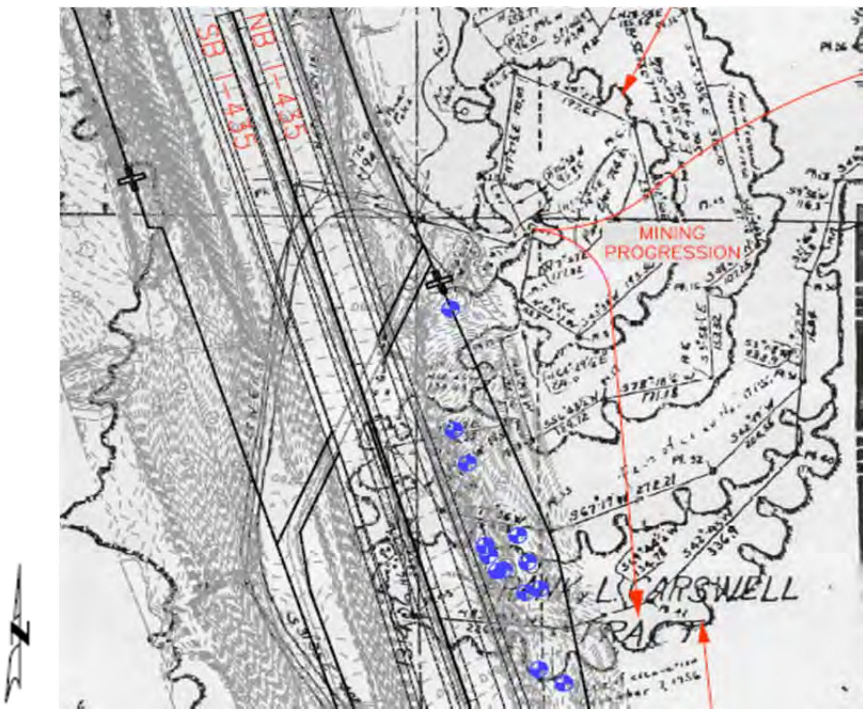


Figure 4 – Mining Progression East of I-435

On the east side of I-435, a portion of the mine space has been developed for secondary commercial use. Interstate Underground Warehouse & Distribution (IUW) encompasses a total developed area of nearly 4,000,000 square feet including a network of roadways, rail, docks, and parking areas. Figure 5 presents a plan view of the commercial underground development in relation to I-435.

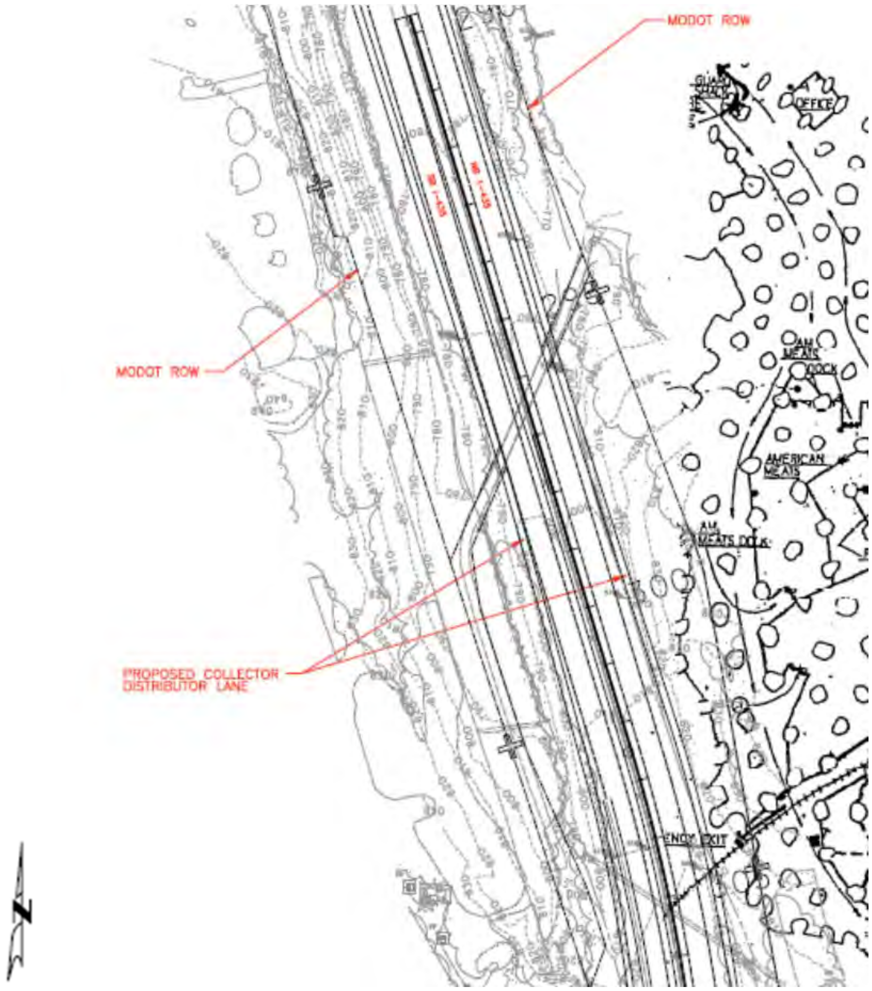


Figure 5 – Commercial Development East of I-435

Exploratory Borings

MoDOT drilled 19 exploratory borings for the mine investigation between May 12 and June 18, 2009. The borings were drilled with either a Versa Drill TR-2 4000 or a CME 850, both equipped with an NQ-sized wire line core barrel. Generally, hollow stem augers were used to advance the boring through the overburden soil and highly weathered bedrock. The NQ-sized core barrel with water as the drilling fluid was then used to obtain rock core samples. Core recovery and rock quality designation (RQD) were calculated and recorded on the final boring logs. In accordance with MoDOT standard field logging, RQD values were only calculated for the limestone portion of the recovered rock core. At several borehole locations, a down hole camera was employed to review the in-situ rock quality and the condition of the mine space (where encountered).

Lidar Laser Scanning

In order to better delineate the mine structure beneath the I-435 right-of-way within the IUW development, HNTB subcontracted with BHC Rhodes (BHC) to conduct lidar laser scanning from within the mine space. The primary area of scanning included the accessible mine space within the MoDOT right-of-way, the rail tunnel, and a limited area within the mine space on the west side of I-435 adjacent to the rail tunnel. In addition to the laser scanning, BHC collected location and elevation shots of prominent joints and other features within the mine space at the direction of HNTB personnel. The survey information collected was used to produce a localized pillar map, mine floor contour map, and mine roof contour map.

Field Reconnaissance

Between May and July 2009, HNTB made numerous field visits during drilling and surveying efforts to document these activities and map existing site features. Furthermore, detailed mapping of mine structure features with respect to the recently developed pillar map was completed. Finally, the limits of several sinkholes within the right-of-way on the east side of I-435 were surveyed. A compilation of the mine horizon and ground surface observations (primarily east of I-435) was prepared.

SUMMARY OF EXISTING CONDITIONS

Surface Conditions

Within the project area east of I-435, the ground surface elevation is variable but generally increases toward the south. Based on 1-foot contour information provided by MoDOT, the existing ground surface elevation of I-435 over mine space ranges between 797 feet and about 825 feet. For reference, the mine roof elevation in this area is on the order of 796 feet. The ground surface generally rises to the east of I-435.

Several small sinkholes are present over the mine space between NB I-435 and the existing right-of-way to the east. The sinkholes appear to be piping soil material into the mine space along solution-enlarged joints in the underlying limestone bedrock. Four of the larger sinkholes were surveyed. Maximum sinkhole dimensions are on the order of 10-foot wide by 30-foot long. Direct communication with the underground mine space was not observed.

Subsurface Conditions

The mine space directly beneath the MoDOT right-of-way is generally undeveloped. An existing loop road for the commercial underground development is the closest feature and is located between 20 and 70 feet east of the right-of-way.

The mine roof elevation in this area ranges between about 796 and 801 feet based on the laser scanning. This is in good agreement with the borings, which encountered the mine roof at elevations ranging between 790 and 798. The borings which encountered the mine space indicate a total overburden thickness ranging between 22 and 39 feet. Based on existing ground surface and mine roof elevations, the total overburden cover above the mine space may be 10 feet or less beneath the existing travel lanes of I-435 and in the vicinity of drainage features.

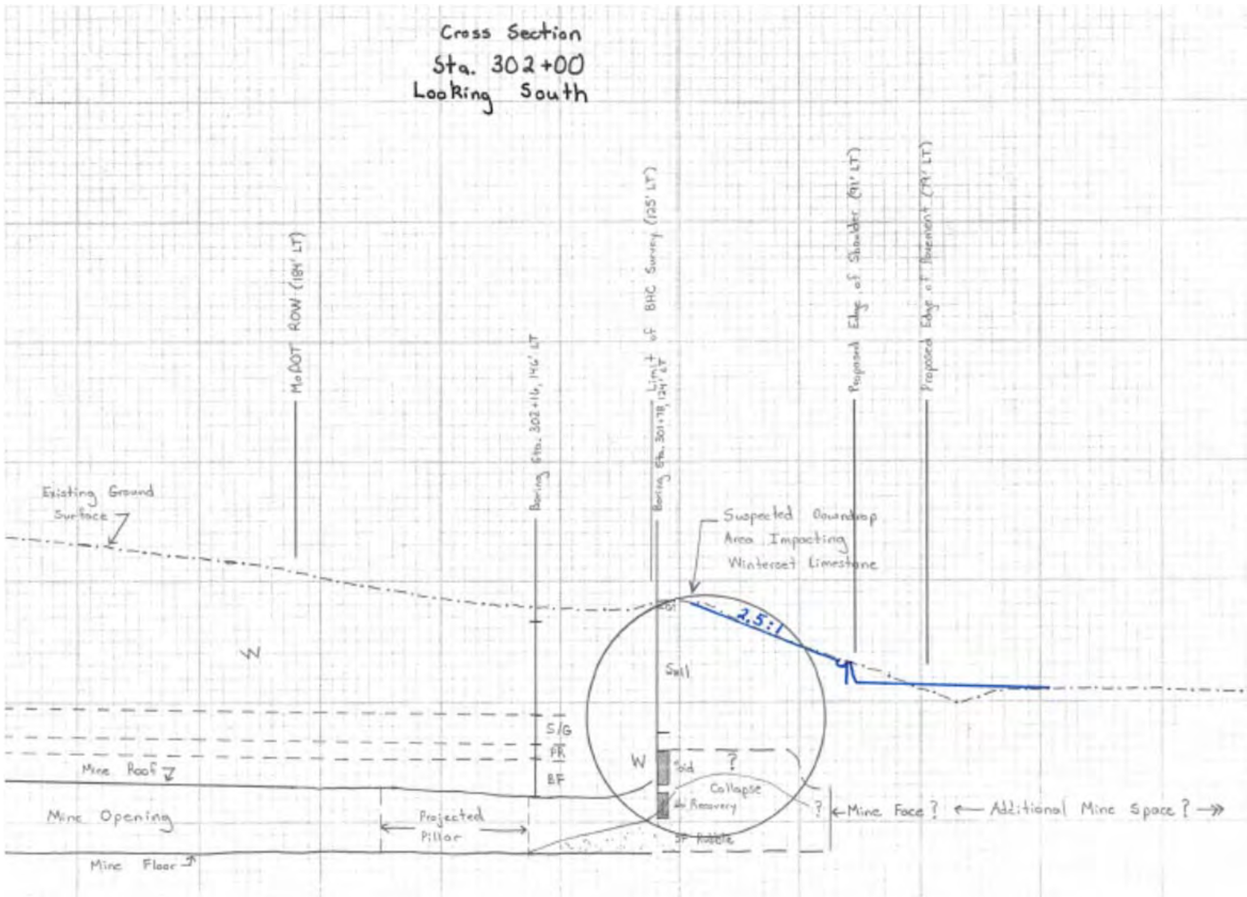


Figure 6 – Typical Cross Section

Information from the 10 borings which encountered the mine space indicates that the roof beam (remaining portion of the Bethany Falls Limestone forming the immediate mine roof) thickness averages 7.5 feet, including the Peanut Rock which is typically 2-feet thick. It is noted that several of the borings with reduced roof beam thickness are located in areas where the lower beds of the Bethany Falls Limestone have detached from the mine roof. The combined thickness of the Stark/Galesburg Shale overlying the Bethany Falls Limestone averages about 5 feet. The Winterset Limestone is the first bedrock unit encountered at the borings located above the mine space and its thickness is highly variable due to weathering. The measured thickness of the Winterset at the boring locations ranges between 6 and nearly 26 feet with an average of about 16 feet. Two borings encountered voids within the Winterset Limestone corresponding to domeouts observed within the mine horizon that have apparently impacted the integrity of the Winterset unit. Furthermore, another boring indicates a uniform downdrop of the Winterset, Stark/Galesburg, and Bethany Falls units of approximately 5 feet. Typically the soil mantle is less than 5 feet thick at the boring locations, except for one boring which encountered nearly 21 feet of moist clay.

The original mine floor is generally positioned at elevation 785 feet, corresponding to the contact between the Bethany Falls Limestone and underlying Hushpuckney Shale. Thus, the typical mine room height is about 10 to 11 feet. In order to provide truck access for the commercial development, the loop road has been downcut through the Hushpuckney Shale and is positioned at approximately elevation 782 feet. North of the rail tunnel, a drainage ditch has been cut down through the remaining Hushpuckney Shale to approximately elevation 783 feet. West of the drainage ditch, the mine rooms have been partially filled with random mine spoil pushed in from within the mine space. Random mine spoil material has also been placed in several mine rooms south of the rail tunnel. Generally, the maximum height of the mine spoil piles is about 4 to 5 feet below the mine roof level.

The mine reconnaissance included the accessible mine space within the limits of the MoDOT right-of-way and generally extended eastward to IUW's loop rood (Figure 7). The reconnaissance included mapping of the general mine conditions including presence and condition of joints, locations of water seepage, presence and condition of backfill material, and other pertinent features. As noted above, mine spoil material consisting of limestone screenings and shale has been used to partially fill several mine rooms near the west edge of the reconnaissance limits. It appears that this material was placed to limit personnel access further to the west where several collapse features were observed. These collapse features consist of roof falls of the Bethany Falls Limestone and upward extension through the Stark/Galesburg Shale and into the Winterset Limestone. As noted above, boring information suggests that the overlying Winterset Limestone has been impacted at several locations. Based on the mining progression maps, mining appears to have extended further west (about 175 to 200 feet from the limits of the laser scan) underneath both lanes of I-435. However, due to the collapses and associated debris present along the western limits of the reconnaissance, visual confirmation of the actual mining extents could not be obtained. At one location, rubble backfill consisting of 6 to 12-inch diameter limestone cobbles and boulders was placed in the mine space extending beneath the NB lane of I-435. Prominent joints are present in the mine roof and generally trend northeast-southwest with a conjugate set trending northwest-southeast. Near the west mine face and the existing collapse features, the joints exhibit deflection and solution widening, indicating distress of the mine roof beam and areas with limited overburden cover, respectively. Finally, tree roots were observed at several locations along the west mine face.

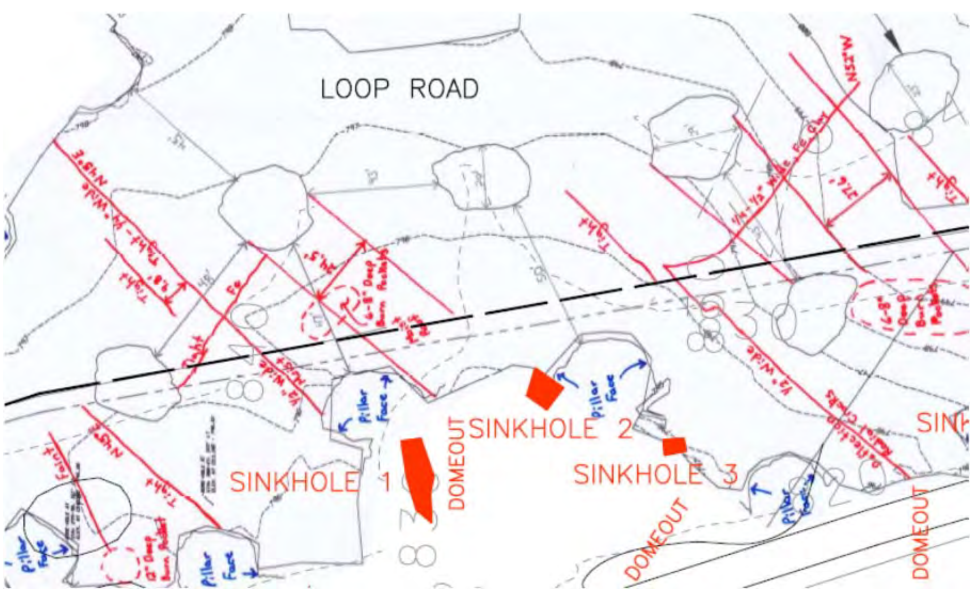


Figure 7 – Example of Detailed Mapping of Mine Structure Features

The rail tunnel crosses beneath I-435 and measures approximately 15 feet wide by 20 feet tall by 300 feet long. The tunnel crown is arched and positioned within the upper portion of the Bethany Falls Limestone. The ballast for the tracks is founded on the Hertha Formation. Tunnel support is provided by steel ribs and lagging, which extends well beyond the limit of the new SB collector distributor lane and just beneath the limit of the NB collector distributor lane. Wood lagging is present at localized areas primarily within the crown and may have been installed as temporary support. The steel components exhibit corrosion (rusting) and some of the wood lagging is rotting. Water inflow is visible along the south wall of the tunnel. The presence of grout pipes indicate that grouting of the annular space between the tunnel support and the underlying bedrock has occurred to some extent. The maximum tunnel crown elevation is on the order of 796 feet with a tunnel invert elevation of 775 feet. The ground surface elevation of the existing NB and SB lanes of I-435 is about 820 feet (20+ feet of cover), while the ground surface elevation in the vicinity of the proposed new collector distributor lanes is on the order of 812 to 815 feet. Thus, existing overburden cover in vicinity of the new lanes is about 20 feet or less.

ENGINEERING ANALYSIS AND DESIGN

Mine Structure Stability

Mine structure stability was evaluated with respect to overstressing the limestone pillars, overstressing the tensile capacity of the mine roof beam, bearing capacity failure, and kinematic instability. Overstressing the pillars and bearing capacity failure were not calculated to be of concern with respect to long-term mine structure performance. Mine roof stability was of concern with respect to overstressing the roof beam and kinematic instability due to the presence of near vertical joints. Roof beam instability and upward progression into the overlying Winterset were present within and adjacent to the MoDOT right-of-way.

A plan view of the area was developed illustrating the anticipated risk of roof beam instability on the east side of I-435 related to the thickness of the existing overburden cover (Figure 8). The area beneath the existing lanes of I-435, where mining progression maps indicated the presence of mine space, was included in the “high” risk area. According to MoDOT personnel, the pavement of I-435 had not experienced distress over the life of the infrastructure. Thus, the assumption was made that this area was backfilled in advance of roadway construction as stability calculations suggested that this area would have been the first to fail. The assumption was also made that the observed collapses at the west edge of the 3D survey information were located outside the lateral limit of the previously placed backfill. These collapses were positioned in near vicinity or directly beneath the limits of the proposed new collector distributor lane. Finally, several collapse features were located at the transition from the “high” to “moderate” risk areas. At the perimeter of the existing collapses, the mine roof exhibited evidence of distress in the form of joint dilation. These observations suggested the potential for continued lateral migration of roof instability into the “moderate” risk area to the east.

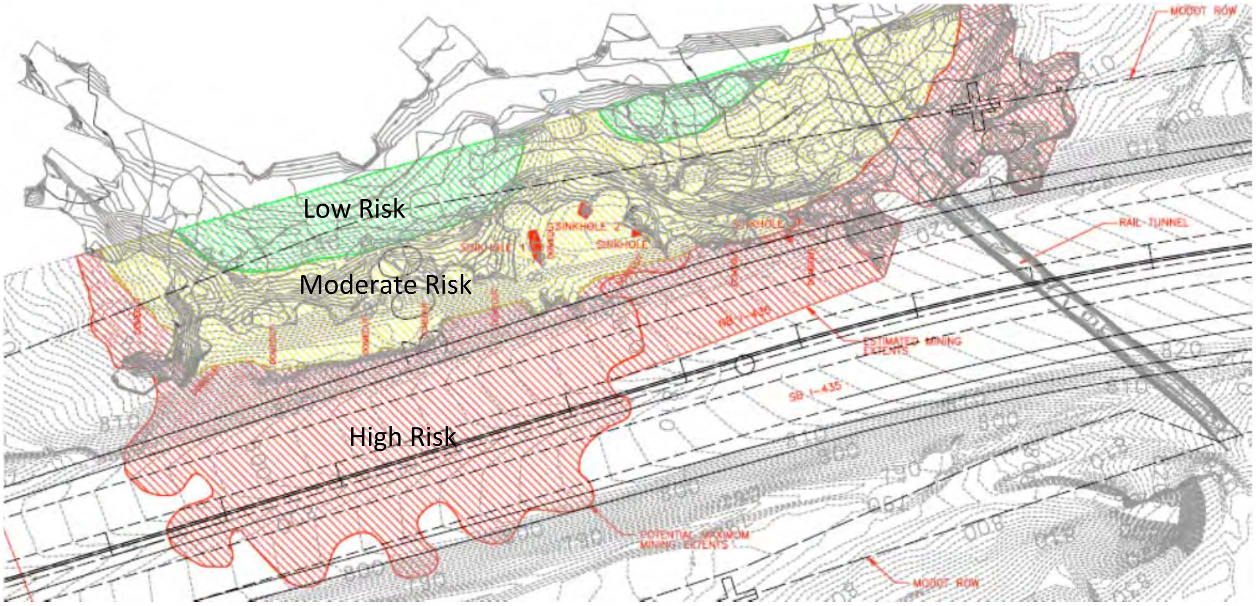


Figure 8 – Risk Map of Mine Roof Instability

HNTB subcontracted with Wyllie & Norrish Rock Engineers, Inc. (W&N) to conduct additional numerical modeling to substantiate the theoretical calculations. Axisymmetric and plane strain 2-dimensional models were constructed to evaluate the mine roof stability. In general, the numerical modeling correlates well with the theoretical calculations and indicates unstable to marginally stable mine roof conditions near the perimeter of the mine where the Winterset Limestone is either of reduced thickness or absent.

Design of Grouting and Rail Tunnel Improvements

Based on design criteria and direction from MoDOT, HNTB prepared design drawings and a job special provision for ground improvements. In general, backfilling operations included drilling and placing grout from the ground surface within a 60- to 80-foot wide corridor extending from the sawcut line for the proposed collector-distributor lane toward the east. Both low slump and high slump grout were specified. The primary purpose of the low slump grout was barrier formation while the primary objective of the high slump grout was void filling within the existing collapses. Refer to Figure 9 for a schematic of the mine grouting.

Figure 9 – Schematic of Mine Grouting Plan

Based on roof beam calculations, the existing rail tunnel and mine approach configuration were considered conducive to long-term structural stability, even with the proposed fill placement. However, roof rock reinforcement was specified to reduce the likelihood of lower bed delaminations, which reduce the overall roof beam thickness. Rock bolts were designed to effectively “knit” the individual beds of roof rock together to form a beam of known thickness. Roof bolts are considered a positive approach to reinforcing the rock mass. A 4-inch thick layer of reinforced shotcrete was also specified to provide confinement of surficial roof rock material between the rock bolts, improve aesthetics, and provide an additional layer of corrosion protection over exposed steel elements.

Final plans and special provisions for the grouting and rail tunnel improvements were submitted on October 30, 2009. The engineers estimate for the work was $1.85 million.

CONSTRUCTION

The prime contractor (Clarkson) for the project subcontracted with Hayward Baker, Inc. (HBI) to complete the grouting and rail tunnel improvements. The bid amount for the grouting and rail tunnel improvements was $1.3 million.

HBI mobilized to the site and began work on the rail tunnel improvements the week of May 17, 2010 (Figure 10). Rock bolt installation was completed on June 10, 2010 with shotcrete placement for the rail tunnel improvements finished on June 16.

Figure 10 – Rock Bolts and Mesh for Rail Tunnel Improvements

The ground improvement grouting operation began on June 10, 2010 with drilling on the south end of Rows A and B for the low slump barrier grout. Drilling was accomplished with a high production type down the hole hammer drill rig, drilling 6-inch diameter holes. The holes were drilled to the minimum specified target elevation and logged by the driller. Grouting was accomplished with 3- and/or 4-inch diameter grout casing lowered and lifted with various crane methods. Grout was mixed at an offsite, HBI run, central batch plant and brought to the site in concrete transit trucks. Grout was pumped with various types of concrete pumps capable of achieving at least 500 psi at pump end. Grout volume and pressure were constantly measured and monitored using an automated system. HBI substantially completed ground improvement grouting south of the rail tunnel on July 9, 2010. HBI advanced the ground improvement grouting north of the rail tunnel by drilling and grouting with low slump grout Rows A and B concurrently, followed by Rows D and E. Finally, high slump grout was placed in Row C and designated holes in Rows D and E. Drilling and grouting of first order holes were completed on August 4, 2010. Verification was accomplished using holes drilled with the 6-inch diameter down the hole hammer. A video camera was lowered into each hole and inspected in real time. Verification hole drilling, video inspection, and grouting were completed on August 10, 2010.

Figure 11 – Barrier Grout Placement

The actual quantities were in close agreement with the estimated contract quantities except for the high and low slump material. The largest disparity was between the estimated volume and actual volume of grout placed in Rows C, D, and E, where the actual volume of grout placed was less than the estimated volume. These rows were primarily positioned outside the limits of the lidar survey data and were inaccessible for visual reconnaissance during design and construction. Furthermore, these rows are located nearest to I-435 in an area where previous backfilling was unknown . Based on the drill logs and grout take, this area was most likely backfilled in advance of the original I-435 construction. In Rows A and B, where evidence of recent instability was observed during the design phase and lidar and visual surveys confirmed the geometry, the grout quantity placed was in agreement with the estimated quantity. Close monitoring of the grouting activities resulted in over $450,000 of savings in grout volume.

The verification holes and video indicated very little evidence of remaining voids. A walk through of the mine area along the perimeter of the grouting indicated satisfactory grouting with grout in contact with the mine ceiling. A final inspection of the rail tunnel improvements indicated the work accomplished fulfilled the intent of the design.

HNTB also provided supporting documentation to MoDOT to refute a claim by the subcontractor due to the underrun of the high slump grout material.

CONCLUSIONS

A risk based approach was taken to mitigate the risk of mine collapse for the addition of a half interchange and collector distributor lanes to I–435. A systematic plan of filling the mined area under the entire right of way would have essentially taken all forms of risk to zero; however, this approach would have been more expensive, in certain areas redundant, and taken a considerable amount of time.

Instead a plan of avoidance and treatment of areas of high or moderate risk was taken. The original plan of placing 60 feet of fill over a high risk sinkhole area under the southbound I-435, northwest quadrant exit ramp was mitigated by changing the interchange configuration and folding the ramp to the south side (southwest quadrant) of the interchange.

Mines along the west right of way that may have slightly encroached, were given a low risk of potential collapse and eliminated from remediation.

Mine maps indicated the former mine extended well under the existing lanes of I 435. No construction or inspection records of the original construction of I–435 were found or recovered during the study. Although no conclusive evidence of backfilling or remediation during the original construction 40 years ago was found, no evidence of distress had been noted. Therefore, this area was not included in the remediation, but may have advanced if voids or large grout takes occurred in the adjacent area during mine filling. Neither case was encountered and the area eliminated from remediation.

Mines along the east right of way were evaluated for high, moderate, and low potential for collapse. Risk was based on right or way, limits of improvements, depth of cut and fill, condition of the existing mine, collapse features and thickness of overburden. Risk was minimized by limiting improvements by steepening cuts and fills and use of barrier. Only then, were areas deemed of high or moderate risk planned for mitigation.

An active on-site engineering and inspection program also matched the ground improvement or grouting volumes to the conditions encountered in the treatment holes giving way to an “investigate and treat” program during construction. This approach and oversight resulted in significant cost savings.

Overall, the evaluation of risk and MoDOT’s approach of accepting some forms of risk led to a cost effective means of remediating the underground mines for this project.

Characterizing Karst Geology Beneath Proposed Roadways Using Geophysical Methods

Gabriel J. Hebert
Mundell & Associates, Inc.
110 South Downy Avenue
Indianapolis, IN 46219
(317) 630-99060
ghebert@mundellassociates.com

John A. Mundell
Mundell & Associates, Inc.
110 South Downey Avenue
Indianapolis, IN 46219
(317) 630-9060
jmundell@mundellassociates.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to thank the lead geotechnical engineer for his contribution to the work described in the paper:

Mr. Thomas Struewing, ATC Associates, Inc.

Disclaimer

Statements and views presented in this paper are strictly those of the authors, and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original authors.

ABSTRACT

The development and expansion of highways on top of karst geology always presents challenges and has the potential for catastrophic failure. Recently, a portion of State Road 56 in Orange County, Indiana known to sit atop potentially karstic limestone was due to be renovated. The presence of several known solution features (including an artesian spring) along this route, led the lead structural engineer on the project to request a subsurface investigation along six portions of the highway (including three bridge locations) totaling 10,800 linear feet, where the proposed roadway would deviate the most from the existing right-of-way. This portion of Indiana, known as the Mitchell Plain, is underlain by Middle Mississippian limestone formations, which are very solution-prone and susceptible to the formation of karst features such as voids and sinkholes, which when left undiscovered, can prove disastrous to roadways. Detection of karst features through traditional test boring methods can be extremely costly and time-consuming. Thus, given the length of right-of-way to be scanned, the lead engineer proposed to locate potential solution features by measuring contrasts in the physical properties of the subsurface soil and bedrock through geophysical methods.

For this project, a preliminary terrain conductivity survey was performed along each of the six areas to yield information regarding the thin residual soils and shallow bedrock, followed by two-dimensional resistivity profiling to detect any karst features deeper within the bedrock. The end result of the geophysical study gave the lead engineer what he wanted – a detailed subsurface investigation which defined site conditions and located potential hazards, all conducted in a timely and inexpensive manner.

INTRODUCTION

It is commonly known by geotechnical and structural engineers that building on top of karst- prone limestone can prove disastrous if potentially hazardous features are not recognized prior to final design and construction, and dealt with appropriately. Given the recent increase in funding available to update and expand the nation's roadways, this preliminary due diligence is critical to the portions of the country affected by solution-prone limestone. As it is, southern Indiana is one of those areas. Recently, a portion of State Road 56 (SR 56) in Orange County was due to be renovated. The presence of known karstic features along this route led the lead structural engineer to request a preliminary subsurface investigation along the six portions of the proposed roadway that would deviate the most from the existing right-of-way, as well as three bridge locations due to be shifted and replaced. Our company was contacted to use geophysical methods to detect and map out karst features in the limestone bedrock beneath those portions of the route.

Site Description and Geology

The section of SR 56 that was investigated is located between the towns of Paoli and West Baden Springs in Orange County, Indiana, approximately two hours southwest of Indianapolis and forty-five minutes northwest of Louisville, Kentucky (see Figures 1 and 2).

Figure 1 – Site Location Map

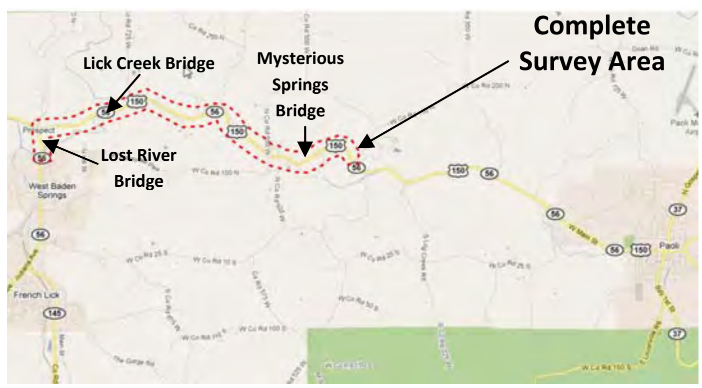


Figure 2 – Section of SR 56 Investigated

Orange County, Indiana is located within the physiographic region known as the Mitchell Plain. The soil in this part of the state is classified as the Crider-Frederick-Caneyville Silty Loam, and it is underlain by middle Mississippian limestone, specifically of the Sanders and Blue River Groups, which is highly susceptible to solutioning (*1*). Karstification occurs when carbonic acid (H_2CO_3) from atmospheric carbon dioxide and rainwater, percolates downward into subsurface waters and dissolve carbonate bedrock. This process continues enlarging fractures into cavities that may collapse, causing a sinkhole. Karst topography is evidenced by numerous closed depressions, sink holes, disappearing streams, springs, and cave openings, all of which are present in this portion of Indiana (*2*). Within mature karst systems, surficial depressions are caused by the dissolving (or solutioning) of underlying carbonate rocks along existing joints and fracture systems that result in enlarged void spaces. Subsequently, the loss of shallow soils occurs as surface water infiltration is directed to these void spaces and the soil is swept into them through the resulting groundwater movement through fractures zones (see Figure 3).

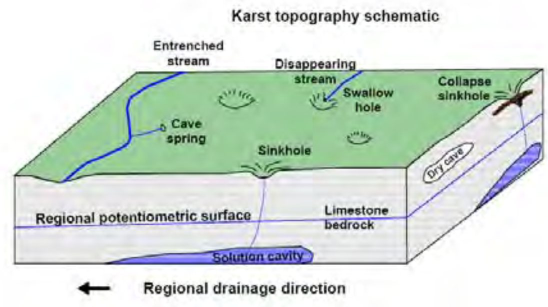


Figure 3. Karst Topography Features

TECHNICAL APPROACH

In cases such as this Site, where karst features are known to exist, geophysical mapping can provide insight into the locations of concealed features such as sinkholes, solution-enhanced fracture zones, and voids. In general, a variety of geophysical techniques can be applied to the mapping of subsurface karst features; however, certain methods, sensitive to a range of contrasting physical properties, can have attributes that make them more suitable than others depending on the site-specific conditions. Contrasting physical properties that typically are found to be useful for mapping soil and bedrock include electrical conductivity or resistivity, acoustic velocity, density, and magnetic susceptibility. Of these, electrical conductivity/resistivity has often been found to have the greatest range of contrast, and is often applicable to karst sites.

The technical approach presented here is similar to the multi-step approach taken by Ahmed and Carpenter (*3*) and Byer et al. (*4*). It begins with reconnaissance mapping with terrain conductivity to form a basic understanding of the soil and bedrock relationships in terms of apparent conductivity or resistivity, preferably at a few depth levels. The interpretive emphasis is directed towards potential air or fluid filled voids and solution-enhanced features, which could contain underconsolidated soils (*i.e.*, materials with high moisture content and low shear strength). Interpretation of the terrain conductivity data is then supplemented with two-dimensional electrical resistivity imaging (2-D ERI) within the context of the interpreted terrain conductivity data. Finally, the terrain conductivity and 2-dimensional resistivity data are interpreted together and a final geologic model is developed. This approach, applied to the survey area, and other sites, has proven to be a useful tool in the investigation of karst limestone (*4*). It has been shown to be relatively rapid and cost-effective while still providing a reasonable degree of assurance to the geotechnical engineer that significant karst features have been addressed. A brief description of these two techniques is presented in the sections below.

Reconnaissance Mapping Using Terrain Conductivity

The geophysical survey area for the SR56 Rehabilitation Project consisted of the six areas where the proposed roadway deviated the most from existing roadway (see Figure 4). Additionally, three bridges that would be replaced as part of this project were investigated as well (the Mysterious Springs Bridge, the Lick Creek Bridge, and the Lost River Bridge). The six roadway areas of investigation consisted of a total of approximately 10,800 linear feet and 37 acres. Given the large area of the survey area, it was determined that a preliminary conductivity survey would be conducted to delineate soil variations, yield insight into the upper most weathered bedrock layer, and guide the layout of the vertical resistivity profiles. For this investigation, a *Geonics EM-31* electromagnetic terrain conductivity meter was used. This instrument can gather apparent conductivity data at a relatively rapid pace without the need for direct (galvanic) ground contact, making it economical and efficient for covering large areas.

Figure 4. The Six Roadway Areas Investigated with Geophysics

To assure that the *EM-31* was operating properly, five instrument checks were made at the start of the day. First, a battery check was made to ensure proper supply voltage over the duration of the survey. Second, a DC null adjustment was made to verify the zero position of the receiver circuitry. Third, a compensation check was made to verify the zero reading of the in-phase component. Fourth, a phase check was made to calibrate the conductivity reading. And finally, a sensitivity check was made to ensure that the instrument was reading as expected. No daily drift correction was necessary with this instrument.

Conductivity data were collected by securing the *EM-3*1 to a rigid plastic sledge and towing it behind an all-terrain vehicle, collecting electrical conductivity data nearly continuously. *EM-31* data were collected along lines spaced approximately 5 to 10 feet (approximately 1.5 to 3 meters) apart with an in-line data point spacing of 1 to 3 feet (approximately 0.3 to 1 meter), depending on the speed of the instrument. Positioning data were provided by a *Trimble Ag114* global positioning system (GPS) receiver with real-time satellite based differential correction. GPS and conductivity data were simultaneously recorded in a handheld field data logger. The data stored in the data logger were in the form of apparent conductivity in milli-Siemens per meter (mS/m). However, this data was later converted to apparent resistivity for mapping purposes and comparison to the 2-D ERI results by inverting the apparent conductivity data and multiplying by 1000. The resulting apparent resistivity data are in units of ohm-meters. Once the data were converted, they were imported into *Surfer* V*ersion 8.0* for contouring and plotting as a color-filled contour map of terrain resistivity (see Figures 5 through 10).

Two-Dimensional Electrical Resistivity Imaging (2-D ERI)

After the conductivity data had been collected and processed from each of the six survey areas, revealing the variable, weathered-bedrock surface, each area was further investigated with 2-D ERI using a *Sting Resistivity Imaging System* from Advanced Geosciences, Inc. This method consists of recording direct measurements of the apparent electrical resistivity of subsurface materials *(i.e.*, resistivity of homogeneous isotropic ground that would give the same voltage-current relationship as that measured) in a profile-type data set known as an apparent resistivity pseudo-section. Once the apparent resistivity data were collected, they were downloaded to a computer and were subsequently inverse-modeled using the software *EarthImager 2D v1.9.9* to obtain a cross-section of the "actual" resistivities of subsurface materials. This is accomplished through the process of generating a model resistivity cross-section, calculating the theoretical apparent resistivity pseudo-section that would result from such a model, and comparing the theoretical pseudo-section to the one collected in the field. The model is then altered through a number of iterations until the theoretical and field-collected pseudo-sections closely match each other. At this point the model is considered to be a reasonable estimation of the "actual" resistivities of the subsurface materials.

It should be noted that electrical resistivity (and its inverse, conductivity) is one of the most widely-varying of the physical properties of natural materials. Certain minerals, such as native metals and graphite, conduct electricity via the passage of electrons; however, electronic conduction is generally very rare in the subsurface. Most minerals and rocks are insulators, and electrical current preferentially travels through the water-filled pores in soils and rocks by the passage of the free ions in pore waters (*i.e*., ionic conduction). It thus follows that degree of saturation, interconnected porosity, and water chemistry (*i.e*., totally dissolved solids) are the major controlling variables of the resistivity of soils and rocks. In general, electrical resistivity directly varies with changes in these parameters. Fine-grained sediments, particularly clay-rich sediments such as silty loam, are excellent conductors of electricity, often much better than fresh water found in the pores of sand and gravel. Carbonate rocks (*i.e*., limestone and dolomite) are very electrically resistive when they are unfractured, but can have significantly lower resistivity values when fractured and/or solutioned.

Based on the preliminary conductivity survey of the six study areas, a total of five (5) resistivity profiles were collected (no resistivity profile was collected in Area 6 – see Figures 5 through 9), utilizing a 60 electrode spread, and a dipole-dipole configuration to characterize the electrical properties of the upper 65 to 230 feet (approximately 20 to 70 meters) of the subsurface, depending on the electrode spacing (which ranged from 1.5 meters for the profile in Area 2, to 5 meters for the profile in Area 5). Additionally, a resistivity profile was collected along the existing and proposed footer locations of three bridges within the study area that were to be shifted to the new right-of-way: the Mysterious Springs Bridge in Area 5, the Lick Creek Bridge in Area 3, and the Lost River Bridge just north of West Baden Springs (see Figures 11 through 13).

It should be mentioned that resistivity cross sections are 2-dimensional representations of the general distribution of electrical resistivity in the 3-dimensional subsurface. Although there is no unique direct conversion from resistivity values to lithology, based on site knowledge, geometric shapes and relationships of various anomalies, and the observed ranges of resistivity values, reasonable geologic interpretations can be made. The interpretations of this survey are presented in the section below.

DATA INTERPRETATION DISCUSSION

The six EM31 terrain resistivity maps representing the six study areas (Figures 5 through 9) show the apparent resistivity[1] within the upper 16 feet (5 meters) of the subsurface. These maps are a reflection of the conditions in the relatively shallow subsurface materials (generally residual soils, fill, or weathered bedrock). Conductivity/resistivity variations on these maps reflect variations in the depth to bedrock as well as changes in porosity, moisture content, fracture density, clay content, and void space material. Lower resistivity values, i.e., approximately 90 mS/m and below[2], are interpreted as thicker soil layers that may also be higher in moisture content and/or clay and silt (presented as orange to purple in color). In upland areas these thicker soils are often moist, and with low strength

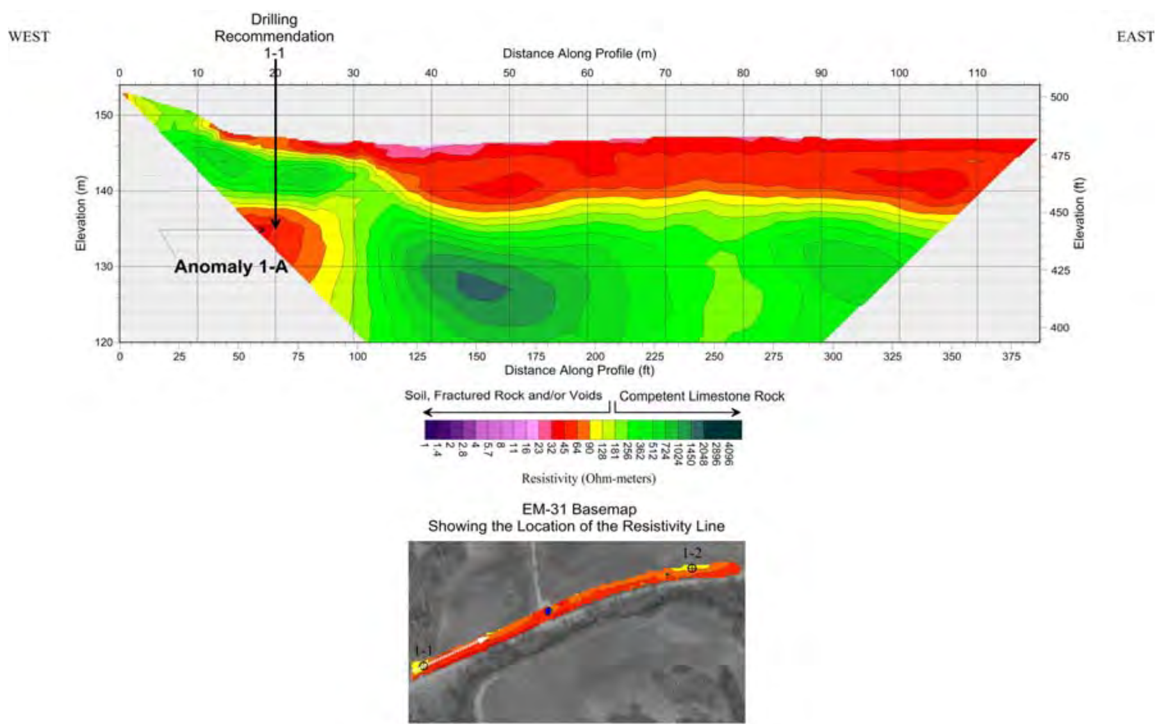


Figure 5. Area 1 Resistivity Profile and Conductivity Map

[1] Essentially the volume-weighted average resistivity of subsurface materials
[2] Extremely low (less than 2 ohm-meters) and negative values are an indication of metallic materials.

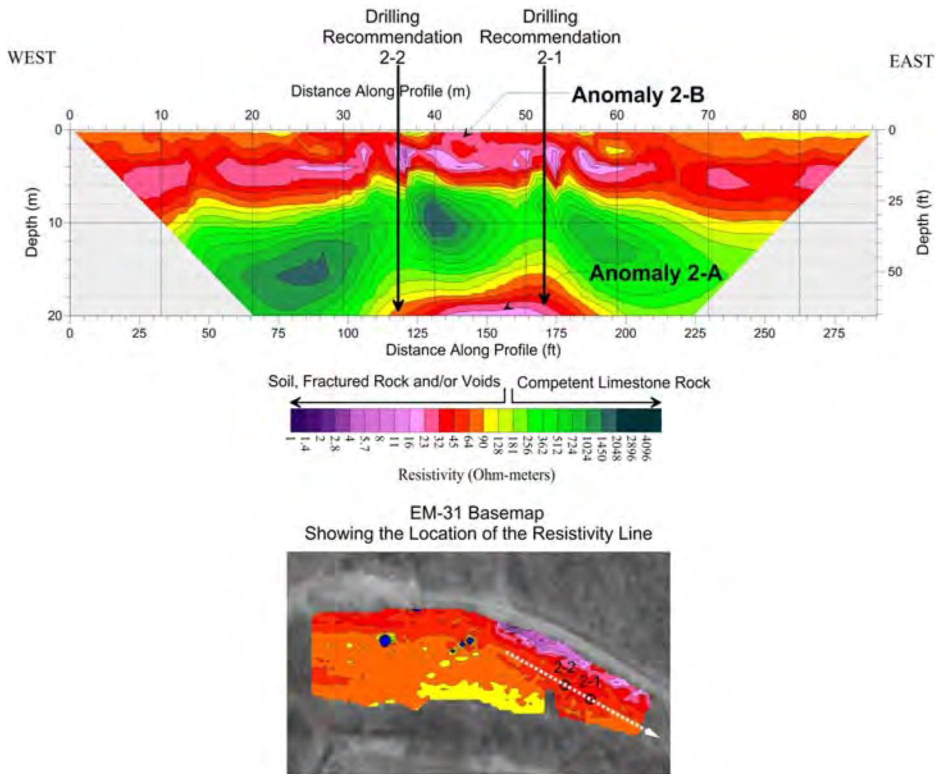


Figure 6. Area 2 Resistivity Profile and Conductivity Map

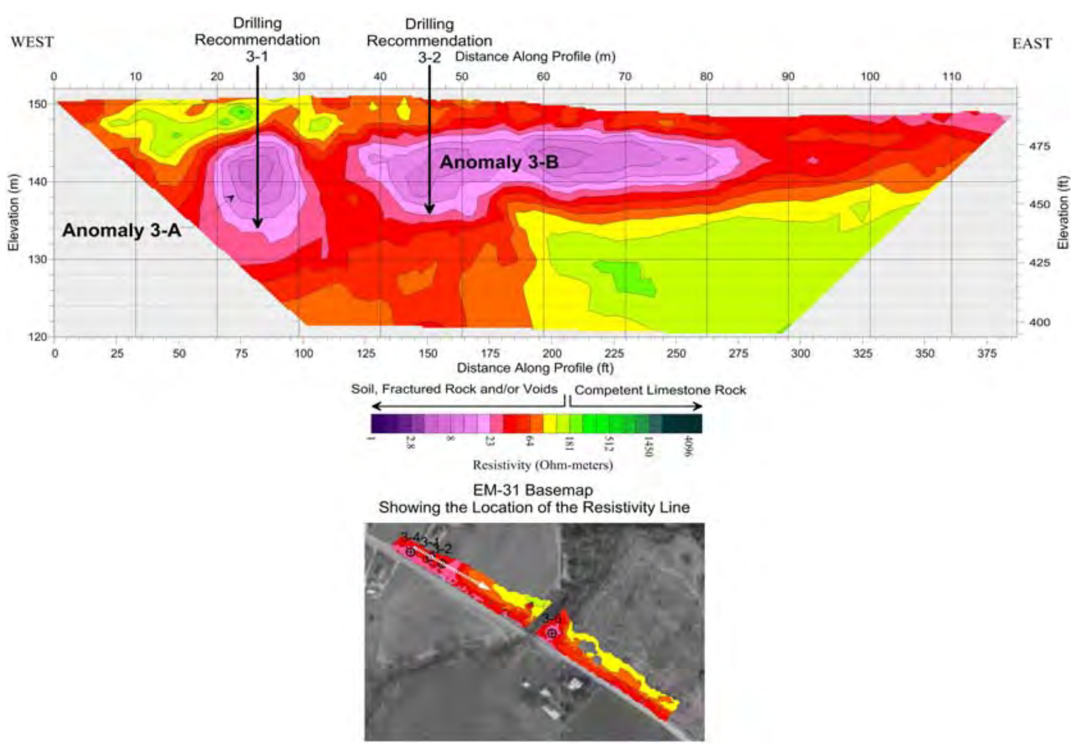


Figure 7. Area 3 Resistivity Profile and Conductivity Map

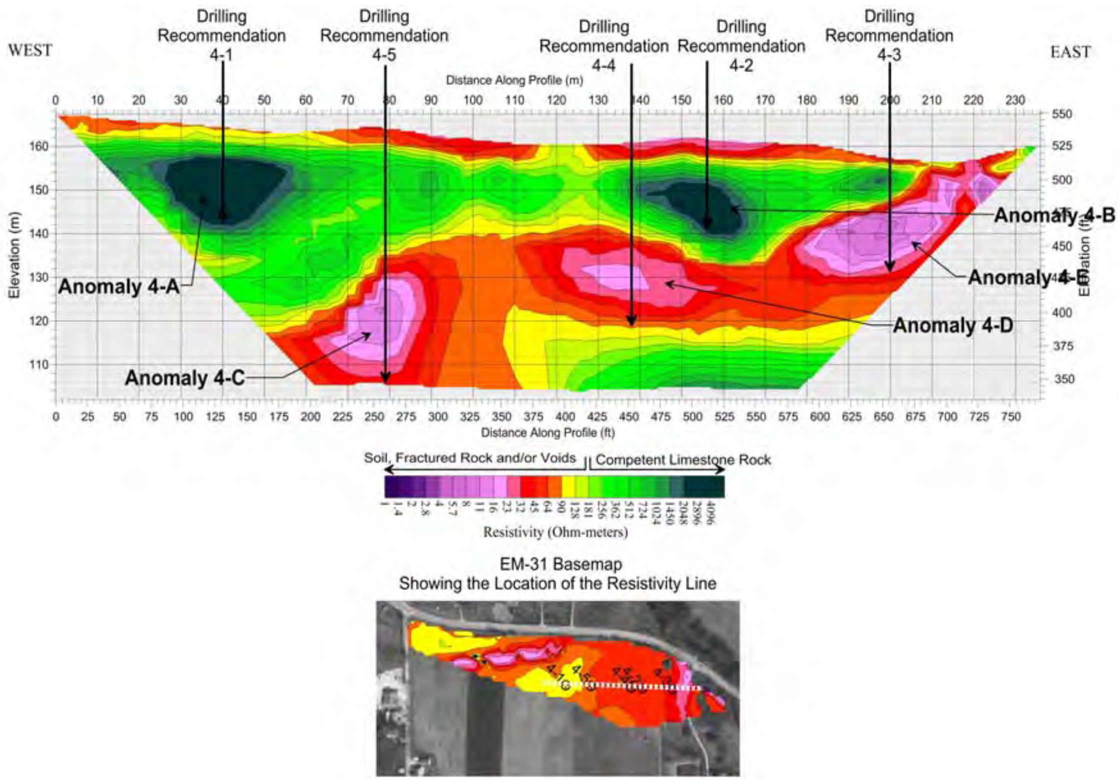


Figure 8. Area 4 Resistivity Profile and Conductivity Map

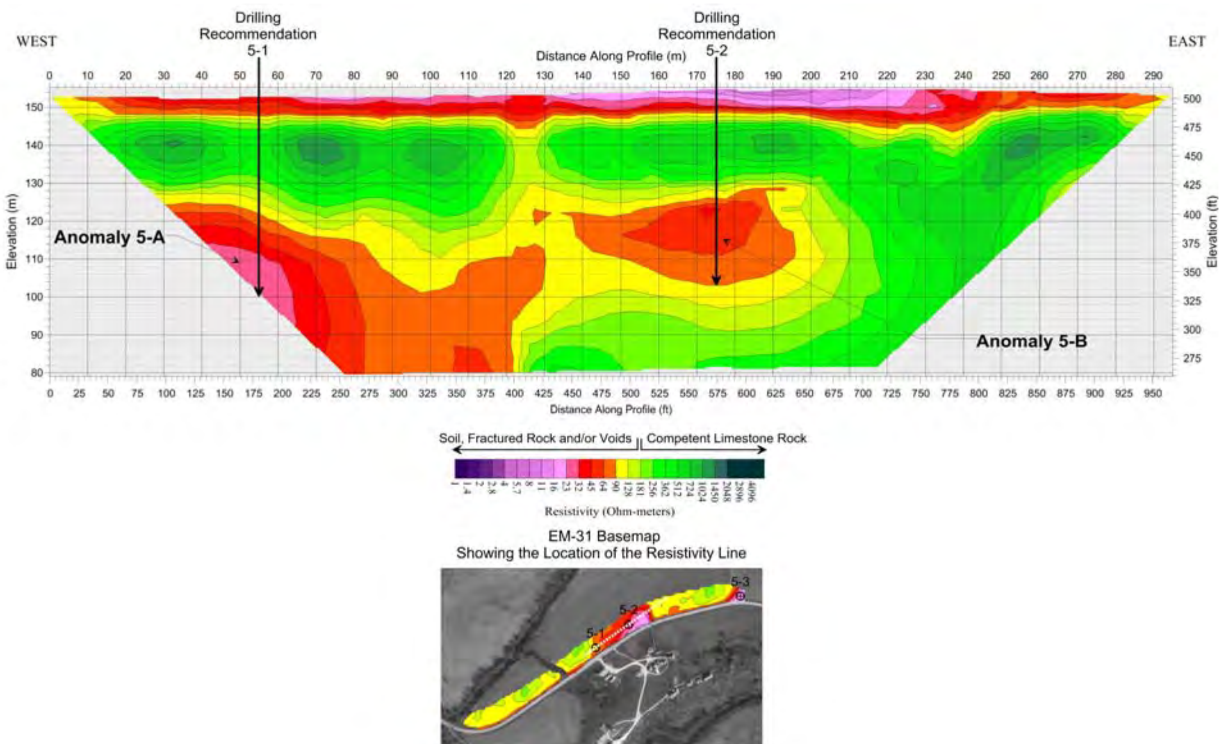


Figure 9. Area 5 Resistivity Profile and Conductivity Map

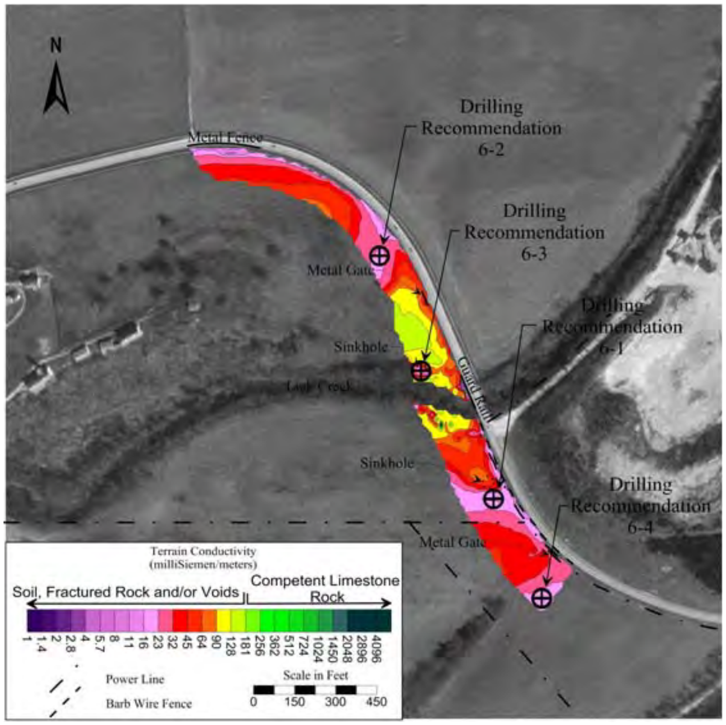


Figure 10. Area 6 Conductivity Map

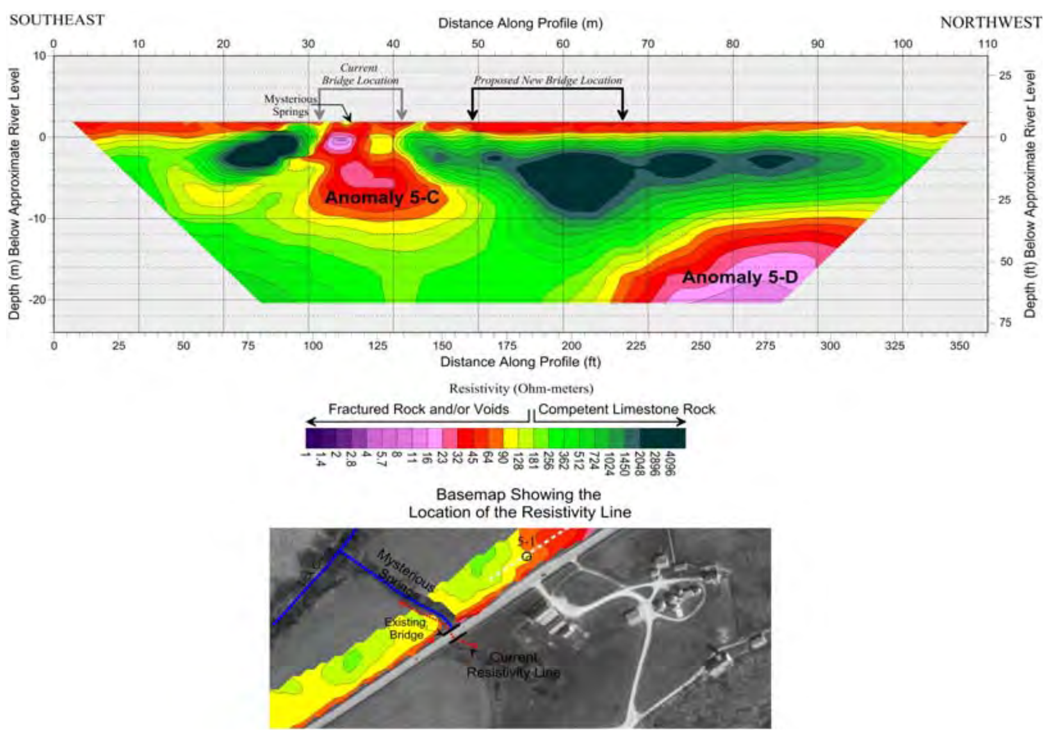


Figure 11. Mysterious Springs Bridge Resistivity Profile

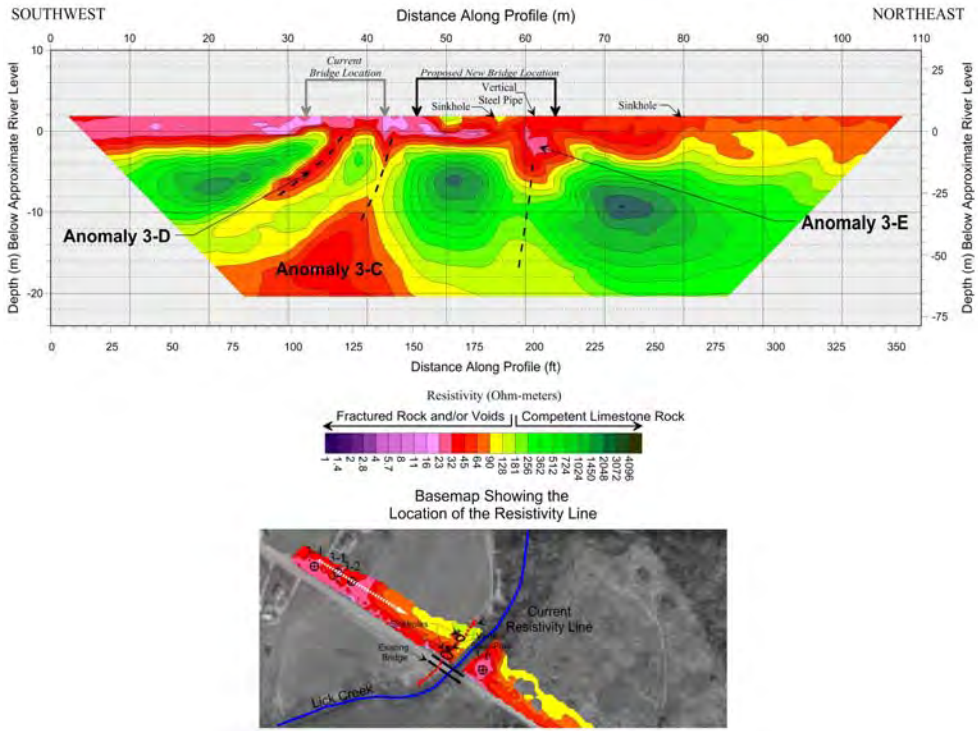


Figure 12. Lick Creek Bridge Resistivity Profile

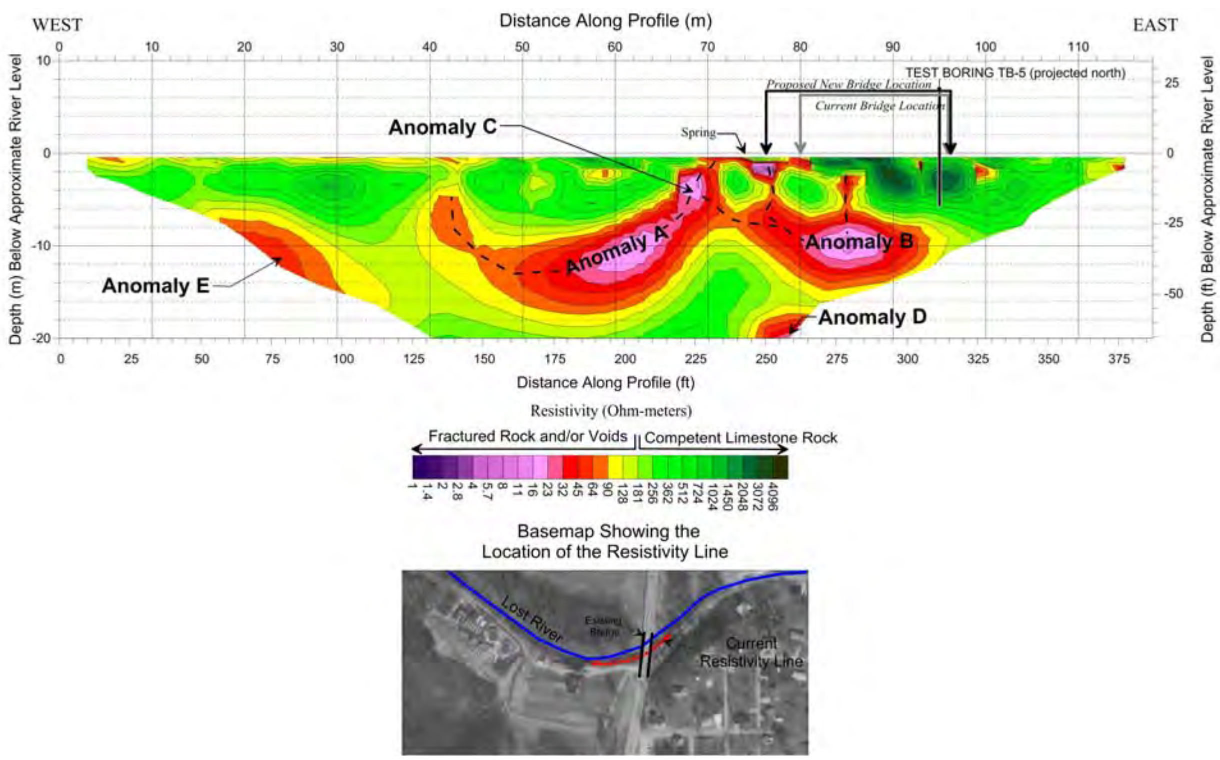


Figure 13. Lost River Bridge Resistivity Profile

materials reflecting transport into a deeper collapse feature. The moderate resistivity values, i.e.

approximately 90 to 180 ohm-meters, are usually interpreted as shallow occurrence of rock (yellow to light green in color).

One 2-dimensional resistivity profile line was collected in each of Areas 1 through 5 (Figures 5 through 9) to provide added characterization and calibration of the EM31 data. High resistivity values, i.e., approximately 180 to 2,000 ohm-meters, are generally interpreted to reflect the presence of competent limestone rock (presented as light green to green color). The opposite range of values, *i.e.*, less than 30 ohm-meters, is interpreted to be very soft, moist soil and/or water-filled, solution-enhanced fractures or voids (pink to purple). Independent and circularly-shaped regions of the very highest values of resistivity, *i.e.*, greater than 2,000 ohm-meters, are interpreted as possible air-filled voids (dark green).

Possible karst features of potential concern to this project have been noted in each of the six areas studied. Because these interpreted features must be directly observed to establish their relative concern, we recommended that these features be investigated with a direct observation method such as geotechnical drilling. We have noted the anomalies of concern both on the EM31 maps and the 2D resistivity sections (Figures 5 through 13 inclusive). A total of 23 areas have been noted, and are also summarized on Table 1.

Table 1 – Anomaly Descriptions and Recommend Drilling Locations

ID	Description
1-1	Possible water-filled void or heavily fractured rock (Anomaly 1-A) beginning at 9-11 meters depth.
1-2	Similar EM31 conductivity values and distribution as Anomaly 1-A.
2-1	Possible soil pipes at edges of sinkhole (Anomaly 2B), with possible void (Anomaly 2-A) beginning at 19-21 meters depth.
2-2	
3-1	Possible soil-filled void or heavily fractured rock (Anomaly 3-A) from 4-6 meters to 15-20 meters depth.
3-2	Possible soil-filled void or heavily fractured rock (Anomaly 3-B) from 4-6 meters to 13-17 meters depth.
3-3	Similar EM31 conductivity values and distribution as Anomaly 3-A and Anomaly 3-B.
3-4	Similar EM31 conductivity values and distribution as Anomaly 3-A and Anomaly 3-B.
4-1	Possible air-filled void (Anomaly 4-A) from 5-7 meters to 21-24 meters depth.
4-2	Possible air-filled void (Anomaly 4-B) from 8-11 meters to 20-23 meters depth.
4-3	Possible soil-filled void or heavily fractured rock (Anomaly 4-E) beginning at 8-12 meters depth.
4-4	Possible soil-filled void or heavily fractured rock (Anomaly 4-D) from 18-25 meters to 35-45 meters depth.

4-5	Possible soil-filled void or heavily fractured rock (Anomaly 4-C) beginning at 26-35 meters depth.
5-1	Possible soil-filled void or fractured rock (Anomaly 5-A) beginning at 30-40 meters depth.
5-2	Possible soil-filled void or heavily fractured rock (Anomaly 4-C) beginning at 25-30 meters depth.
5-3	Similar EM31 conductivity values and distribution as Anomaly 5-A and Anomaly 5-B.
6-1	Similar EM31 conductivity values and distribution as Anomaly 3-A and Anomaly 3-B.
6-2	Similar EM31 conductivity values and distribution as Anomaly 3-A and Anomaly 3-B.
6-3	Known sinkhole location.
6-4	Similar EM31 conductivity values and distribution as Anomaly 3-A and Anomaly 3-B.
A	Possible soil and/or water-filled void or heavily fractured rock (Anomaly 4-E) beginning at 0-5 meters depth.
B	
C	

Karst features of potential concern include primarily water-filled or air-filled voids, moist low-strength soils, and solution-enhanced fracture zones. Examples of each were observed throughout the six roadway areas and bridge locations. For example, Anomaly 1-A (Area 1, Figure 5), with a resistivity somewhat lower than the surrounding competent rock, is a possible large water-filled void given its shape, resistivity, and encasement within bedrock. Anomaly 3-A and Anomaly 3-B (Area 3, Figure 7) are examples of possible moist, low-strength soil or soil-filled voids, having very low resistivity values. These two anomalies may also be in a relatively large zone of solution-enhanced fractures, as they are surrounded by relatively low resistivity material. Anomaly 4-A and Anomaly 4-B (Area 4, Figure 8), having very high resistivity values, are examples of possible air-filled voids within competent bedrock. Area 4 also contains several possible soil-filled or water-filled voids (Anomaly 4-C, Anomaly 4-D, and Anomaly 4-E), which may indicate the possible presence of a cave system. In identifying potential karst features of concern in each area, both the EM31 conductivity/resistivity maps and the 2-dimensional resistivity profiles were considered and interpreted together.

SUMMARY AND CONCLUSIONS

It is commonly known by geotechnical and structural engineers that building on top of karst-prone limestone can prove disastrous if potentially hazardous features are not recognized prior to final design and construction. For the SR 56 Rehabilitation Project, the geophysical consultant was confronted with the challenge of limited resources and a multi-mile long stretch of roadway (as well as three bridge locations) to characterize the subsurface geology. A solution was required which conformed to these restrictions, while still providing the lead engineer with an acceptable level of assurance that a reasonable effort had been made to detect critical karst features.

To provide this assurance to the client, a combination of terrain conductivity/resistivity mapping in conjunction with localized 2D-ERI were implemented on this project. The resulting data were synthesized with regional data to develop a conceptual geologic model that was used as a tool to aid the company and the structural engineer responsible for the roadway rehabilitation project.

In conclusion, several key observations were provided to the client regarding the geophysical survey area. First, the site is indicative of a typical highly-developed, karst terrain, with numerous sinkholes, an artesian spring, and a disappearing stream observed at the surface over the survey area, as well as numerous subsurface karst features including fracture zones, soil pipes, and possible air-, soil-, and/or water-filled voids, observed as well. The position of the observed surface features correlated well with the subsurface features imaged from the conductivity and resistivity data. Also, these observed sinkholes appeared to be in active development, based on the detected weathered/solutioned zones emanating from them leading to deeper karst features. Finally, the resistivity profiles collected beneath the three bridge locations indicated that the existing footers are located in highly weathered bedrock on top of serious karst features, but that the proposed footer locations for the Mysterious Springs and Lick Creek Bridges are in substantially more competent bedrock. The profile for the Lost River Bridge indicated that the proposed bridge alignment was not in an optimal location and should be shifted further east onto more competent rock or redesigned for the possibility of loss of support or the need for substantial grouting measures. In the end, had the client chosen to characterize the subsurface in this study area with widely-spaced test borings and not from the benefit of a detailed preliminary geophysical survey, some or all of the karst features may have been missed, possibly leading to a future catastrophic roadway disaster.

REFERENCES:

Government Reports

1. Fenelon, J.M and Bobay, K.E. Hydrogeologic Atlas of Aquifers in Indiana, USGS, Water Resources Investigations Report 92-4142, 1994, pp.6-9.

2. Wingard, R.C. Soil Survey of Orange County Indiana, USDA Report, 1984, pp. 2.

Conference Proceedings

3. Ahmed, S. and Carpenter, P.J. Integrating Electrical Geophysical Surveys and Subsurface Probing to Locate Karst Aquifer Recharge Features, Northern Illinois. *Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems*, *Denver*, *Colorado*, 2001, Paper ASP-1.

4. Byer, G. B., Mundell, J. A. and Vanderlaan, J. H.M. A Geotechnical/Geophysical Evaluation of Karst Limestone Sites - A Case History. *Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems*, Las Vegas, Nevada, 2002, Paper 13CAV2.

That "Sinking" Feeling: What's Next for Twin Bridges on Extreme Karst

Kerry W. Petrasic, P.E.
Chief Geotechnical Engineer
PA Department of Transportation
Bureau of Construction and Materials
81 Lab Lane
Harrisburg PA 17110-2543
717-787-4319
kpetrasic@state.pa.us

Donald V. Gaffney, P.G.
Geotechnical Manager
Michael Baker Jr., Inc.
4301 Dutch Ridge Rd
Beaver, PA 15009
724-495-4254
dgaffney@mbakercorp.com

Ryan S. Tinsley, P.G.
Senior Engineering Geologist
Michael Baker Jr., Inc.
1801 Bayberry Court
Richmond, VA 23229
(804) 287-3183
rtinsley@mbakercorp.com

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The authors would like to acknowledge that the work described has been performed by many companies and agencies over a period of more than ten years. For additional background information on this project, see:

"Extreme Karst: Investigation at PA State Route 33"
By
Kerry W. Petrasic, P.E.
Chief Geotechnical Engineer
PA Department of Transportation
Bureau of Construction and Materials
81 Lab Lane
Harrisburg PA 17110-2543
717-787-4319
kpetrasic@state.pa.us

Presented at the 58th Highway Geology Symposium
Pocono Manor, PA, October 16-18, 2007

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

Local residents and state agencies in Eastern Pennsylvania are familiar with "That Sinking Feeling," as historically, the Epler and Jacksonburg Formations and their associated sinkholes have impacted our transportation network. State Route 33, and its twin bridges over Bushkill Creek in Northampton County, PA, has been plagued by a highly active and deep "Extreme" karst. Replacement single span bridges and approach embankments have experienced significant documented differential settlement and lateral displacement since PennDOT began periodic monitoring in the form of traditional surveys and inclinometers shortly after construction in 2004. Numerous geotechnical, geophysical, and hydrologic investigations over the last 8 years defined the extreme (greater than 400 feet deep) karst and contributing factors in the vicinity of the S.R. 33 Bridges. A plethora of data has been collected to better understand this extreme karst system.

This paper presents recent investigations and refined understanding of the subsurface conditions, contributing factors, and the apparent shifting uncertainties after each investigation as more information became available. The apparent shifting uncertainties created challenges for presenting and discussing this data with individuals outside the geotechnical discipline. After compilation and evaluation of the extensive subsurface data, conceptual "What's Next" alternatives were developed for the S.R. 33 twin bridges. These alternatives are being evaluated to develop practical solutions that minimize future karst impacts on the S.R. 33 bridges.

INTRODUCTION

State Route 0033 (S.R. 33) is a four-lane divided highway located in eastern Pennsylvania in Northampton County and PennDOT District 5-0. Between Stockertown and Tatamy, Pennsylvania, S.R. 33 is currently carried over Bushkill Creek by twin single-span bridges (See Figures 1 and 2). S.R. 2017 (Bushkill Street) is approximately 0.10 mile to the east and the Hercules Limestone Quarry is located approximately a half of mile to the west of the S.R. 33 bridges.

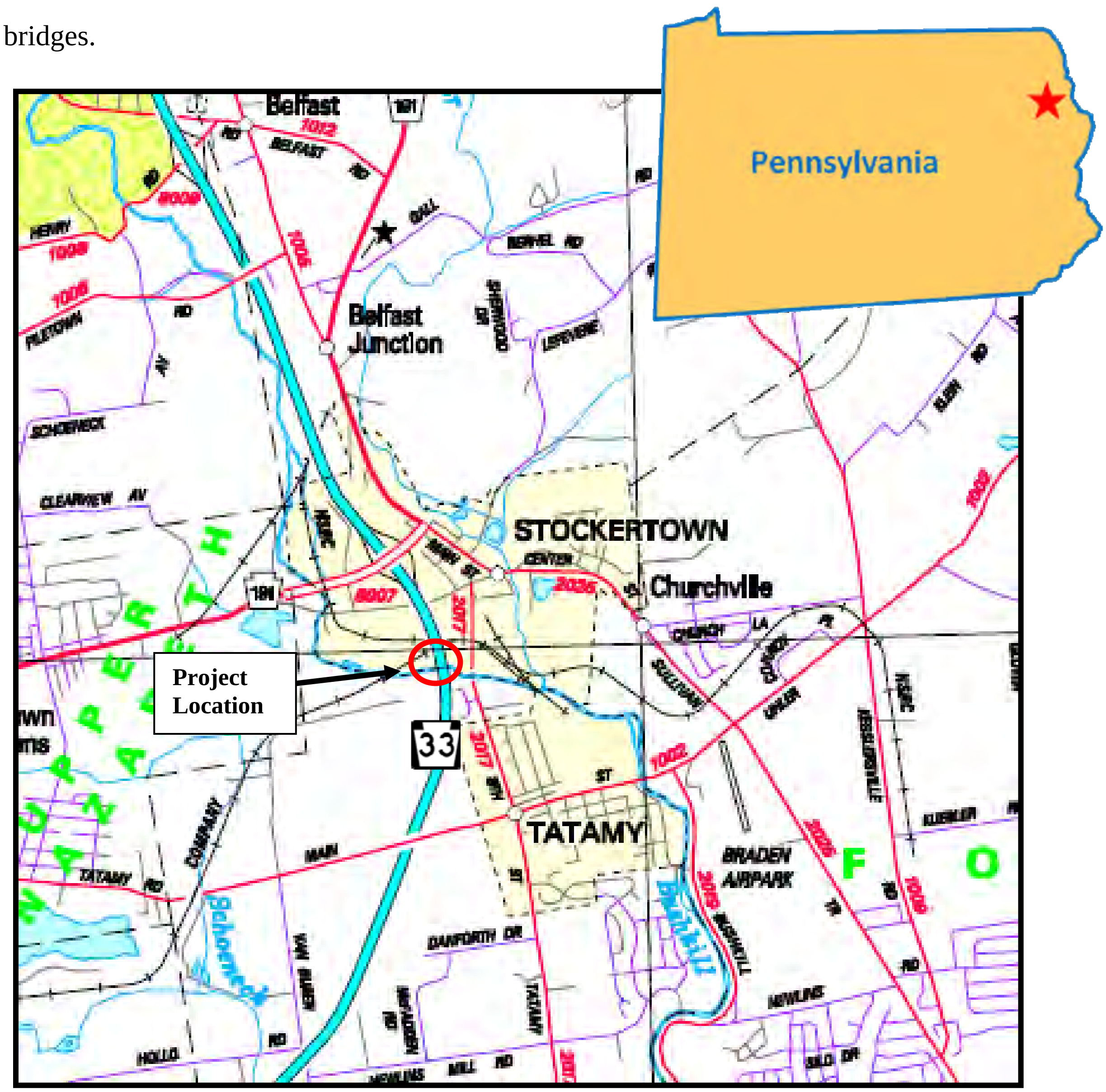


Figure 1: Project Location

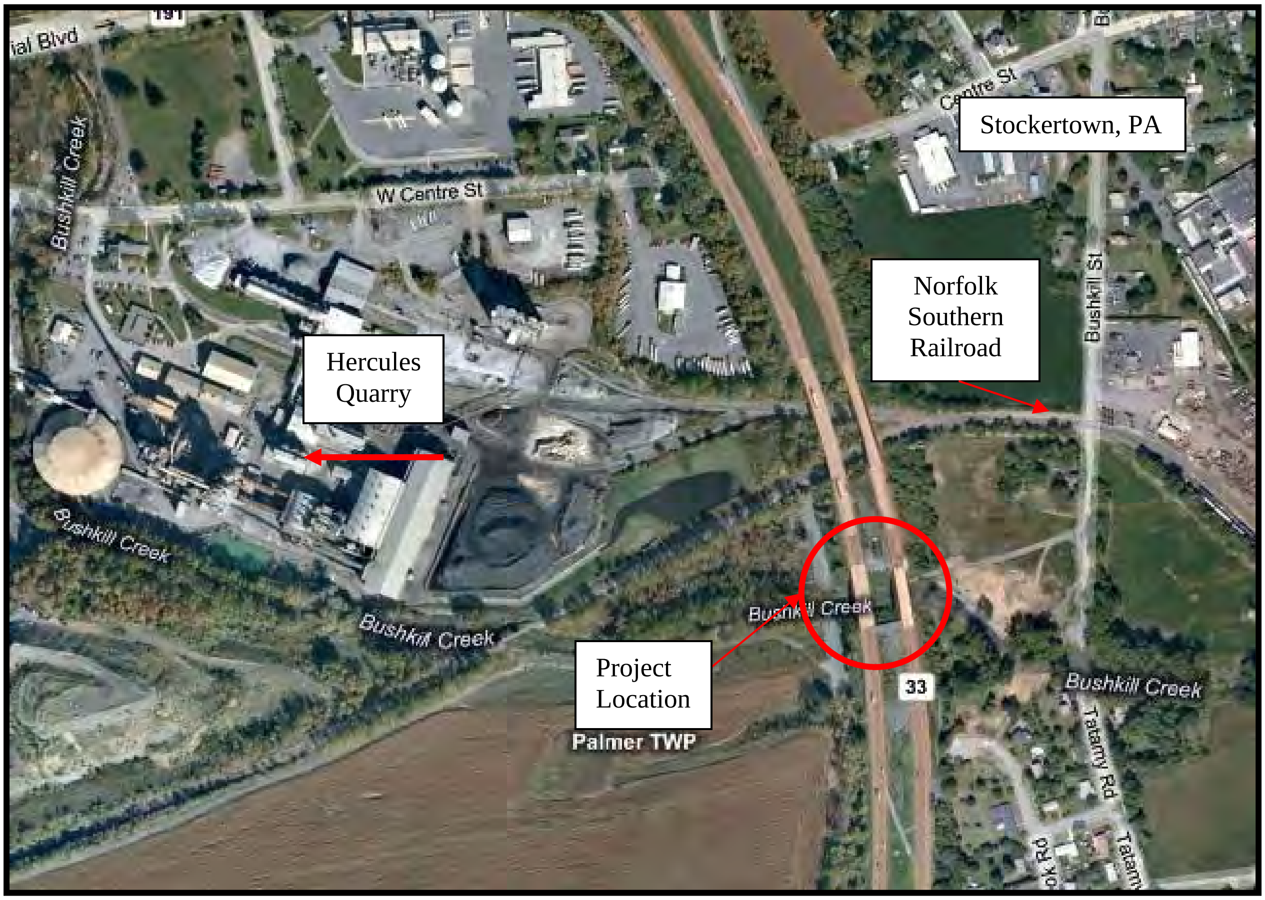


Figure 2: Project Aerial

The original S.R. 33 bridges were constructed in the late 1960's. According to as-built plans, the original structures were 3-span prestressed concrete I-beam bridges. The abutments were supported by vertical and battered H-piles. The piers were supported by spread footings on very dense gravel of glacial till origin.

During the mid to late 1990's, a significant drought plagued eastern Pennsylvania. The drought came to an end with Hurricane Floyd in September 1999. During the summer of 2000, sinkhole activity increased in the vicinity of the S.R. 33 bridges. This sinkhole activity continued and in January of 2004, significant settlement of the southern northbound pier was observed due to an open sinkhole. Attempts were made to stabilize the existing structure without success, which resulted in the closure of the northbound structure.

In summer and fall of 2004, the S.R. 33 bridges were replaced under an emergency response contract. According to as-built plans, the current structures consist of single span pre-stressed concrete I-beam bridges. The existing northern abutments of the northbound and southbound bridges were built just south of the old abutment locations and the existing southern abutments of the northbound and southbound bridges were constructed just north of the old abutment locations. This resulted in spans lengths of 156.5 feet and 162.5 feet for northbound and southbound structures, respectively, which were shorter than the original structures (3 spans totaling 180.5 feet long for NB and 174.0 feet long for SB). The replacement abutments are supported by vertical and battered 7-inch diameter micropiles.

In early fall of 2004, the newly constructed northbound structure began to show evidence of movement in the vicinity of the southern abutment and approach embankment. Hurricanes Francis and Ivan hit the area as major precipitation events during September of 2004. Sinkhole activity continued around the S.R. 33 bridges and in the vicinity of Bushkill Creek to the east. During the winter of 2005, the S.R. 33 southbound bridge began to show evidence of movement, mostly at the south abutment.

Kerry Petrasic's paper on this project entitled "Extreme Karst" was presented at the 2007 HGS, and discussed subsurface conditions in the immediate vicinity of the existing structures. It also discussed the condition of the replacement structures and mitigation options. Monitoring since then has indicated continued movement and distress to the existing structures, prompting additional investigations and development of other mitigation options including replacement of the structures. This paper considers all investigations at the site to date to evaluate the subsurface conditions and facilitate consideration of options for the structures in extreme karst.

SOILS, GEOLOGY and GROUNDWATER

Northampton County is in the east central portion of the Commonwealth. This region lies within the Great Valley Section of the Ridge and Valley Physiographic Province, comprising the lowlands south and east of the Appalachian Mountain chain. The bedrock in the site vicinity consists of folded, faulted and weathered carbonate units, resulting in a rolling topography. The bedrock unit underlying the immediate site area has been identified as the Ordovician Age Epler Formation, a limestone/dolomite sequence particularly susceptible to solution activity, yielding a karstic terrain. A geologic map is included in Figure 3 below.

The approach embankments north and south of the existing bridges are approximately 25 to 40 feet thick and consist of variable amounts of fine and coarse-grained material, typically consisting of gravel to cobble size rock. This fill is underlain by silt and fine sand alluvium and gravel to boulder size till. The thicknesses of the alluvium and glacial till are variable, but generally become thicker towards Bushkill Creek.

Sinkhole development in the vicinity of the project has generally occurred along lineal trends that mimic the primary joints and prevalent fracture traces. Figure 3 also shows the location of mapped sinkholes near the SR 33 bridges. In addition, there was a braided stream segment roughly where S.R. 33 crosses Bushkill Creek, which was channelized when S.R. 33 was first constructed. Braiding is an indication of concentrated karst activity and high surface water loss.

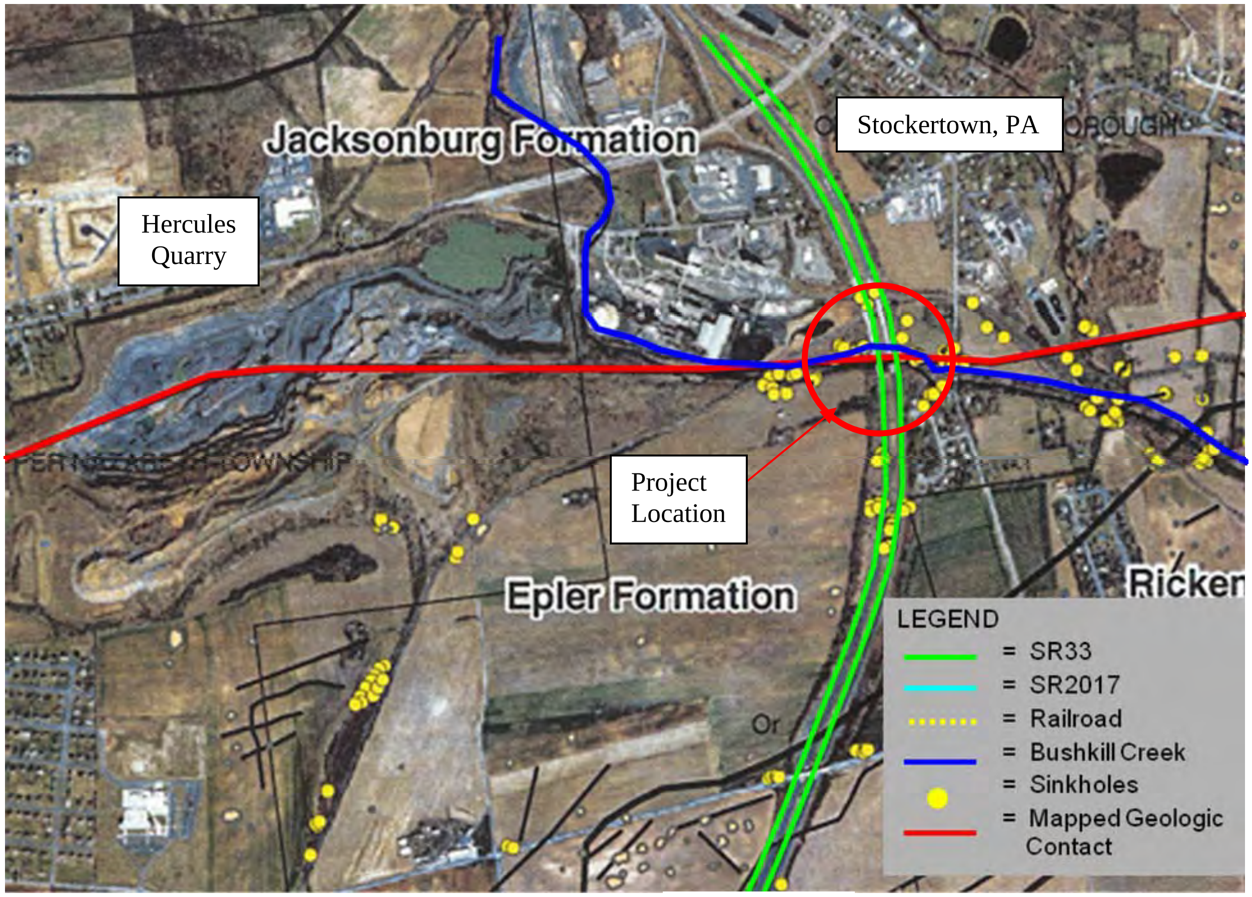


Figure 3: Geologic Map

Sinkhole development directly in and adjacent to the Bushkill Creek bed near the structures confirms a rapid, large volume loss of stream water. Surface failure in these sinkholes is reasonably assumed to be a result of the "flushing" of fine-grained soil material previously filling the solution openings in the bedrock, resulting in loss of support for the overlying material. This flushing commonly occurs following larger precipitation events such as hurricanes and may be increased as sinkholes develop in or adjacent to streams.

Dewatering operations at the nearby Hercules Cement quarry have contributed a significant water volume to the stream while simultaneously drawing down the groundwater level in the area surrounding the stream and the bridges.

EXISTING SUBSURFACE INFORMATION

Primarily over the last 8 years, a significant number of investigations by numerous consultants and State agencies have been performed to better understand the karst environment that plagues the S.R. 33 bridges. The following section is intended to be a sequential brief summary of each geotechnical, geophysical, and hydrological investigations conducted for the S.R. 33 bridges.

1965 Original Bridge Borings: The original S.R. 33 Bridges over Bushkill Creek was constructed in the 1960's. The as-built plans included twelve (12) borings. The depths of borings ranged from 29 to 60 feet.

2004 Bridge Borings: A subsurface investigation involving test borings was conducted for the replacement structures. Thirteen (13) borings were drilled for the proposed abutments of the northbound structure. Ten (10) borings were drilled for the proposed abutments of the southbound structure. The borings ranged from 40.8 to 290.8 feet deep.

2004 Bridge Construction with Micropile Foundations: The original three span bridges were removed and replaced with twin single span bridges in summer and fall of 2004. The replacement abutments were supported by vertical and battered 7-inch diameter micropiles socketed into rock with a 10-foot bond zone. The as-built plans, micropile layouts, and micropile logs were available for each structure.

2004 to 2006 Ground Survey Monitoring: Shortly after construction of the replacement northbound bridge, PennDOT began a surface survey of the S.R. 33 bridges and approach embankments to monitor settlement and lateral movement. Survey points extended approximately 225 feet south of the southern abutments and approximately 283 to 350 feet north of the northern abutments to the existing Hercules bridges over the railroad.

2004 Geophysical Investigation: In February 2004, two geophysical investigations were performed adjacent to the S.R. 33 bridges. The purpose of the investigations was to collect both electrical resistivity imaging data and seismic refraction transverses in order to assess the karst activity/surface depressions, and the depth of the soil/rock interface adjacent to the bridge structures. Both surveys were correlated with available borings.

2005 Deep Investigation Borings (Rotosonic and Air-Rotary): A very deep site investigation was conducted to better understand the subsurface conditions around the existing structures and delineate the conduit/karst system (Petrasic, 2006). The deep investigation consisted of 2 rounds of drilling, the first consisting of rotosonic/rock core borings and second consisting of air-rotary borings. The rotosonic borings were drilled to depths of 153 to 500 feet deep with an average depth of 352 feet. The air-rotary borings varied in depth from 65 to 545 feet deep with an average of 377 feet.

2005 to Present Inclinometer Data: Twelve (12) inclinometers were installed in the Rotosonic (B-series) borings of the 2005 deep investigation. PennDOT has conducted periodic readings since installation and continues to monitor the inclinometers.

2005 Hydrologic and Temperature Data: Water wells consisting of perforated plastic pipe were installed into each of the deep investigation borings to measure water parameters such as temperature, conductivity, and pH. The water readings were recorded continuously from bottom to top during fall and winter of 2005. Significant correlations were made relative to groundwater temperature variations with depth.

2005 Geophysical Investigation: In late fall and early winter of 2005, a geophysical investigation was conducted consisting of electrical resistivity imaging (ERI) and spontaneous potential (SP) data. ERI was conducted along 5 survey lines. Spontaneous Potential (SP) data was collected west of S.R. 33, both north and south of the creek.

2005-2006 DEP Stream Flow Surveys: Two stream flow surveys were conducted along Bushkill Creek by the Pennsylvania Department of Environmental Protection (PADEP) during July of 2005 and March of 2006. A creek loss of 73% to 50% was estimated between S.R. 33 and S.R. 2017 during low and high flow conditions, respectively.

2006 Brine Trace Study: During spring of 2006, a brine trace study was conducted to confirm and quantify the subsurface flow pattern. Three existing sinkholes adjacent to the creek between S.R. 33 and S.R. 2017 were selected as the injection points for the brine solution. Sixteen In-Situ Troll 9000 data loggers were installed between the injection points and the quarry to measure the brine concentration in the groundwater.

2007 Bridge Borings: With a better understanding of the karst system, a subsurface investigation was conducted in 2007 to determine the locations of more competent rock and abutment locations for possible future replacement structures. The borings were located to provide subsurface profiles along the shoulders and to determine changes in rock quality moving away from the existing abutments. The borings depths ranged from 63.5 feet to 136.5 feet deep. In general, the more competent rock was encountered in borings located further away from the existing abutments and Bushkill Creek.

2008 Bridge Borings: A supplemental subsurface investigation was conducted in 2008 to optimize the future abutment locations. The primary focus was to confirm rock quality and depth between the 2007 borings where competent rock was encountered. The borings depths ranged from 85.0 feet to 105.0 feet deep.

EVALUATION OF SUBSURFACE DATA

Two surfaces were identified to help characterize the depth and quality of the karst rock within the project site. These rock surfaces were top of epikarst and top of competent rock. For purposes of this investigation, the top of epikarst was generally taken as “Top of Rock” (depth at which rock was first encountered) and top of competent rock was generally defined as 25 feet or more of continuously cored rock having recoveries of 90% or greater without encountering soil zones and voids (fractures or cavities with an encountered length of 6-inches or more).

Contour maps were generated that incorporated data from all previous boring programs and micropile construction. Rock conditions as reported on available borings and micropile logs were evaluated to determine top of epikarst and top of competent rock. The top of epikarst, epikarst thickness, and top of competent rock contour maps were developed.

The data sources, method of data collection, and depth of data varied among the investigations. As a result, certain data were removed and some assumptions were made to develop the maps. Angled borings and battered micropiles were not included in the data set, because excessive migration during drilling caused by the highly variable karst terrain made the depth corrections and correlations questionable. Competent rock was not encountered in every boring. A majority of borings that did not encounter competent rock were within the deeper karst system, because the borings often were terminated at a depth of roughly 100 feet. The maps can be used to see trends with respect to the epikarst and competent rock across the project site, but competent rock depths and epikarst thicknesses may vary significantly from the interpolated surfaces shown on the maps.

SITE SPECIFIC CONCLUSIONS

Since before the original S.R. 33 construction, sinkholes have developed in the vicinity of the S.R. 33 bridges and along Bushkill Creek, and they appear to be triggered by large precipitation events and pumping operations of the Hercules Quarry. Since the construction of the existing replacement bridges in mid-late 2004, sinkholes have continued to develop periodically within the S.R. 33 right-of-way (ROW) and potentially are impacting the existing structures. These sinkholes have been repaired by PennDOT Maintenance. Hercules Quarry has started monitoring sinkhole activity along Bushkill Creek, and they have repaired recent sinkholes outside of the S.R. 33 ROW.

There appears to be a direct connection between the creek loss to the east, alignment of the deeper karst system and the quarry location and pumping operations. The primary flow of the subsurface conduit is to the west towards the quarry. The water of Bushkill Creek is on a continuous surface to subsurface cycle utilizing the sinkholes to the east and quarry to the west.

The karst system in the vicinity of the bridges extends to depths greater than 450 feet and deepening trend towards the west. In general, the epikarst thickness decreased and depth to competent rock became shallower north of the north abutments and south of southern abutments or away from Bushkill Creek. The epikarst generally becomes wider and deeper from east to west. The deeper karst system mimics a northeast-southwest trending trough feature that appears to encompass the existing NB Abutment 2, SB Abutment 1 and SB

Abutment 2 as indicated by the zones of thicker epikarst and deeper competent rock. The shallower competent rock and decreased epikarst thickness to the north and south of the existing abutments or away from Bushkill Creek also appear to follow a general northeast-southwest trend.

BUSHKILL CREEK CROSSING ALTERNATIVES

There are many factors to be considered in the development of feasible crossing alternatives for S.R. 33 over Bushkill Creek. Some of these include:

- Impacts to surface drainage and subsurface flow that will influence future sinkhole development within the S.R. 33 project, the adjacent Brookwood Residential Community and other property

- On-going subsurface erosion caused by the hydraulic conduit within the deeper karst system that is being fed by and underlies Bushkill Creek

- Environmental impacts to Bushkill Creek, and the local and regional groundwater table

- The existing differential settlements and lateral movement of the current abutments, and the existing stresses imposed on the pre-stressed concrete beams and decks.

- Potential changes in lateral stresses on existing superstructures and substructures, and roadway due to continual movement of surrounding approach embankment fills

- The depth of the karst system and the difficult foundation construction as a result of subsurface conditions in the vicinity of the existing structures.

- Site constraints such as existing line and grade, roadway alignment, and design flow for Bushkill Creek.

- Constructibility of each alternative and impacts to the traveling public

The following conceptual alternatives were developed to reduce current risk associated with the above factors for the Department. It is important to recognize that any alternative that addresses one or more of above factors could have negative consequences on others.

Alternative 1 – Preservation of Existing Structures: Preservation of the existing structures would involve constructing redundant foundations to minimize settlement or lateral movement towards Bushkill Creek. This alternative would include constructing pile cap extensions or outriggers from the existing abutments supported by large diameter drilled caissons. The alternative would also include additional bracing to reinforce the connection between the superstructure and the abutment extensions.

Alternative 2 – Replacement Structures with New Abutment Locations: This alternative would involve a complete replacement of the existing structures with longer single span twin bridges. The new abutments would be located north and south of the existing abutments away from the deeper karst system and over shallower competent rock. Estimated span lengths would likely be between 330 and 400 feet, which are more than double the existing spans. The new abutments would be supported by micropiles or caissons. The foundations would be designed to withstand a loss of support of up to 35 percent, considered as a localized loss in karst (PennDOT DM4, Section 10.7.3.2.4.) and a reduced lateral support.

Alternative 3 – Pipe System and Embankment: This alternative would involve complete removal of the existing superstructures and replacing them with a pipe culvert system and embankment fill. The substructures would remain in place and would be buried with embankment fill. The pipe culvert system would consist of a series of steel pipes adjacent to one another to span Bushkill Creek. The multiple pipe system was considered because the system would be flexible and have built-in redundancy.

The embankment would be constructed out of lightweight structural fill to reduce loads and associated settlement of the multiple pipe system and existing overburden soils. The alternative would also include strategically placed geosynthetic reinforcement for added strength and flexibility to prevent impacts of settlement due to sinkhole development. Reinforcement would also be considered to steepen the side slopes (RSS) of the fill and reduce the overall pipe length. The embankment fill would have to be protected against scour with either riprap or armored with articulated concrete block (ACB) mat.

Stream lining could be considered as an additional option to this alternative. Stream lining would extend from the existing lined portion of the creek 400 feet to the west to the east end of the proposed pipes.

Alternative 4 – Modifying the Deeper Karst System: The deeper karst system includes epikarst to a depth greater than 450 feet, and a subsurface hydrologic conduit with velocities that range from 5 to 19 feet per minute with an estimated depth of 300 feet at the zone of highest flow. The epikarst and conduit appear to be more developed and deeper to the west. This deep karst system is also within the zone of influence of the Hercules Quarry pumping operation. Modifications to the system would involve changes to the flow regime above (Bushkill Creek) and below (subsurface conduit).

This alternative would involve injection wells to the west of the SR 33 SB roadway. The intent of the wells would be to introduce the Bushkill Creek flow into the hydrologic conduit sooner and reduce the flow under S.R. 33. A grout curtain could be a possibility west of the S.R. 33 SB roadway to provide a cut-off or reduce the conduit's western flow and provide more stable or equilibrium condition of groundwater conditions to the east. However, the conclusions reached previously (Petrasic, 2006) still apply to this alternative, which make it less feasible due to costs, constructability, and unquantifiable/unknown impacts of modifying the karst system.

RECOMMENDATIONS FOR FURTHER INVESTIGATION

We recommend proceeding with the following general recommendations regardless of the alternative(s) to be considered further:

- Preliminary costs estimates and risk assessment of each alternative to be considered further should be developed to help determine the financial feasibility. The investigation recommendations below will help refine the parameters that will control the cost of each alternative
- Due to the historic sinkhole activity, consider a real time or periodic monitoring system to provide an early warning of a possible catastrophic failure. Some applicable systems could include, but not limited to TDR, GPR, total robotic stations, or satellite imagery comparison. A study could be conducted to determine the most feasible (cost and applicable) system for this application.

- Continue periodic bridge inspections and obtain an updated bridge data.
- Continue ground surveys and inclinometer readings on a regular basis and incorporate data into the future design efforts. Additional readings or data should be collected following significant rainfall events.
- In order for any of the above alternatives to adequately address the ongoing approach embankment deflections, a better understanding is needed. A more thorough search for available ground surface survey and inclinometer data from 2004 to 2010 would help determine the locations and magnitudes of deflection across the site. Also, consider a satellite imagery analysis to determine the settlement that has occurred over the same time period and to compare it with the existing survey and inclinometer data.

Use of Ground Penetrating Radar for Void Detection and Hydro-Geochemical Water Testing Results at the Cumberland Gap Tunnel

Brad Rister P.E.

University of Kentucky

Kentucky Transportation Center

176 Raymond Bldg.

Lexington, KY 40506

(859)-257-7331

brister@engr.uky.edu

James Dinger Ph.D.

Kentucky Geologic Survey

302 Mining and Mineral Resources Bldg.

Lexington, KY 40506

(859)-323-0523

dinger@uky.edu

Prepared for the 62nd Highway Geology Symposium, July, 2011

Acknowledgements

The author(s) would like to thank the individuals/entities for their contributions in the work described:

Mr. Chuck Knowles--Kentucky Transportation Cabinet

Mr. Greg Duncan--Tennessee Department of Transportation

Mr. Don Miller--Federal Highway Administration (Eastern Federal Lands Division)

Mr. Jose Sepulveda--Federal Highway Administration (Kentucky Division)

Mr. Mark Woods--National Park Service

Mr. John Burke--Cumberland Gap Tunnel Authority

Disclaimer

Statements and views presented in this paper are strictly those of the author(s), and do not necessarily reflect positions held by their affiliations, the Highway Geology Symposium (HGS), or others acknowledged above. The mention of trade names for commercial products does not imply the approval or endorsement by HGS.

Copyright Notice

Copyright © 2011 Highway Geology Symposium (HGS)

All Rights Reserved. Printed in the United States of America. No part of this publication may be reproduced or copied in any form or by any means – graphic, electronic, or mechanical, including photocopying, taping, or information storage and retrieval systems – without prior written permission of the HGS. This excludes the original author(s).

ABSTRACT

Both Ground Penetrating Radar (GPR) surveys and Hydro-Geochemical Water Testing (HGWT) have been performed at the Cumberland Gap Tunnel to determine why the reinforced concrete pavement has settled in various areas throughout both tunnels. To date, approximately 7,300 total square feet of pavement surface has voids beneath it that range from 0.05 to 40 inches in depth. Both GPR and HGWT results indicate that approximately 0.75 to 1.5 cubic yards of limestone sub-base material leaves the tunnel in solution form on a monthly basis. Furthermore, HGWT results indicate that the ground water beneath the tunnels is calcium deficient. Thus allowing the water to dissolve the limestone sub-base. Approximately 500,000 to 1 million gallons of water flows through the tunnel's ground water collection system on a daily basis.

Attempts to fix/shore-up the settled pavement areas were performed in 2002, 2007, and 2008. In 2002, UreTek foam was placed beneath approximately 2000 square feet of settled pavement for shoring purposes. In 2007, approximately 150 lineal feet of both pavement and backfill were removed and replaced with inert granite backfill material and a new reinforced concrete pavement. In 2008, approximately 51 cubic yards of cement grout material was placed beneath approximately 7,400 total square feet of settled pavement for shoring purposes.

There are several strategies outlined in this report to address both short-term and long-term remediation. However, there are certain strategies that may prevail over others. It is proposed that grout material should be placed beneath the pavement structure, at an estimated cost of $50,000 to $100,000/year, as a short term assurance measure. It is proposed that approximately 2,800 lineal feet of pavement and backfill material be removed in both tunnels and replaced with an inert granite backfill and a new 10 inch reinforced concrete pavement be installed for a long-term remediation (estimated costs $10,000,000).

INTRODUCTION

This is a summary report of the pavement settlement issues (distressed areas) and hydro-geochemistry (water quality testing) results from the recent Cumberland Gap Tunnel pavement inspection project. This report will briefly highlight the following:

a. History of the distresses incurred to the pavement structure
b. Quantify the settlement areas (void areas)
c. Explain the hydro-geochemical water testing results
d. Discuss the traffic impacts in the event that the tunnel would need to be closed for emergency repairs
e. Discuss future traffic impacts
f. Offer suggestions for short-term remediation efforts (maintenance)
g. Offer recommendations for long-term remediation efforts for the settled areas
h. Provide preliminary costs estimates for both the short and long term repair recommendations

BACKGROUND

The Cumberland Gap tunnel is a twin-bore-four-lane mountain tunnel that carries US 25E from southeastern Kentucky into Tennessee. It resides within the Cumberland Gap National Park, and carries an average annual daily traffic (AADT) volume of 22,500 vehicles bi-directionally per day. Approximately ten percent of the AADT volume is trucks, which predominately transport fuel and coal between the two states.

Both the design and construction oversight for the tunnel was performed under the direction of Eastern Federal Lands, a division of the Federal Highway Administration. The tunnel was completed in 1996 with an approximate total project cost of 260 million dollars.

Currently, the tunnel is maintained and operated by the Cumberland Gap Tunnel Authority (CGTA). The CGTA performs its duties as an over-site agency for the maintenance and operation of the tunnel under a joint contract with both the Kentucky Transportation Cabinet and the Tennessee Department of Transportation.

HISTORY OF DISTRESS

Distresses to the continuous-reinforced-concrete-pavement (CRCP) were first noticed in 2001 by the CGTA. These distresses consisted of multiple areas starting to settle in the southbound tunnel between stations 119+50 and 140+50. The magnitude of the pavement settlement was approximately 1-3 inches at that time. In efforts to bring the pavement structure back into proper elevation, it was suggested that an expansive foam material be installed beneath the pavement to lift the pavement back into proper elevation in the settled areas. This process worked with limited success. The foam material only filled the void space between the concrete pavement and the aggregate sub-base. Therefore it was unable to lift the pavement into proper elevation.

In 2005, the Kentucky Transportation Center (KTC) conducted an experimental research project using ground penetrating radar (GPR) to determine if there were voids beneath the CRCP pavement in the distressed areas. This inspection determined that approximately 6,000 square feet of pavement surface between both the north and southbound tunnels had some type of void beneath it. These voids ranged from 2 to 40 inches in depth. Figure 1 below displays a 40 inch deep void located beneath the left driving lane of the southbound tunnel at approximately station 128+90. It can be inferred that the concrete pavement is essentially performing as a bridge in these void locations. Only because reinforcing steel was placed inside the concrete, is the pavement structure able to be in-service without complete failure today. Structural loading calculations indicate that the concrete pavement should only be able to span 6 feet before permanent deformation of the steel takes place. As seen in Appendix A, some of the void areas are spanning across both lanes (30 feet wide) and extending 1 to 70 feet in length.

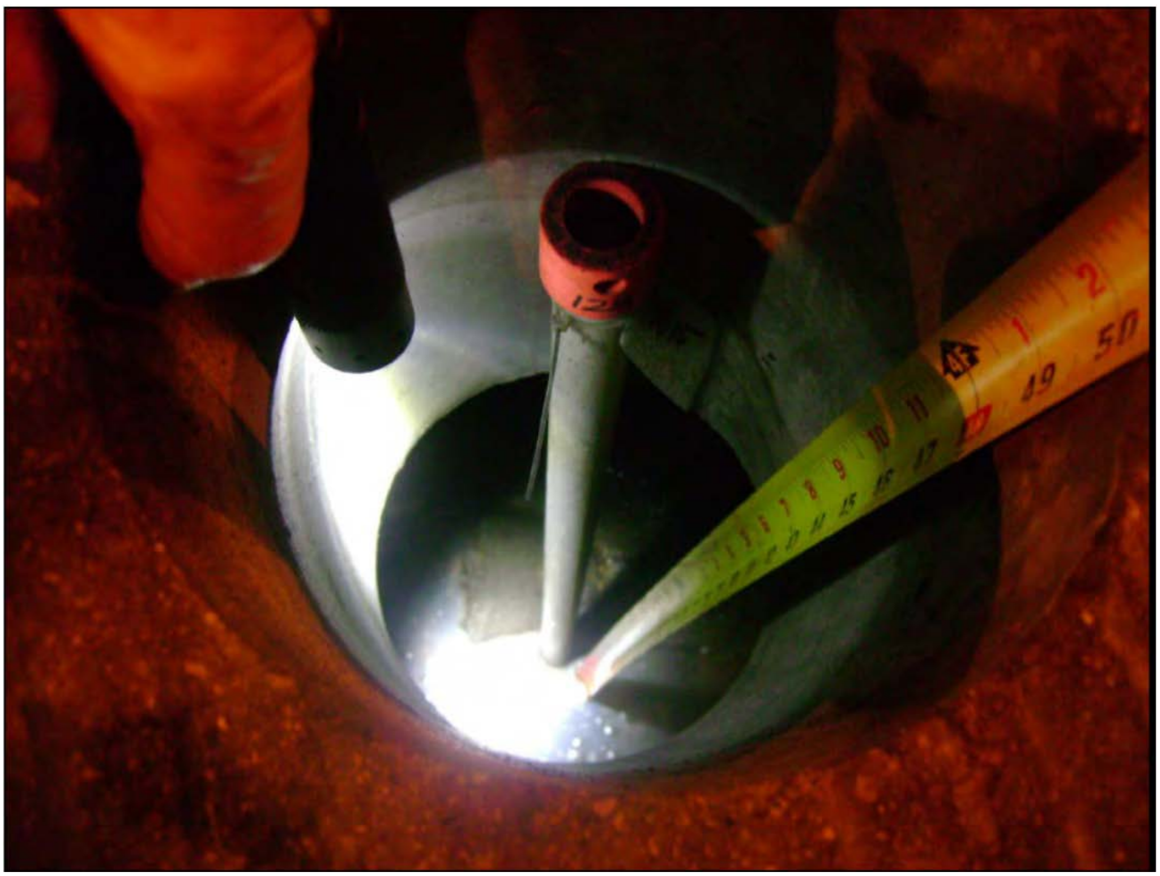


Figure 1: Forty-inch void beneath concrete pavement

In April of 2007, a technical group was formed to study the pavement settlement issues at the Cumberland Gap Tunnel. This group consisted of representatives from the following: Kentucky

Transportation Cabinet (KYTC), Tennessee Department of Transportation (TDOT), Federal Highway Administration-Kentucky Division (FHWA), Federal Highway Administration-Eastern Federal Lands Division, National Park Service (NPS)-Cumberland Gap National Park, Cumberland Gap Tunnel Authority (CGTA), Kentucky Transportation Center (KTC), and the Kentucky Geological Survey (KGS). It was determined in that meeting that a significant amount of settlement was taking place in the southbound tunnel from stations 122+24 to 123+41 and that an investigative repair would be necessary to eliminate a potential pavement collapse and to gain a better understanding of the mechanisms which may have been causing this distress.

A new discovery was determined during this investigative repair. It was found that the ground water inflow into the tunnel backfill material beneath the concrete pavement was aggressive to calcite. The tunnel backfill material is a limestone material (approximately #57 size aggregate) that is rich in calcium. Figure 2 displays the ground water inflow into the repaired area. Approximately 500,000 to 1.2 million gallons of ground water flows beneath the tunnels on any given day depending on the rainfall events.

Figure 2: Water inflow into the repaired area

This limestone backfill material ranges from 4-6 feet in depth by design throughout both tunnels. The technical advisory group determined that the appropriate repair would be to replace the excavated material with an inert granite backfill material. The backfill material consisted of a number 57 size aggregate, overlaid by a six inch layer of dense-graded-aggregate (DGA) separated by a geo-grid fabric. Next, a new 10 inch CRCP pavement was installed (Figure 3).

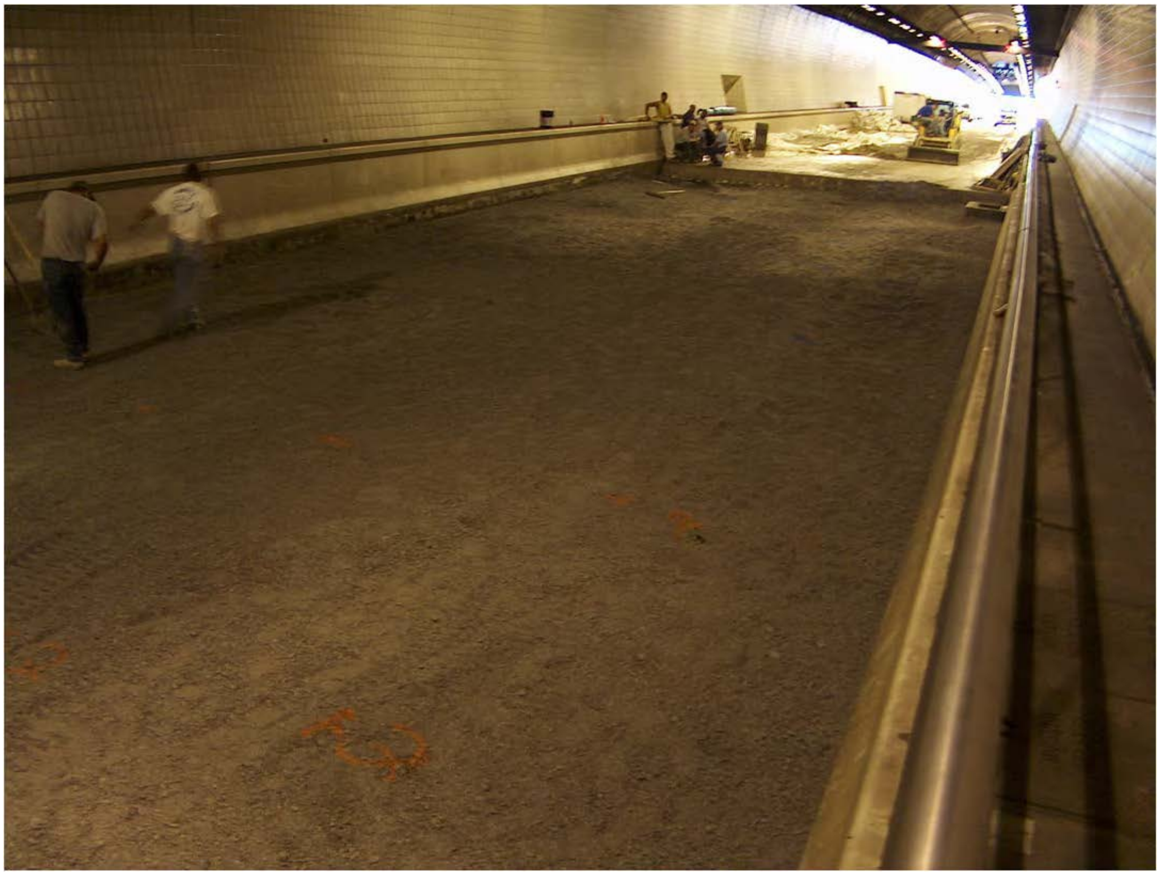


Figure 3: Repair area with granite backfill and DGA prior to concrete pavement placement

A more detailed summary of the hydro-geochemical water testing results will be provided in the hydro-geochemistry section of this report.

Another discovery made during the investigative repair was that the groundwater collection system is elevated approximately 2 to 3 feet above the invert of the tunnel (Figure 4). The ground water collection pipe can be seen in Figure 4 as the green pipe on the left side of photo.

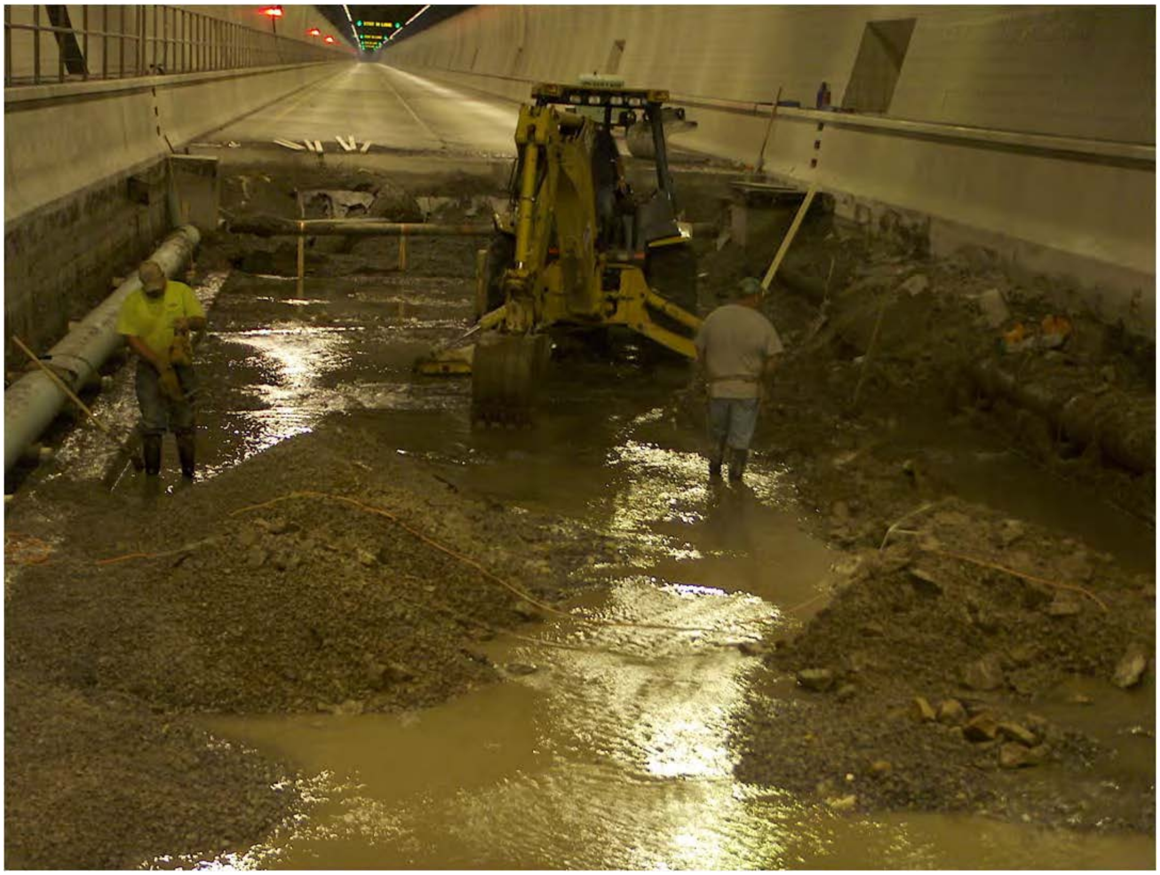


Figure 4: Groundwater collection pipe location in relation to tunnel invert

For convenience of construction, the elevation of the groundwater collection system was constructed higher than the invert of the tunnel. Thus, the limestone backfill material throughout the tunnel was constructed to act as a natural drainage structure for the ground water in-flow to pass through. It has been presumed, after research of both design and construction documents, that no water test were conducted to measure calcium deficiency during either the design or construction phase of the tunnel.

In the spring of 2008, as a precautionary measure to avoid further settlement, both the KYTC and TDOT decided that the other void areas (approximately 7,460 square feet) needed to be filled with cementatious grout. Approximately 51 cubic yards of cement grout was placed into all known void areas at that time. As of August 2009, approximately 90% of the voids grouted in the spring of 2008 have reappeared and the repaired area with the granite backfill appears to be unchanged and performing well.

QUANTIFICATION OF SETTLEMENT (DISTRESSED AREAS)

The distressed areas started to appear in the southbound tunnel in 2001, just five years after completion of construction. The distress was first noticed by the Cumberland Gap Tunnel Authority during routine maintenance. In an attempt to monitor the progression of void growth, the Cumberland Gap Tunnel Authority asked the Kentucky Transportation Cabinet to involve the Kentucky Transportation Center in its use of its falling-weight deflectormeter (FWD) and ground penetrating radar (GPR) equipment to monitor and evaluate void growth. As mentioned above, approximately 7,300 square feet of void space are present today. However, the voids are not as deep as they were in 2005 because of the grouting that took place in the spring of 2008. Preliminary GPR results obtained from the latest survey performed in August 2009 indicate that the void depths range from 0.5 to 6 inches deep depending on their location in respect to the hydro-geochemical data (Appendix A). Figure 5 outlines the time line of combined void growth for both tunnels. The green bar indicates the quantity of voids that were removed during the investigative repair in the summer of 2007. This figure demonstrates that even with the reduction in total void surface area of 1,419 square feet in August 2007, the total void space in December 2007 had surpassed the quantity from January 2007.

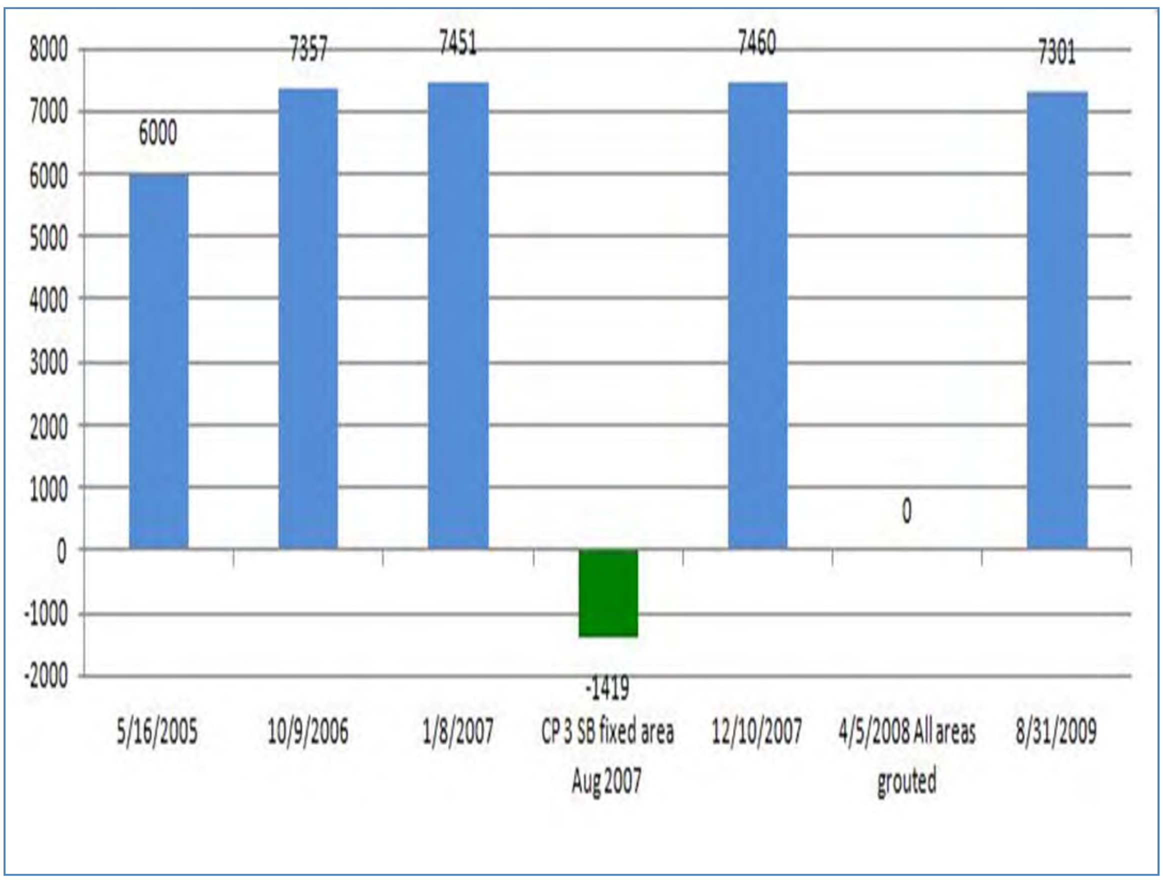


Figure 5: Total square feet of void surface areas

HYDRO-GEOCHEMISTRY RESULTS

After the investigative repair was completed in 2007, the technical group decided that a much broader hydro-geochemical water-chemistry testing study was needed. This study was conducted to validate the extent of the calcium deficient water entering into the tunnel. Laboratory tests have confirmed that water samples that have a calcium deficiency less than 0.10 will start to dissolve limestone material.

Approximately 120 water-sampling wells were drilled and instrumented in both tunnels during the fall of 2008. As shown in Figure 6, the geological composition of the rock material from stations 140+50 to 160+00 (Tennessee portal) consists of limestone composition while the composition from the Kentucky portal to station 140+50 is sandstone.

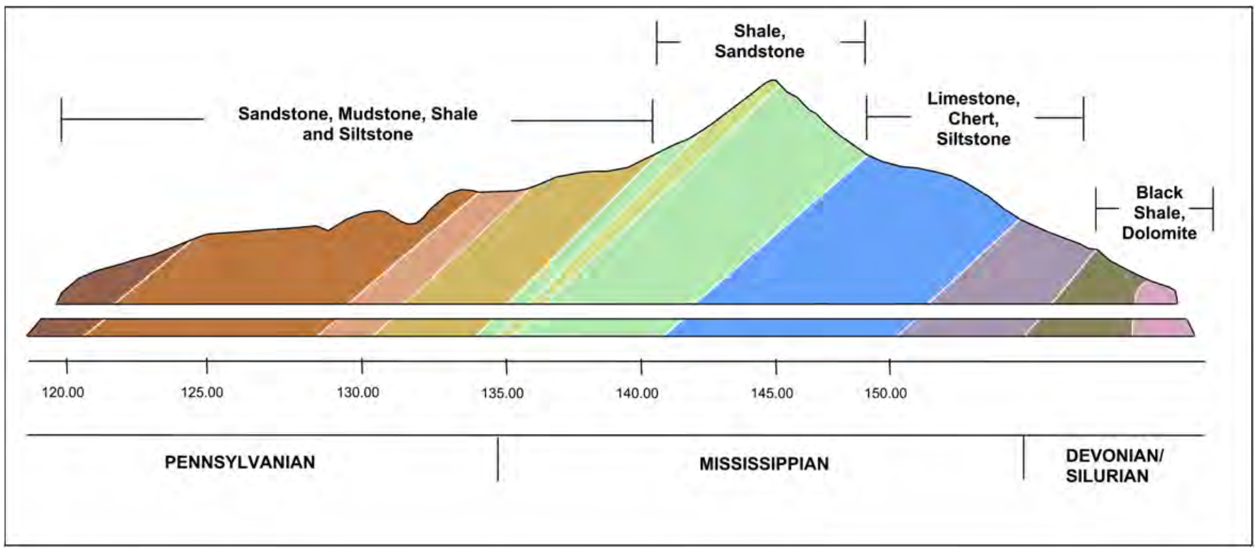


Figure 6: Geologic map of Cumberland Gap Tunnel

Water samples obtained between stations 140+50 to 160+00 appear to be chemically balanced with respect to calcite. Thus, there was no noticeable chemical breakdown noted in the limestone backfill beneath the pavement in this area. The water is apparently naturally aggressive in this location and is using the native formation of limestone to balance itself with respect to calcite. This gives rational to the presence of the cave systems located in these areas. However, the remainder of the tunnel has a different geological composition (i.e. siltstones, mudstones, sandstones, etc.) that is incapable of chemically balancing the water with respect to calcite before entering the limestone road-base aggregate. In these locations (stations 119+50 to 140+50) water samples collected and analyzed by KGS appear to be aggressive with respects to calcite. Figure 7 summarizes the hydro-geochemical water- testing data. Figure 7 demonstrates that 84% of the southbound and 77% of the northbound water samples are aggressive to calcite.

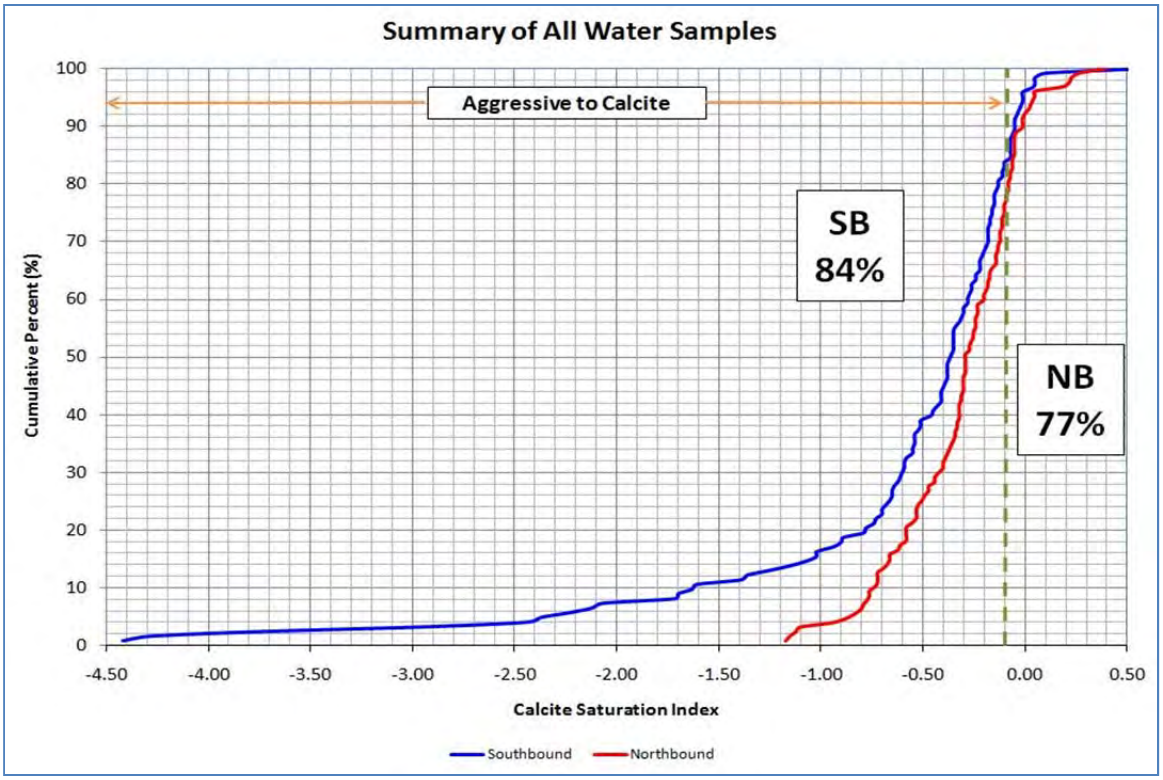


Figure 7: Summary of Water Samples

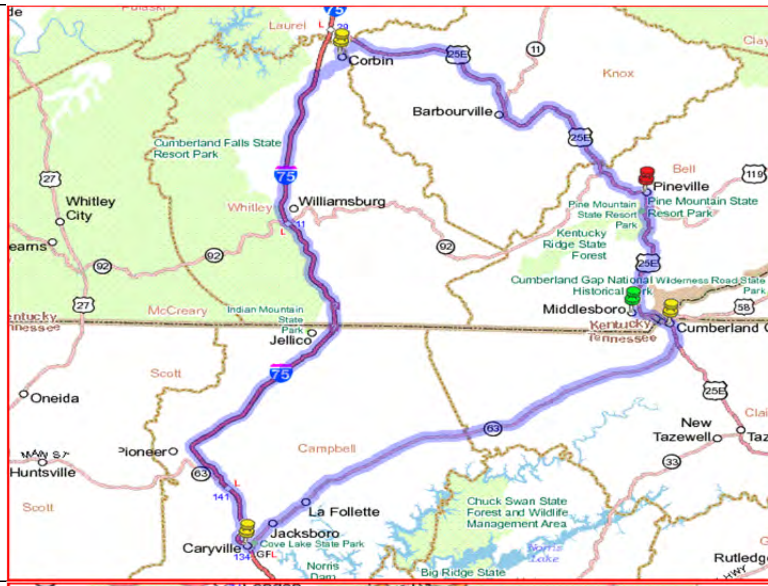
Therefore, the majority of the ground-water samples between stations 119+50 and 140+50 have the potential to chemically dissolve the limestone aggregate backfill. This material then exits the tunnel in solution through the ground water collection system on a continual basis. Preliminary results of the amount of material leaving the tunnel in solution have been quantitatively compared between mass-flux models, ground penetrating radar results, and visional calculations during the grouting process. These preliminary results estimate that approximately 0.75 to 1.25 cubic yards of limestone material are being removed in solution from beneath the concrete pavement on a daily basis. This also translates into approximately 70 to 150 square feet of new void space opening up beneath the pavement surface on a monthly basis.

TRAFFIC IMPACTS FOR DIVERTED TRAFFIC

Considerations were made for the impacts imposed on the traveling public (approximately 22,500 AADT) during the repair conducted in 2007. These considerations for complete traffic diversion can also be used to guide future repairs and or emergency maintenance repairs (Table 1). All dollar values have been adjusted using the 2007 consumer price index published by the Bureau of Labor Statistics.

Table 1: Diversion routes and daily user costs 2007 dollars

<table>
<tr>
<td>Via La Follette, TN and Williamsburg, Ky 130 miles, 3 hrs 6 min, <u>User Cost -- $1,197,096 (daily)</u></td>
<td>
</td>
</tr>
<tr>
<td>Via Pennington Gap, VA and Harlan, Ky 110 miles, 2 hrs 50 min <u>User Cost -- $1,071,085 (daily)</u></td>
<td>
</td>
</tr>
</table>

Via La Follette, TN and Corbin, KY 140 miles, 3 hrs <u>User Cost --</u> <u>$1,134,091(daily)</u>	
Alternate Long Haul Interstate Route, 30 additional miles, 30 minutes <u>User Cost -- $189,015</u> <u>(daily)</u>	

However, no complete diversion of traffic was necessary to the general population of traffic during the repair. Only wide load cargo vehicles were subject to the complete diversion routes as mentioned above. During the construction phase of the repaired area, the southbound traffic was diverted over to the northbound tunnel, with traffic running bidirectional in the northbound tunnel. No noticeable delays in traffic were experienced in the northbound tunnel despite the reduced travel speed of 25 mph.

Figure 8 displays the maximum work zone capacity of 1,300 vehicles-per-hour-per-lane (vphpl), ref. 2001 Highway Capacity Manual that can be processed in a single lane on an hourly basis without backups. The hourly traffic distribution for the 22,500 ADT can also be found in Figure 8, which identifies that the traffic would have to increase by an approximate 30 percent before backups would occur.

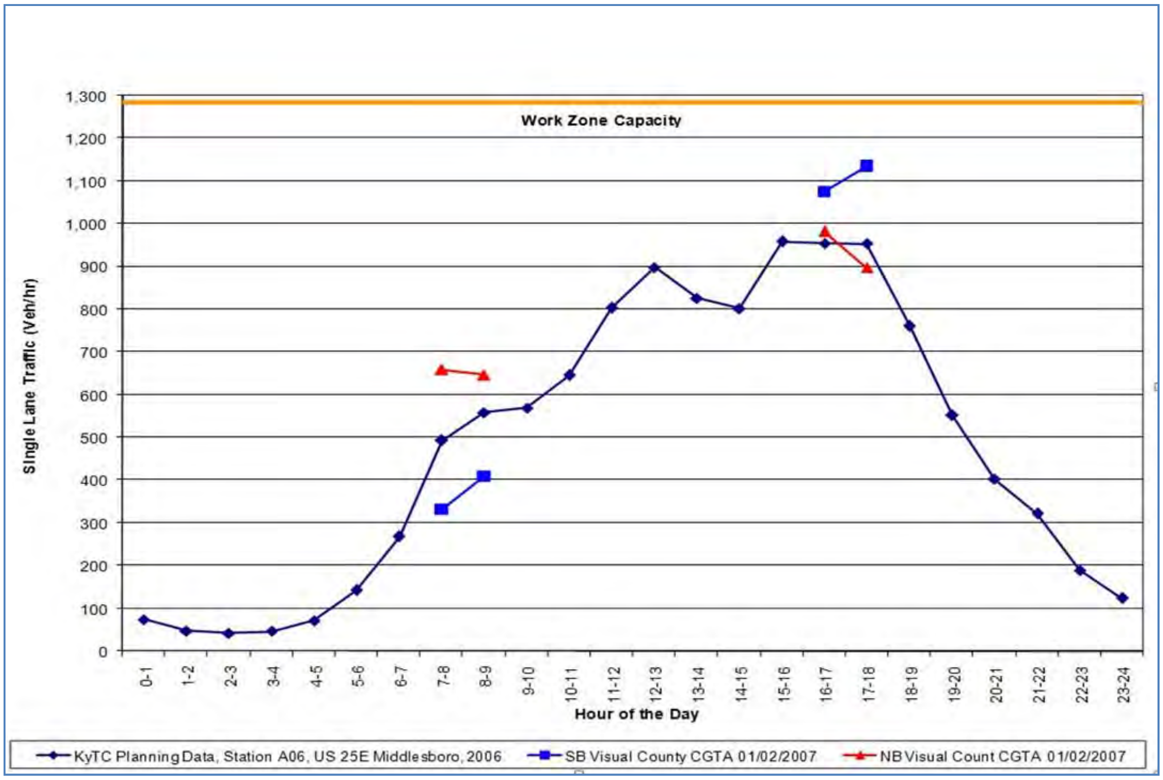


Figure 8: Single lane work zone capacity vs. hourly traffic distribution US 25 East Cumberland Gap Tunnel

FUTURE TRAFFIC IMPACTS

With the near completion of US 25E widening project from Harrogate, Tennessee to I-81 near Morristown Tennessee, it is conceivable that the traffic flow on US 25E will increase throughout the Cumberland Mountain Region in the near future. Once this construction is completed in late 2010, a driver will be able to reduce their driving time by an approximate 45 minutes when traveling from I-81 to I-75. With this reduction in travel time between the two major interstates, it is highly probable that the total volume of vehicles passing through the Cumberland Gap Tunnel will increase. Consideration should be given to construction scheduling in an effort to avoid excessive delays as traffic volumes increase. In addition, consideration for diverting traffic during the NASCAR racing season hosted in Bristol, Tennessee, also needs to be reviewed prior to scheduling of construction.

SHORT TERM REMEDIATION (MAINTENANCE)

Table 2:Short term remediation estimates for the settled areas may consist of the following

Maintenance Item	Construction Costs provided by KYTC Division of Highway Design
Grout all void areas on annual basis (consideration needs to be made for potential damming the ground water over repeated grouting sessions)	$1300/cubic yard. (includes coring, ground penetrating radar, and placing grout) Total annual costs $40-70K depending on void depth/growth
Remove concrete pavement (major settled area) northbound tunnel approximately 1,500 square feet and backfill with concrete (consideration needs to be made for potential damming the ground water for full depth concrete)	$150/square yard Total cost $25,000
Micro-piles, spaced 6 feet on center in void areas	$35/square feet (approximate areas 7300 square feet) Total Costs: $255,500

LONG TERM REMEDIATION

Based on the findings from the investigative repair, the hydro-geochemical water-chemistry data, and the continual growth of the void areas, it is of opinion that both the pavement and backfill material should be completely removed and replaced starting at approximate station 140+50 and proceeding to the Kentucky portal. It is also believed that a trench be excavated out in the invert of the tunnel to allow a majority of the ground water to channel through the tunnel (Figure 9). This trench would have to be of sufficient depth and width to lower the water table beneath the tunnels' concrete sidewall structure. All backfill material must be inert granite material, and the paved surface would be a CRCP pavement (Figure 9). An approximate construction cost of $10,000,000 has been estimated by the KYTC Division of Highway Design based from recent unit-bid-pricing to perform such tasks. However, this estimate may vary depending on the economic climate and contractor availability.

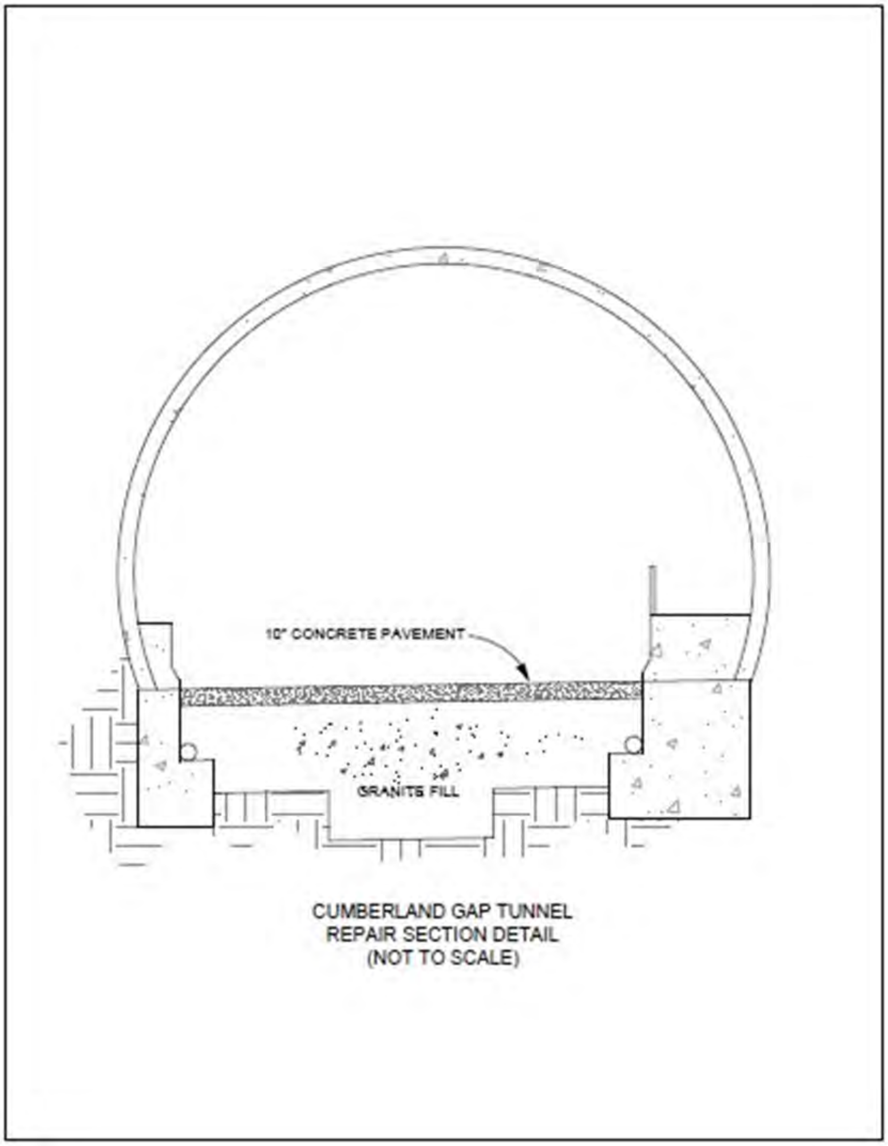


Figure 9: Conceptual long-term fix design--not intended for construction purposes.

CONCLUSIONS

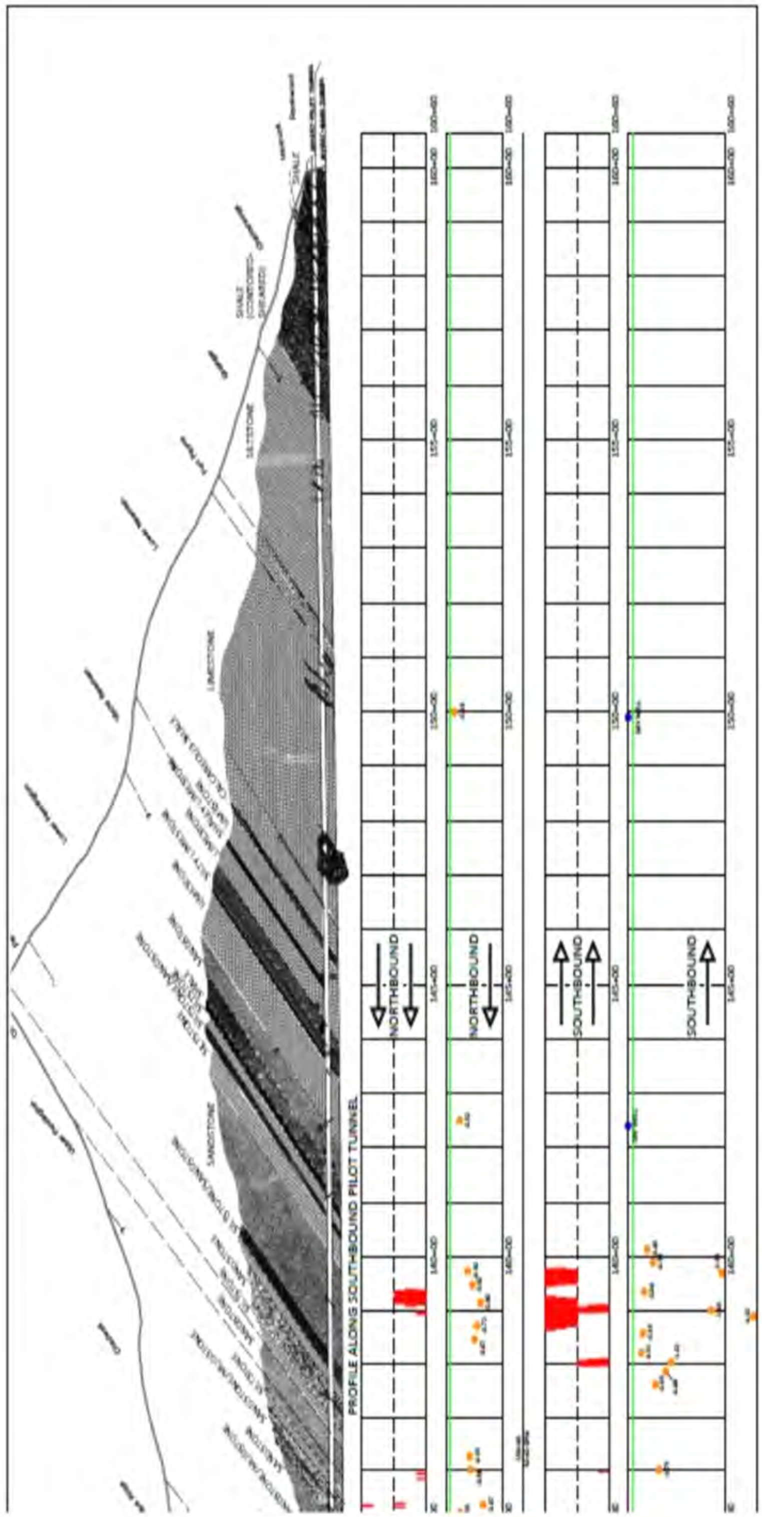
The concrete pavement structure at the Cumberland Gap Tunnel has been showing signs of pavement distress since 2001. The primary distress observed has been vertical displacement (settling) throughout various areas of both the north and southbound tunnels. To date approximately 7,400 square feet of continuously reinforced concrete pavement (CRCP) has voids beneath it. These voids range from 0.05 inch to 40 inches in depth.

In 2007 an investigative repair was conducted in the southbound tunnel to repair the most severely damaged section and to provide insight into the potential cause of the settlement issues.

From this investigation, it was determined through hydro-geochemical water-chemistry testing that the ground water in-flow throughout both tunnels is aggressive to calcite. Therefore, the 4-6 feet of calcium rich limestone backfill material placed beneath the concrete pavement is dissolving and leaving the tunnel through the ground water collection system on a daily basis. The calculated rate of removal has been estimated to be between 0.75 and 1.5 cubic yards per month or approximately 70 to 150 square surface feet of new void area is opening up beneath the concrete pavement on a monthly basis.

It has been proposed by a technical advisory group that the concrete pavement and limestone backfill material be removed from station 140+50 to the Kentucky Portal approximately 2,800 lineal feet in both tunnels. This removed material would be replaced by an inert granite backfill material and a new 10 inch continuously reinforced concrete pavement (CRCP). Preliminary construction estimates taken from previous unit-bid-pricing of the investigative repair, estimate that the repair will cost approximately $10,000,000. It is also proposed that annual maintenance be performed in the settled areas in efforts to avoid any potential pavement collapse until a long term fix is put into place. An approximate annual maintenance cost would be from $50,000-$100,000 per year.

APPENDIX A: Void and Water Well locations on strip map of tunnel.


PROFILE ALONG SOUTHBOUND PILOT TUNNEL
NORTHBOUND
NORTHBOUND
SOUTHBOUND
SOUTHBOUND
LIMESTONE
SANDSTONE
SILTSTONE
145+00
150+00
155+00
160+00

FIELD TRIP GUIDEBOOK

Compiled by
William M. Andrews Jr. and
Steven L. Martin
Kentucky Geological Survey

62nd HIGHWAY GEOLOGY SYMPOSIUM

www.HighwayGeologySymposium.org
Lexington, Kentucky
July 25th - 28th, 2011

Hosted By
The Kentucky Geological Survey
The Kentucky Transportation Cabinet, Geotechnical Branch

Coordinated By
University of Kentucky Transportation Center
Technology Transfer Program

Table of Contents

Field Trip Logistics

Welcome to the Field Trip for the 62nd Highway Geology Symposium, headquartered in Lexington, Kentucky! Field trip participants will be divided into three groups (**A**, **B**, and **C**) to accommodate the sites we will be visiting. All groups will be visiting the same sites but at different times and traveling on deluxe, 55 passenger coaches with video screens.

All groups will have lunch in the Grandstand area overlooking historic Keeneland Race Course. The food services will be provided by award-winning Turf Catering, Keeneland's sole caterer. Groups will arrive at Keeneland in 10-15 minutes staggered times for a box lunch and to complete their visit with a behind-the scenes tour with their Blue Grass Tour Guide.

The lunch is sponsored by **Geobrugg** and the field trip refreshments sponsored by **Golder Associates.**

Itinerary for Field Trip

Wednesday, July 27, 2011

All Groups

8:00 AM Coaches arrive at side entrance of Hilton Hotel to pick up guests.
8:30 AM Coaches departs Hilton Hotel

Group A

8:30 AM – Group A departs Hilton for Camp Nelson Geology Site
9:15 AM – Group A arrives at Camp Nelson
9:55 AM – Group A departs Camp Nelson for Civil War Heritage Park
11:20 AM- Group A departs Heritage Park for Keeneland
12:00 PM – Group A arrives at Keeneland
1:30 PM – Group A departs Keeneland for Darby Dan Thoroughbred Horse Farm
1:45 PM - Group A arrives at Darby Dan
2:45 PM - Group A departs Darby Dan for Woodford Reserve
3:00 PM - Group A arrives at Woodford Reserve for tour & tasting
4:30 PM - Group A departs Woodford Reserve for the Hilton
5:00 PM - Group A arrives at Hilton

Group B

8:30 AM- Group B departs the Hilton for Civil War Heritage Park
9:10 AM – Group B arrives at Heritage Park
10:30 AM – Group B departs Heritage Park for Camp Nelson Geology Site
11:10 AM – Group B departs Camp Nelson for Keeneland
11:50 AM – Group B arrives at Keeneland
1:30 PM – Group B departs Keeneland for Woodford Reserve
1:50 PM - Group B arrives at Woodford Reserve for tour & tasting
3:20 PM - Group B departs Woodford Reserve for Darby Dan Thoroughbred Horse Farm
3:50 PM - Group B arrives at Darby Dan
4:50 PM - Group B departs Darby Dan
5:20 PM - Group B arrives at Hilton

Group C

8:30 AM – Group C departs the Hilton for Woodford Reserve
9:05 AM – Group C arrives at Woodford Reserve for tour & tasting
10:35 AM – Group C departs Woodford Reserve for Darby Dan Horse Farm
11:00 AM – Group C tours Darby Dan
12:00 PM – Group C departs Darby Dan for Keeneland
12:15 PM – Group C arrives at Keeneland
1:30 PM – Group C departs Keenland for Camp Nelson Geology Site
2:10 PM - Group C arrives at Camp Nelson
2:55 PM - Group C departs Camp Nelson for Civil War Heritage Park
4:20 PM - Group C leaves Heritage Park for Hilton
5:00 PM - Group C arrives at Hilton

Field Trip Overview

The geology of Kentucky has profoundly influenced the history of settlement, migration, industry, and transportation in the state. A thick accumulation of thin-bedded, phosphatic limestone is the key geologic element that has helped central Kentucky's Bluegrass area develop distinctive industries of world renown. This field trip will examine a few selected examples of geology's impact on the history of this area. This one-day trip will, however, be only scratching the surface of the region's rich natural and cultural heritage.

Much of the material for this guidebook has been adapted from field trip guidebooks by Andrews and others (2002a) and Andrews and others (2002b).

Field Trip Area Physiography

Most of central Kentucky is within the Bluegrass Section of the Interior Low Plateaus Province (Fenneman, 1938). The Bluegrass Section is underlain by Ordovician limestone and shale, and is bordered on the east and south by the rugged Knobs Region. Depending on the proportion of limestone versus shale, the Bluegrass landscape varies from a gently rolling upland plain to an intensely dissected plateau with steep hills and narrow ridges. The origin of the name comes from a grass (*Poa pratensis*) imported by the earliest European explorers, which sometimes appears bluish green when lit by the sun.

The Bluegrass Section can be divided into three primary regions: Inner Bluegrass, Bluegrass Hills, and Outer Bluegrass. This field trip will only visit the Inner Bluegrass region. The Inner Bluegrass consists of gently rolling hills underlain by the Middle to Upper Ordovician Lexington Limestone (Fenneman, 1938; McFarlan, 1943). Karst development is locally extensive, especially in areas with minimal shale in the underlying bedrock. The gently rolling landscape of the Inner Bluegrass drew the attention of early explorers and settlers because of its legendary beauty and fertile soil. This region is home to several of Kentucky’s more famous industries: burley tobacco production, bourbon whiskey distilleries, and thoroughbred horse farms. The horse farms of Kentucky’s famous thoroughbred industry are almost exclusively found on the gentle hills around Lexington, and the state’s renowned bourbon whiskey industry had its beginnings using karst springs in the Inner Bluegrass.

The Kentucky River has carved a steep gorge known as the Palisades across the Inner Bluegrass. The Kentucky River Palisades stood as a barrier to north–south communication and transportation for early settlers, while the river itself provided a seasonal trade route for moving timber, coal, and other products from otherwise isolated eastern Kentucky communities to downstream commercial markets.

The Inner Bluegrass of Kentucky is the thoroughbred capital of the world. Photo by Dan Carey, KGS.

Bedrock Stratigraphy

Camp Nelson Limestone

The High Bridge Group contains the oldest exposed rocks in Kentucky, and consists of the Camp Nelson Limestone, the Oregon Formation, and the Tyrone Formation. Each of the three formations in the High Bridge Group is characterized by a dominant lithology. The Camp Nelson Limestone is the oldest unit in the High Bridge Group and is composed largely of yellowish-brown, fine-grained limestone with abundant dolomite-filled burrows. Minor shales and dolostones are present in the unit. Preferential weathering of the burrow fillings gives rise to the characteristic honeycomb appearance in weathered sections. A sparse, molluscan–ostracod–tabulate coral fauna is locally present in the limestone; gastropods and cephalopods are locally common. Open-marine faunas, including brachiopods, bryozoans, gastropods, crinoids, sponges, corrals, ostracods, and trilobites are more common in major shale partings. The Camp Nelson is interpreted to represent a shallow-ramp, somewhat restricted, platform-lagoonal environment (Cressman and Noger, 1976).

Old US 27 Bridge and the Palisades along the Kentucky River, with cliffs of Camp Nelson Limestone visible in the background. The Kentucky River provides 80 million gallons of water per day to 600,000 people. It also provides boating, swimming, and fishing recreation. The Kentucky River palisades support the highest concentration of rare plant species in the Bluegrass Region. The valley walls, almost vertical, rise 400 or more feet above the stream. Photo by Dan Carey, KGS.

Tyrone and Oregon Formations

The Oregon Formation is a rather arbitrarily defined unit based on the presence of mappable dolostones between the Camp Nelson and Tyrone. Major parts of the unit are composed largely of sugary-texured dolostone or calcareous dolostone, but limestone typical of the Camp Nelson or Tyrone are locally present. Ribbon-bedded limestone and dolostones, as well as limestone-dolostone breccias and psuedobreccias, also occur. The Oregon is interpreted to represent a mosaic of very shallow subtidal to intertidal environments in a transit zone between the dominantly subtidal Camp Nelson and the dominantly intertidal Tyrone.

The Tyrone Formation is the youngest unit in the Middle Ordivician High Bridge Group. The Tyrone Formation is typically laminated to thick-bedded, fine-grained limestone, which weathers light gray to white, while thin dolostones and dolomite-mottled calcilutites also occur locally. The Tyrone is well known for its sedimentary structures which include laminae, prism cracks, mudcrack polygons, spar-filled vertical burrows, mud-chip conglomerates and breccias, and fenestral fabric or birdseyes. Fossils are not common in the Tyrone. Tyrone lithologies, sedimentary structures, and biota all suggest a very shallow-ramp, high-intertidal environiment with minor intervals of subtidal and supratidal deposition (Cressman and Noger, 1976; Kuhnhenn and others, 1981).

The Tyrone also commonly contains several thin bentonites, most of which have the appearance of shale beds. These bentonites are interpreted to represent volcanic ash-fall layers, deposited into the quiet Ordovician environments of the High Bridge Group from eruptions occurring in the distant precursor Appalachian Mountains. Some of these bentonite beds are regionally widespread (Missouri to Canada), and have even been geochemically correlated with similar beds in northern Europe.

Weathered bentonite bed in an outcrop of the Tyrone Limestone. Photo by William Andrews, KGS.

Lexington Limestone

The Lexington Limestone consists of the Curdsville, Logana, Grier, Tanglewood, Brannon, and Sulphur Well Members. The Curdsville, and Grier Members are mapped as lower Lexington Limestone in the KGS Digital Mapping Program, while the Brannon and Sulphur Well Members are mapped as upper Lexington Limestone (Ciszak, 2000).

Lower Lexington Limestone

The lower Lexington Limestone consists of the Grier and Curdsville Limestone Members and consists of irregular bedded to nodular, light-gray to bluish-gray calcarenites, calcisiltites, and interbedded shales. They are mapped together because they complexly intertongue in the area. The Curdsville Member is typically a fossiliferous, light-gray, well-sorted, evenly-bedded, coarse-grained calcarenite. Bentonite shale layers are also common in and near this member as the Curdsville was deposited early in the Taconic Tectophase when volcanism would have been intense. The overall lithology is characteristic of a shallow-ramp, skeletal shoal or sandbelt facies (Cressman, 1973).

The Grier Limestone is a predominately dark gray, nodular to irregularly bedded, argillaceous, fine-grained calcarenite and calcisiltite interbedded with shale. Many beds are graded and scour-based or show hummocky crossbeds, swaley crossbeds, amalgamation, and gutter casts. The intense bioturbation, presence of shales and muddy limestones, and a diverse fauna suggest an intermediate-ramp, welloxygenated, shallow open-marine environment below normal wave base, but not above storm wave base. The hummocky crossbeds, crudely graded beds, and related features, which occur in small cycles nested in larger cycles, suggest deposition by storms.

Tanglewood Member of Lexington Limestone

The Tanglewood largely exhibits thin to medium, regular to wavy beds with bimodal crossbedding. Iron-stained, phosphorictic hardgrounds are especially abundant. Brachipod and bryozoan fossils are common. The Tanglewood has been interpreted to represent an upper-ramp, skeletal-shoal facies deposition above normal wave base and largely within the tidal zone (Cressman, 1973). The Tanglewood Member occurs as three, large, interconnected shoal complexes (the lower, middle, and upper tongues).

General bedding character of Grier Limestone of Lower Lexington Formation. Tanglewood lithology at top of cut. Photo by Jerry Weisenfluh, KGS.

Upper Lexington Limestone

The Upper Lexington Limestone Members include the Brannon Member and the Sulphur Well Member. The Brannon Member consists of thin-bedded, nodular to irregular bedded, fine-grained calcarenites, interbedded with thin shale beds and partings. Fossils are rare and there is evidence of soft-sediment deformation. The Brannon represents one of the deep-water, transgressive incursions onto part of the Lexington Platform and has been interpreted to reflect distal storm deposition in a deep-ramp setting well below normal wave base (Kulp, 1995; Ettensohn and Kulp, 1995).

The Sulphur Well Member is composed of irregular, thin-bedded to nodular, coarse-grained, bryozoan-rich calcarenites, interbedded with shale. Bryozoan and brachiopod fossils are common and the beds are locally crossbedded and rippled. The unit has been interpreted to represent an intermediate-ramp, bryozoan biostrome, equivalent to–but basinward of–high-energy, shallow-ramp, skeletal shoals represented by the middle tongue of the Tanglewood Member (Ettensohn and others, 1986).

Hydrology and Karst

The field trip area is within the Inner Bluegrass karst region, as defined by Thrailkill and others (1982). The bedrock of this region is highly susceptible to dissolution, with some members of the Lexington Limestone containing less than 5 percent insoluble material (Fisher, 1968). Isolated karst groundwater basins with well-organized dendritic conduit systems have developed to feed major base-level springs. These groundwater basins typically exhibit prominent surficial karst features. These include dolines, swallets, blind valleys, and karst windows. Beyond the margins of the groundwater basins are "interbasin" areas, where groundwater flow is relatively shallow and discharges at high-level springs. Many of the high-level springs are perched above bedrock strata with higher insoluble-residue contents. Surficial karst features may be more subdued in the shallow-flow inter basin areas. The inter basin areas commonly contribute to the surface catchment area of the groundwater basins.

Although lithology is–in the broadest sense–critical to the formation of karst landforms, only the smaller karst conduits appear to be stratigraphically controlled. Even the "shales" of the Lexington Limestone contain over 50 percent calcite, and are thus somewhat soluble. The larger karst conduits are capable of enough flow to sweep away any residual insoluble material, and thus effectively disregard the shalier layers. Structure also appears to only locally control karst development. DoIines and conduit passages commonly align with fractures, but extensive karst development parallel to mapped faults has only been documented in a relatively few locations, and in some instances flows perpendicular to mapped faults or bedrock anticlines (Thrailkill and others, 1982). Hydraulic paleogradient appears to be the primary influence on the development of karst landforms and trends.

Across most of the Bluegrass upland, the karst conduits are primarily horizontal, or nearly so, flowing to springs feeding base-level streams. Much of the karst development in even the largest groundwater basins in the Bluegrass is limited to 75 ft or less below the land surface, due to the subdued relief between the highest parts of the upland and adjacent base-level streams (Hopper, 1992). However, near the Kentucky River Palisades, where fluvial erosion has removed the impermeable bentonites in the upper parts of the Tyrone Limestone, numerous vertical karst pits and shafts up to 200 ft deep have developed.

The freshwater aquifer in the area is fairly shallow, and apparently confined primarily to the conduit system and associated epikarst flow. Saline and sulfur-rich brines are encountered at relatively shallow depths below the conduit system. The shallow saline waters were the source for the numerous salt springs and salt licks the earliest settlers and explorers found in the area.

Fractures in the Lexington Limestone in Fayette County are enlarged by slightly acidic rainwater to produce underground conduits. A sinkhole is formed when the ceiling of an underground cavity collapses. These fractures can also contribute to roadway failure. Photo by Bart Davidson, KGS.

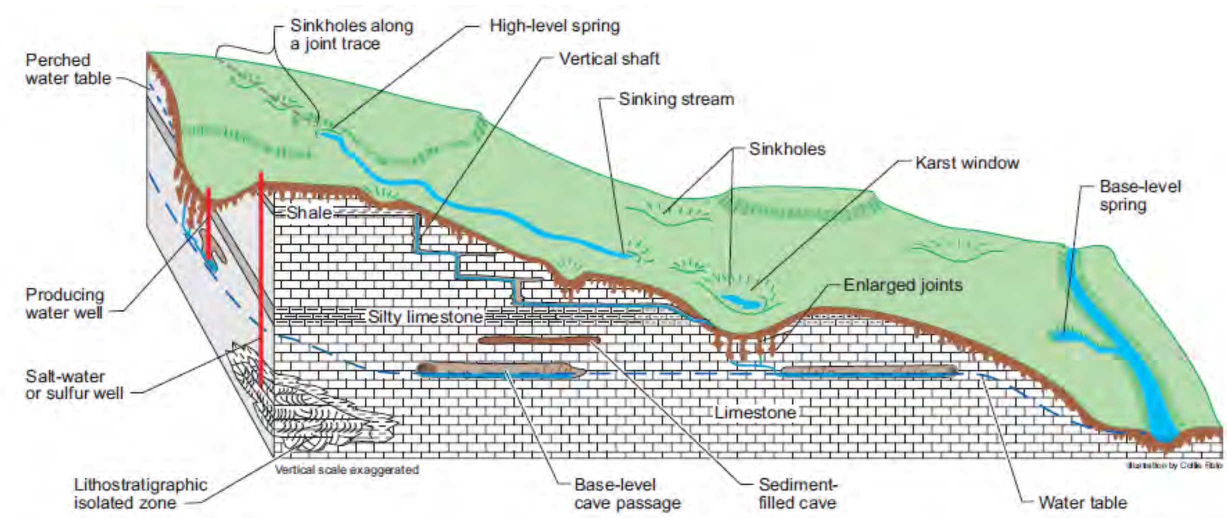


Diagram of typical karst features found in the Inner Bluegrass region of central Kentucky. From Currens (2001).

Bedrock Geology and Phosphatic Soils

It has been suggested that during Ordovician time Kentucky had fairly direct access to waters of the open ocean. This provided central Kentucky with abundant phosphate, presumably more than was needed to support Ordovician flora and fauna (Cressman, 1973). The average limestone contains only 0.04 percent phosphate, whereas the Tanglewood Limestone Member averages 2.4 percent. The phosphate content in the Tanglewood occurs as apatite fillings and phosphorite nodules, both of which vary in abundance from bed to bed. Phosphorite nodules in the Tanglewood occur as dark gray, poorly sorted, abraded bodies that have been reworked. Apatite is present as fillings of bryozoans, crinoid plates, and gastropod steinkerns (Cressman, 1973). The exact depositional or diagenetic origin of the phosphate in the Lexington Limestone is poorly understood.

Phosphatic soils from the Lexington Limestone provide the foundation for a strong agricultural economy. Bourbon County ranks near the top of Kentucky counties in livestock and total cash receipts. Photo by Dan Carey, KGS.

Weathering of Lexington Limestone bedrock produces a very phosphate-rich soil. The rich soil, via nutrients in grasses, is the basis of a healthy life for horses and other animals in the region. The unusually high concentration of phosphate allows this region to be the top producer of thoroughbreds in the country. There is such an abundance of phosphate in the soils of the central Bluegrass that it must be balanced with added calcium. Calcium is added by liming the horse pastures, in order to obtain a proper calcium-to-phosphate ratio. This process allows the soils to maintain a satisfactory pH level of 6.5 to 6.8 to allow maximum nutrient uptake by grasses and plants (Allman, oral communication, 2001). A proper calcite-to-phosphate ratio will produce both good quantity and a good quality of bone for the thoroughbred horses.

Other factors such as topography and a temperate climate provide thick soils for the major thoroughbred farms. The gently rolling karst landscape not only provides rich soil, but an adequate water supply due to an abundance of springs. Karst topography also allows the land to be very well drained; this is especially important for animals in the rainy or winter season. In dealing with a karst terrain, sinkholes and local areas of high relief are common. Horse farms are strategically placed where the land is less steep and away from sinkholes as much as possible. It is important for horses to grow in an environment that is not physically demanding or dangerous.

Overview of Kentucky History

Early Settlement, Industries, and Transportation (abridged from Harrison and Klotter, 1997)
Kentucky's human inhabitation began at least 10,500 years ago, as Paleo-Indian tribes migrated into the area following the retreat of Ice Age glaciers. Native peoples lived in Kentucky until as late as 1750, when disease, intertribal conflicts, and growing pressures from European settlers and explorers led to the abandonment of the last native villages within the area of modern Kentucky.

Daniel Boone Escorting Settlers through the Cumberland Gap, Oil on canvas (1851-52) by George Caleb Bingham (1811-1879); Washington University Gallery of Art, St. Louis, Missouri.

Spanish and French explorers and traders travelling along the Mississippi and Ohio Rivers during the late 17th century were probably the first Europeans to view the western parts of Virginia, which was later to become Kentucky. The first documented travels through the interior of the state were in 1673 when a young English colonist, Gabriel Arthur, accompanied a native war party across the state. Intensive exploration of the state began in the 1750's with Dr. Thomas Walker's and Christopher Gist's expeditions through the state, at the request of speculative land companies. Walker and Gist reported abundant coal deposits in eastern Kentucky, as well as salt springs, salt licks, and fertile savannas and cane breaks in the central Bluegrass. Long hunters were trekking through the area during this time, and the tales they took back east with them encouraged the first settlers to take a chance in this transmontane wilderness. The first permanent settlements were established in 1775, but the outbreak of the American Revolution (1776–

1783) slowed the early flow of settlers. During the Revolution, British agents supported and encouraged attacks against the Kentucky settlements by the Shawnee and other tribes. As the war came to a close, many who had suffered through the partisan strife in the east moved west to find a quieter life, and the flow of settlers into Kentucky dramatically increased.

The rapidly increasing population, the great distance across the mountains from the state capital in Richmond, and continued problems with hostile natives led to an effort to gain independence from Virginia. Some Kentuckians favored a complete split from the fledgling United States, seeking instead an alliance with France or Spain to secure a trade route down the Mississippi River. After 10 constitutional conventions held in Danville, however, Kentucky was admitted to the Union as the 15th state in 1792. Kentucky had grown from an uninhabited native hunting ground to a thriving Commonwealth in 17 short years.

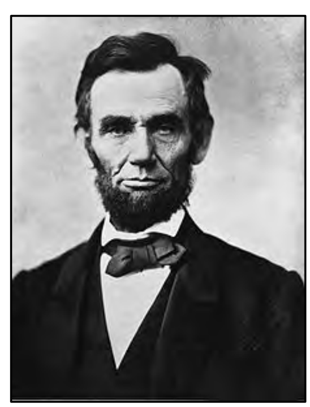
Through the early 19th century, Kentucky rose to prominence as an agricultural, commercial, political, and educational center for the expanding American "West," in large part because of the state's rich soils and convenient location along major rivers. The rivers were key transportation corridors for early Kentucky commerce. Coal and timber were shipped down the Kentucky River from Beattyville as early as 1790. In the early 19th century, Kentucky's livestock, crops, and bourbon whiskey were sold primarily down the Mississippi River through New Orleans, and rapidly gained a reputation for high quality. River transportation was unreliable, however, being dependent on adequate flow of the rivers across the numerous shallows and riffles. Early Kentuckians began to exploit the Commonwealth's varied mineral reserves; coal, iron, salt, saltpeter from cave deposits, and timber were key exports. The limiting factor in the industrial development of the Commonwealth was that many of the resources lay in remote areas not yet served by reliable transportation routes.

Kentucky also played a major role in the political life of the young nation during this time: Henry Clay and others served in key roles in Washington, D.C. The earliest schools west of the Appalachian Mountains were established here, and several, including Transylvania University, Berea College, and Centre College, continue to carry a very high reputation for their quality of education.

Civil War

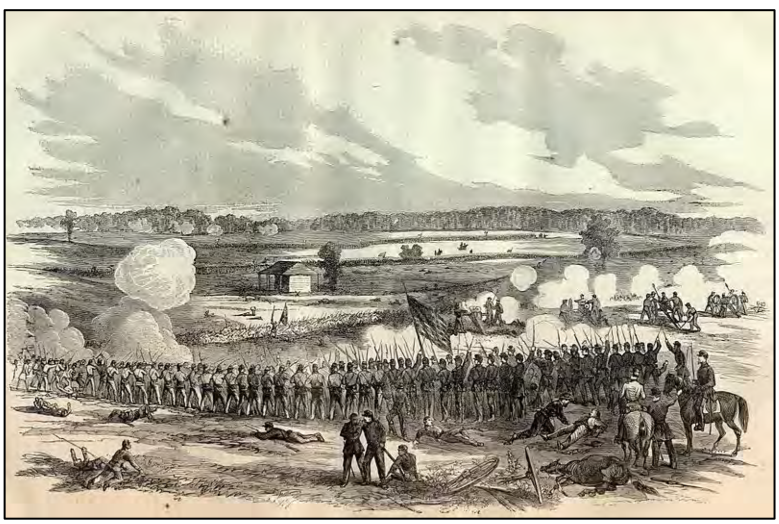
Sectional tensions rose by the mid-19th century, and Kentucky found herself a border state between the seceded southern slave states and the Unionist northern free states. The leaders of the two sides of the Civil War, Abraham Lincoln and Jefferson Davis, were both born in Kentucky, and Kentuckians brokered many compromises that delayed the onset of war. Kentucky occupied a precarious position, with strong industrial and manufacturing ties to the North, but equally strong agricultural and trade ties with the South. Kentucky chose a stance of neutrality through the early months of the conflict, but eventually sided predominantly with the Union. Over 100,000 Kentuckians served in the Union armies, and over 35,000 Confederate soldiers called Kentucky home.

"I, too, am a Kentuckian."
"I think to lose Kentucky is nearly the same as to lose the whole game."
–Abraham Lincoln



Richard Robinson, a staunch Garrard County Unionist, offered his farm for use as a Federal training camp and recruiting center, and Camp Dick Robinson was established in August 1861. Several Federal regiments from central and eastern Kentucky, as well as eastern Tennessee, were organized there. Confederates viewed the camp as a hostile violation of Kentucky's declared neutrality, which led to the Confederate occupation of far-western Kentucky in September 1861, and the subsequent establishment of a Confederate defensive line through southern Kentucky. Through late 1861 and early 1862, several small skirmishes and battles were fought across Kentucky as Federal forces sought to unhinge the tenuous Confederate line. The Federal army used Camp Dick Robinson as a supply depot for the Camp Wildcat (October 1861), Mill Springs (January 1862), and Cumberland Gap (May–June 1862) campaigns (Hughes, 1992), which ultimately pushed the war into central and western Tennessee.

The late summer of 1862 brought a large-scale Confederate invasion of Kentucky, which was part of a larger effort to move the war into northern territory and out of the South, to enable southern farmers time to gather their harvests for the war effort. The battles of Antietam (Maryland), Corinth (Mississippi), and Perryville (Kentucky) ended this offensive. The Battle of Perryville was the largest Civil War battle fought on Kentucky soil, and effectively ended Confederate hopes of occupying the state. On October 8, 1862, approximately 16,000 Confederate soldiers attacked portions of a 60,000-man Union army on the outer margin of the Inner Bluegrass near Perryville, driving parts of the Union army back nearly a mile. Troops on both sides suffered tremendously from thirst before and during the battle because of a severe drought in the region. Confederate troops initially camped at Perryville to secure a water supply from the springs in the area, and attempted to refuse the Union troops access to the precious pools. The fighting here was some of the fiercest of the Civil War for the numbers engaged: over 7,600 men were killed, wounded, or captured in a half-day battle. At day's end, however, the Confederates realized they were greatly outnumbered and retreated from the field, and ultimately from the state (Noe, 2001).



Harper's Weekly image of Battle of Perryville, the largest Civil War engagement in Kentucky, fought on October 8, 1862.

Because of the poor defensibility of the camp on the gentle Inner Bluegrass hills, Confederate forces easily captured the supplies at Camp Dick Robinson during the Perryville campaign. Upon regaining control of central Kentucky after the Battle of Perryville, Union forces moved the depot a few miles to a more defensible location on the north side of the Kentucky River. The new depot, named Camp Nelson, occupied a naturally fortified position atop the Palisades of the Kentucky River between deeply entrenched tributaries. Steep limestone cliffs prevented attack of the depot from three sides; only a narrow neck of land on the north side of the facility required fortification against overland approach. The warehouses for the depot were constructed in a blind karst valley nestled within the central part of the facility, which hid the warehouses from outside viewers and potential artillery attacks. Despite numerous forays into the state by General John Hunt Morgan and his Confederate raiders, the depot was never attacked, in large part because of the natural strength of the position (Sears, 1992).

From early 1863 until the close of the war, Camp Nelson served as a major supply depot for the Union war effort in central Kentucky. Aside from several Caucasian Kentucky and Tennessee units, over 20,000 former slaves and free blacks were recruited and trained as United States Colored Troops here. Camp Nelson was the third largest USCT training facility in the country. Many of the USCT recruits brought their families with them to the camp to protect them from Confederate or racist reprisals. At one point, many of these nonmilitary refugees were driven out of the camp by the military authorities. After many of them died of exposure, disease, and starvation, they were allowed to return. The refugee community that was subsequently established remains today as the community of Hall. In 1867, many Union dead buried throughout central Kentucky were moved to Camp Nelson when Camp Nelson National Cemetery was established (Sears, 1992).

The remainder of the war in Kentucky consisted mostly of cavalry raids and guerrilla attacks. Harsh treatment of slave-holding Kentuckians by occupying northern armies led to a gradual erosion of Unionist support from the state, until the end of the war, when sympathies for the defeated Confederates became more prevalent. Many famous feuds, especially in eastern Kentucky, had their roots in animosities developed during the Civil War.

Postwar Years

Following the Civil War, Kentucky did not regain her prominent position as a national economic and political leader. The unsettled years of conflict and discord had discouraged additional development of industry in the state, and the geographic isolation of the state became more acute as railroads took precedence over steamboats as the primary mode of transportation. Completion of railroads in the late 19th and early 20th century finally allowed extensive development of Kentucky's eastern coal field, and construction of the Federal Interstate Highway system in the mid-20th century facilitated industrial development across much of the state by providing inexpensive and reliable transportation corridors. Many of Kentucky's communities are now experiencing an unprecedented period of growth, expansion, and development as a result.

Bourbon Whiskey Production

Differing and conflicting versions of the history of Kentucky bourbon production exist. What is well accepted is that the production of whiskey in the Bluegrass Region began in the late 1700's. Many of the first settlers in this area were of Scots-Irish descent, and brought their whiskey-making knowledge with them to America. Because of the fertile soil, harvests of their crops of corn and other grains quickly outgrew what one family could reasonably consume. The sale of this surplus grain was problematic due to difficulties in transporting it to markets in more populated areas because of its bulky and perishable character. Fermenting and distilling these grains into whiskey solved both of these problems. A wagon or barge full of whiskey barrels could easily survive the trip to larger markets, as well as bring a much higher price than the original corn or other grains once it got there. By the mid-1800's, the sale of this whiskey became so popular that farming in central Kentucky became secondary to whiskey production.

This "corn whiskey" would later evolve by accident into what is known as bourbon today. One of the first pioneers in whiskey production, the Rev. Elijah Craig of Georgetown, Ky., used one batch of white oak barrels even though they had been partially burned in a cooperage fire. By the time these barrels had made it all the way down the Ohio and Mississippi Rivers to New Orleans, the raw corn whiskey had aged in the charred oak, giving it a mellower, caramel taste and amber color. The customers loved this "new" product, and its popularity caused the distillers to purposely toast or char the interiors of their barrels before adding the whiskey. Much of the whiskey that was shipped out of central Kentucky went through Bourbon County, which was stamped on the barrel. Thus, regional recognition of the whiskey from Bourbon County, or "Bourbon whiskey," was started. Today, Bourbon whiskey is legally defined as being made from at least 51 percent corn, being aged for at least 2 years in new white oak barrels charred on the inside, and bottled at a minimum of 80 proof (40 percent alcohol by volume).

Kentucky is renowned for premium bourbon whiskey. Image from www.bourbonblog.com.

The geology and environment of the Bluegrass support the production of bourbon in several ways. These include the groundwater, the fertile soil for grain production, the local native white oaks for the barrels, and the climatic temperature swings needed to properly age the bourbon. The most important of these is undeniably the water. All of the distilleries in this area obtain their water from limestone springs or limestone aquifers. The Ordovician limestones act as a natural filter, yielding clean, iron-free water for brewing (Thornton, personal commun., 2001). The lack of these impurities prevents undesired flavors from forming in the final product.

Thoroughbred Horse Industry

Early pioneer settlers arriving after Daniel Boone in the late 1700's came by horseback or on foot through Cumberland Gap, in the southeastern part of Kentucky. The horse remained the predominant means of transportation at that time, and their population increased as the central and southern Bluegrass Region was settled. Boonesborough was the first town to adopt a law to preserve and improve horse breeds (Kentucky Horse Park, written communication, 2001).

Many social and cultural changes took place in Kentucky after the Revolutionary War as new settlers arrived in the Bluegrass area. The first Kentucky racetrack was established in between Boonesborough and Harrodsburg. Racing became so popular that the downtown streets of Lexington were used as racetracks (Kentucky Horse Park, written communication, 2001). Many of these settlers came from Virginia, where the "English" way of life was still prevalent. This included love of land, love of the horse, love of horse racing, and fine breeding (Kentucky Horse Park, written communication, 2001). Virginians encouraged the breeding of thoroughbreds, as the first one, Bull Rock, was imported to Virginia in 1730 from England.

The first thoroughbred made its way to Kentucky in 1779. The growing popularity of racing, in addition to the arrival of English thoroughbreds, boosted Kentucky's young breeding industry to a national level.

The early settlers referred to horses with English thoroughbred lineage as "thoroughbred-bred blooded horses" or "high bred" horses (Kentucky Horse Park, written communication, 2001). By the 1800's, Kentucky was a leader in thoroughbred breeding and had a national demand for its horses. With Kentucky in the national breeding spotlight, the best of American thoroughbred bloodlines were brought into Kentucky's horse farms, solidifying the thoroughbred industry.

During the Civil War, thoroughbred racing was disrupted and almost eliminated in the South. Many Kentucky horse farms suffered when guerrillas stole horses to use in the war. A breed of horse known as the Kentucky Saddler was the favorite cavalry mount of the Confederates (Kentucky Horse Park, written communication, 2001). The Confederacy's initial success was due, in part, to the quality of the Saddler as a cavalry horse. Racing ceased in Louisville, but Lexington missed only one racing season during the war, when Union soldiers were camped at the racetrack. With southern racetracks destroyed, the North took control of racing after the Civil War. Kentucky still remained the capital of the horse industry, as wealthy folk bought Kentucky horse farms to raise and breed horses and found new markets in the Northeast, Midwest, and West.

The horse industry has played a major role in Kentucky's history, and will continue to do so in the future. Kentucky's horse farms flourish due to the fertility of soils that help build strong bones, and abundant water supply fed by Lexington Limestone springs. Kentuckians benefit from the horse industry in a recreational fashion by participating in many types of equine sporting events. One such event is the famous Kentucky Derby, where Kentucky thoroughbred farms produce a large percentage of the winners. Kentucky's affection for the horse allows various horse industries to bring economic growth, knowledge, and tradition to the Bluegrass State.

Stone Fences

One of the most honored symbols of the Kentucky Bluegrass landscape is the stone fence. These low walls once lined almost every turnpike and horse farm in central Kentucky (Murray-Wooley and Raitz, 1992). This field trip will pass numerous examples throughout the day. In the 18th and 19th centuries, large groups of immigrants from Ireland, Scotland, and northern England settled in this area of Kentucky. These immigrants encountered rocks and landscapes that were similar to what they had left in Great Britain, due to the Middle to Upper Ordovician strata exposed here in the Bluegrass. The thinly bedded limestones of the Lexington Limestone lent themselves especially well to "dry" fence (mortarless) construction used by these immigrant masons. Due to the alternating layers of clay or shale with layers of limestone less than 6 inches thick, blocks of appropriate size were easy to quarry with simple hand tools. Many of the original fences have been lost due to neglect, development, or agricultural efficiency efforts that encouraged use of wire or plank fencing.

Stone fence constructed of Lexington Limestone, next to a pond in Bourbon County. Rock fences built with local limestones by Irish immigrants line roads throughout the region. The terrain of the Lexington Limestone presents many such beautiful scenes in the Inner Bluegrass Region. Photo by Dan Carey, KGS.

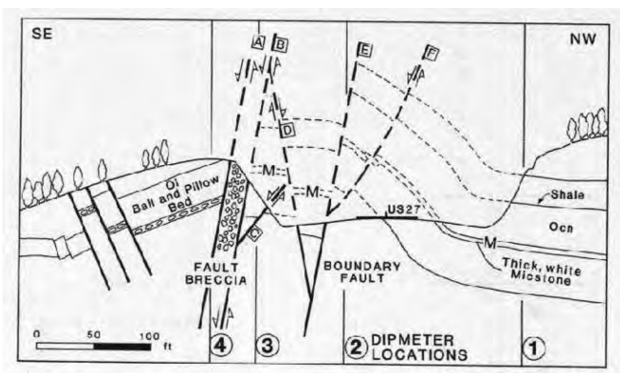
Field Trip Stops

STOP: Camp Nelson Geology

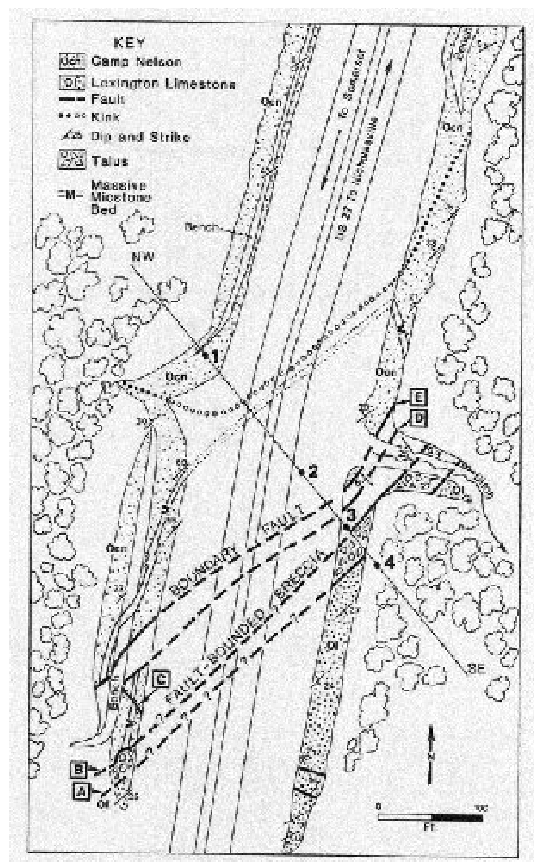
The entrenched valley of the Kentucky River near Camp Nelson provides access to numerous distinctive geologic features. Camp Nelson is one of the best areas in which to observe the nature of the Lexington Fault Zone and nearby Kentucky River Fault Zone and associated fracture systems, and it has been the site of several geologic field trips. The superposition of Hickman Creek on the Lexington Fault Zone provided the break in the river gorge and the natural ford that would become the north–south transportation corridor known as U.S. 27 today. Construction of the highway has provided access to the rocks of the High Bridge Group and Lexington Limestone and their interesting juxtaposition along the fault zone. Numerous karst features are exposed in the limestone roadcuts, and deposits of the ancient Old Kentucky River can be found mantling parts of the landscape. In 1838, a 240-foot-long, covered, wooden wagon bridge, called Hickman Bridge (or the Wernwag Bridge), was built across the Kentucky River near the site. The single-span bridge was built without metal and was considered an engineering triumph at the time, but has since been succeeded by two concrete and steel structures, both of which are still extant nearby. Because of the quality of exposures, accessibility, and the importance of the site in research and geologic education, the Kentucky Society of Professional Geologists named the Camp Nelson area as a Distinguished Geologic Site in 2002.

Large kink fold in the Camp Nelson Limestone along U.S. Highway 27 near the Kentucky River. Oblique movement along the irregular surface of the Lexington Fault System caused compression and deformation of these beds. The fault is exposed in another roadcut at this stop, to the right (south) of this view.

At this stop, the field trip investigates the Lexington Fault System and related features along the Kentucky River near Camp Nelson. The Lexington Fault System is a major structural feature in central Kentucky. Along U.S. Highway 27 near the Jessamine-Garrard County line the fault system consists of a complex mosaic of closely spaced grabens and horsts. The system trends northeast-southwest and is down-dropped to the southeast, and has as much as 700 feet of structural relief. As the fault system crosses the river northeast of the stop, it changes from a simple graben about 0.4 miles in width to a broader series of horsts and grabens, and en echelon faults with nearly double the width (Wolcott, 1969). At this stop, the highway crosses the northwestern boundary fault of this northeast–southwest-trending fault system. Based on the presence of ball-and-pillow structures, upper parts of the Grier Member of the Lexington Limestone have been downdropped nearly 300 feet to the southeast against the Camp Nelson Limestone on a normal fault that dips 75 to 90 degrees to the southeast (Wolcott, 1969; Gilreath and others, 1989). About 0.7 miles farther south, the highway crosses two southeastern bounding faults on a small graben complex in the larger fault system. Displacement on these faults is down to the northwest and is on the order of several tens of feet.

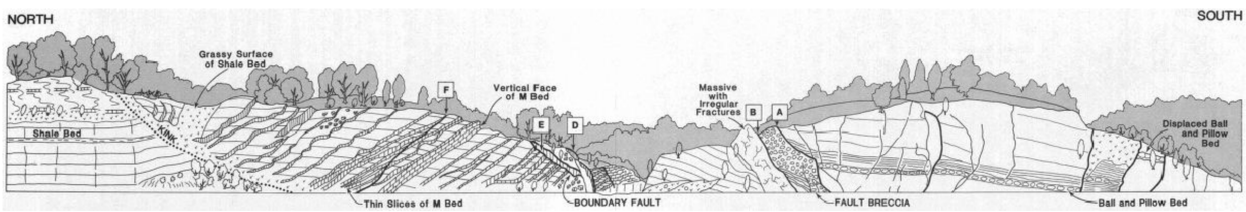


Detailed cross-section (above) and map (right) of the structures exposed at the Camp nelson Geology stop. Oblique movement along the faults has caused folding and minor faulting of the Camp Nelson Limestone. From Gilreath and others (1989).

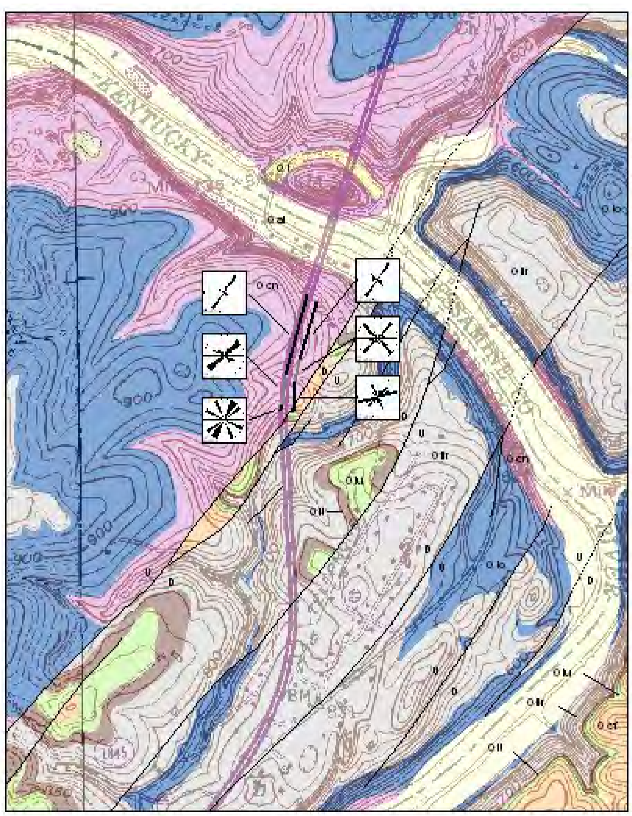


Although sense of movement on all the faults is now normal, both local and regional evidence indicates multiple episodes of movement, which includes strike-slip, normal, and reverse displacements (Black and Haney, 1975). The roots of the system were formed during the late Precambrian–Early Cambrian break-up of Rodinia, which was an oblique rift in Kentucky (Hickman, 2011). Evidence from jointing indicates that the strike-slip movement was largely left-lateral, and other evidence suggests that this movement was prevalent both before and after Ordovician time. Some normal movement also occurred during the

Ordovician, probably from compaction and dewatering of Cambrian clastics (Hickman, 2011). Reactivation of these faults occurred during the Alleghanian Orogeny resulting in the formation of the sharp kink fold at Camp Nelson. The large kink fold is probably the most outstanding feature along the exposure, and is a large drag fold in the footwall of northwestern boundary fault, which may reflect a shallow level deformation (Black and Haney, 1975; Gilreath and others, 1989). This fold may reflect local rotation from left-lateral rotation in the fault zone. In addition, the drag on this fold is the reverse of the present sense of displacement, suggesting a later episode of reverse movement, perhaps related to compressional forces generated during the Alleghanian Orogeny (Greb and Dever, 1997). This sharp fold is easily mistaken for a fault, but careful tracing of the shale beds along both side of the road proves otherwise. There are several other related features along the exposure which include fault breccia, large and small normal faults, several small reverse and thrust faults, fault related jointing, mineralization along joints, and slickenslides.



Sketch of features in roadcut along the east side of U.S. Highway 27 south of the Kentucky River near Camp Nelson. Major features to look for at this exposure include: shale bed in the Camp Nelson Limestone; kink fold; massive white micritic bed (M bed); boundary fault; fault breccia; and displaced ball-and-pillow bed in Lexington Limestone. Letters in boxes refer to faults. From Gilreath and others (1989).



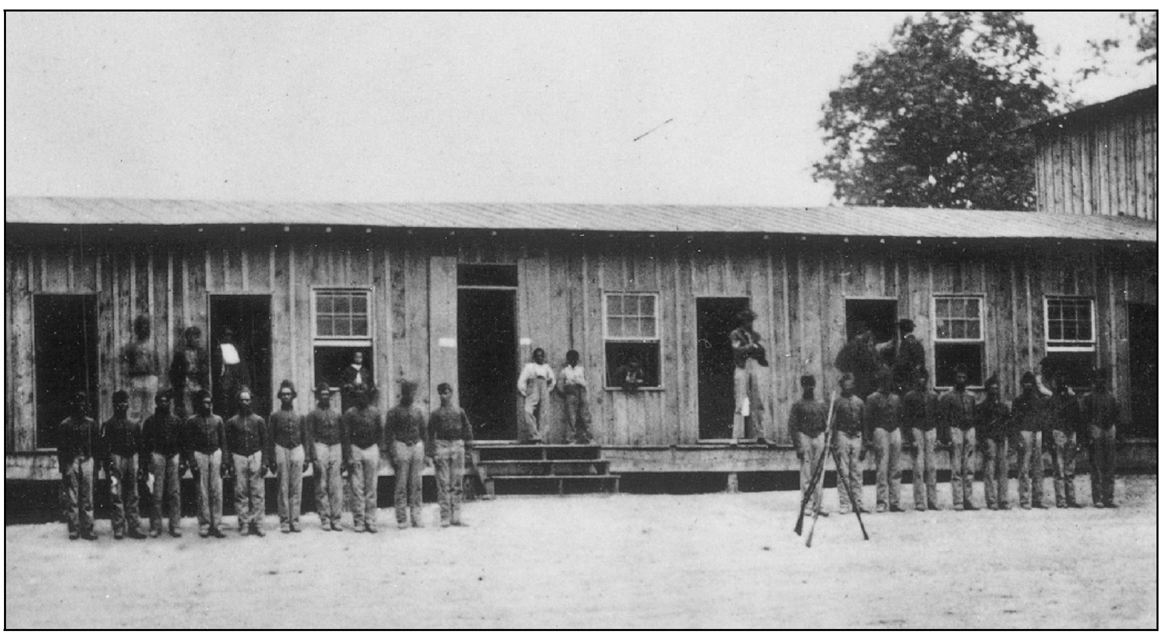
Rose diagrams showing the orientation of joints associated with the Lexington Fault Zone near Camp Nelson, Kentucky. Original map scale is 1:12,000. Map abbreviations include U = up thrown fault block, D = downthrown fault block, Ocf = Clays Ferry Formation, Olu = upper members of the Lexington Limestone, Olt = Tanglewood Member of the Lexington Limestone, Ollr = lower members of the Lexington Limestone, Oto = Tyrone and Oregon Limestones, Ocn = Camp Nelson Limestone, Qt = Terrace deposits, and Qal = alluvium.

STOP: Camp Nelson Civil War Heritage Park

Camp Nelson has been strategic from the 1700's–when settlers first entered the area–until today because the area provides an important north–south transportation route across the barrier created by the Kentucky River gorge. In contrast to the relatively flat-lying to gently rolling terrain that characterizes the surrounding Inner Bluegrass Region, the valleys of Kentucky River and its tributaries are steep-walled or entrenched, a situation that creates a natural obstacle to transportation and communication. In the Camp Nelson area, however, Hickman Creek, which follows the Lexington Fault System, empties into the Kentucky River, generating a natural ford across the river. This ford became the site for a small community on the Nicholasville-Danville Pike following the Revolutionary War. During the 19th century, the ford was a crossroads for those carrying their goods overland along the north–south roads and those carrying their goods east–west along the river. In 1838, a 240-foot-long, covered, wooden wagon bridge, called Hickman Bridge (or the Wernwag Bridge), was built at the site. The bridge was built without metal and was considered an engineering triumph at the time (Federal Writers' Project, 1939), but has since been succeeded by two concrete and steel structures, both of which are still extant near the site.

During the American Civil War, the Union Army used the natural defenses at Camp Nelson to shelter a large supply depot and training center on a 4,000-acre peninsula on the north shoulder of the Kentucky River Palisades. Thousands of Union Army recruits trained here, including over 20,000 U.S. Colored Troops. A large blind karst valley provided shelter for the warehouses of the supply depot; an elaborate water supply system drew water from the Kentucky River for the camp. A short line of forts and entrenchments on the north end of the peninsula provided a solid line of defense for the landward approaches to the camp; Camp Nelson was never attacked, despite numerous Confederate raids through central Kentucky.

The Camp Nelson Civil War Heritage Park is a 525-acre (2.12 km^2) historical museum and park located in southern Jessamine County, Kentucky, 20 miles (32 km) south of Lexington, Kentucky.



Camp Nelson was a Union Army supply depot and training camp during the American Civil War. Visitors can tour original earthworks, visit historic structures, and see a reconstructed barracks like this one.

STOP: Darby Dan Thoroughbred Horse Farm

The Bluegrass Region is underlain by limestones with a high phosphate content, which weather to produce soils high in calcium and phosphate, two key components that help animals grow strong bones. This is critical for raising successful thoroughbred race horses, which carry a body weight of 1000 to 1300 lbs on ankles not much larger than those of an average human. When combined with a temperate climate, the limestone soils have helped to make Kentucky the Horse Capital of the World!

The Darby Dan Farm represents the history of Thoroughbred breeding and racing in America like few other farms. For a century its white-fenced rolling pastures have blended great horses and colorful characters into lasting legends of the Bluegrass.

At the beginning of the 20th century, the legendary Colonel E.R. Bradley developed a top-rate equine facility and assembled breeding stock from the finest Thoroughbreds in the world at a farm he named Idle Hour. Bradley's efforts produced four Kentucky Derby winners: Behave Yourself, Bubbling Over, Burgoo King and Brokers Tip.

After Colonel Bradley's death, John W. Galbreath purchased the core property of Idle Hour. Re-naming the farm Darby Dan, Galbreath's results were profound. In 1961, Galbreath imported the undefeated European superstar Ribot. That effort paid large returns when Ribot's sons proved very successful in the following decades. In 1963 and 1697 Darby Dan's Chateaugay and Proud Clarion won the Kentucky Derby. In 1972 Darby Dan's homebred Roberto bested favored Rheingold in the English Derby at the famous Epsom Course. In 1985 Darby Dan's Proud Truth won the Breeders' Classic, making Darby Dan the only farm in history to accomplish this most coveted trinity of races. Galbreath's grandson, John Phillips, remains committed to this heritage. The 1990's saw an impressive list of Grade I winners.

Since the days of Colonel Bradley, horse racing and breeding has changed considerably. Darby Dan transitioned from a private facility to a commercial one, offering a full array of services. At Darby Dan there are over 150 mares owned by thoroughbred enthusiasts from all over the world. The farm has also seen the rebirth of a thriving stallion operation, with stallions representing some of the most regal bloodlines including sons of A.P. Indy, Unbridled, Sadler's Wells, and Mr. Prospector.

Rooted in a distinguished private history and now flourishing in the commercial present, Darby Dan is a special farm committed to its motto: Devoted to the horse, Dedicated to our clients.

Information and image provided by Darby Dan Farm.

STOP: Woodford Reserve Distillery

Early farmers in the Bluegrass Region discovered that the fertile limestone soils produced abundant harvests of corn, whereas the regions numerous karst springs provided steady and clean water supplies that supported the production of bourbon whiskey. Numerous distillers of distinctive whiskey developed across the region, and many still survive today.

The Woodford Reserve Distillery, a National Historic Landmark, is known throughout the world as the "Homeplace of Bourbon." Guests who visit the distillery find it much like it was in the 1800s, right down to the copper pot stills. Woodford Reserve is the only distillery in Kentucky using this time-honored and hand crafted method of production. It is the pot stills that give Woodford Reserve products the unique quality and taste that today's consumers of premium spirits demands. It is more than just the finest Kentucky bourbon whiskey produced today; it's a rare taste of history. The guided tour through the distillery explains everything from the grains they use to the maturation and bottling process. At the end of the tour, guests 21 and over can sample the award-winning bourbon. You also will have time to visit the popular gift shop or browse one of our many informational displays in our Visitor Center. Woodford Reserve is handcrafted and authentic.

Information and images provided by Woodford Reserve Distillery.

LUNCH: Keeneland Race Course

Field Trip participants will have lunch in the Grandstand overlooking the historic Keeneland Race Track, and afterwards will experience a behind-the-scenes tour of the landmark thoroughbred race track.

For 75 years, Keeneland has been to the Thoroughbred industry what Augusta is to golf or Hollywood is to the movies. It is where traditions are treasured, innovations are made and where expertise resides. It is the pulse point for a significant global business. Located in the heart of Central Kentucky, Keeneland is a rare combination of high-stakes commerce, genteel sporting tradition and remarkable innovations. As the world's largest Thoroughbred auction company, Keeneland's horse sales totaled almost $400 million in 2010. Our racing program offers some of the richest purses —- and therefore the most competitive racing —- of any track on the continent. Many of the sport's best stories have Keeneland connections.

Further enhancing Keeneland's reputation as a leader in the Thoroughbred industry is our strict adherence to the traditions of the sport. As one of our founders, Hal Price Headley, said in 1937, "We want a place where those who love horses can come and picnic with us and thrill to the sport of the Bluegrass."

The words ring as true today as they did then. With the installation of a Polytrack racing surface on the main track in 2006, Keeneland continues its mission of being one of the safest, most modern racetracks in the world.

And down the stretch they come! Thoroughbreds thunder out of the turn toward the finish line during the Keeneland Spring Meet.

Information and image provided by Keeneland.

References

Andrews, W.M., Crawford, M.M., Hickman, J.B., 2002a, Influence of geology on the military and cultural history of the Bluegrass Region, Kentucky, in Ettensohn, F.R., and Smath, M.L., eds., Guidebook for Geology Field Trips in Kentucky and Adjacent Areas: Geological Society of America/University of Kentucky, p. 108-128.

Andrews, W.M., Ettensohn, F.R., Gooding, P.J., and Smath, M.L., 2002b, Impact of Geology on Human History at Camp Nelson and Perryville, Central Kentucky: Kentucky Society for Professional Geologists Annual Field Trip Guidebook, September 19-21, 112 p.

Black, D.F.B., and Haney, D.C., 1975, Selected structural features and associated dolostone occurrences in the vicinity of the Kentucky River Fault System (roadlog for Geological Society of Kentucky 1975 field conference): Kentucky Geological Survey, ser. 10, 27 p.

Ciszak, E.A. , 2000, Spatial database of the Little Hickman quadrangle, central Kentucky: Kentucky Geological Survey, series XII, Digitally Vectorized Geologic Quadrangle, DVGQ-0792; adapted from Wolcott, D.E., 1969, Geologic map of the Little Hickman quadrangle, central Kentucky: U.S. Geological Survey Geologic Quadrangle Map GQ-792, scale 1:24,000.

Cressman, E.R., 1973, Lithostratigraphy and depositional environments of the Lexington Limestone (Ordovician) of central Kentucky: U.S. Geological Survey Professional Paper 768, 61 p.

Cressman, E.R., and Noger, M.C., 1976, Tidal-flat carbonate environments in the High Bridge Group (Middle Ordovician) of central Kentucky: Kentucky Geological Survey, ser. 10, Report of Investigations 18, 15 p.

Currens, J.C. , 2001, Generalized Block Diagram of the Inner Bluegrass Karst: Kentucky Geological Survey, series XII, Map and Chart no. 15, 1 sheet.

Ettensohn, F.R., Amig, B.C., Pashin, J.C., Greb, S.F., Harris, M.Q., Black, J.C., Cantrell, D.J., Smith, C.A., McMahan, T.M., Axon, A.G., and McHargue, C.J., 1986, Paleoecology and paleoenvironments of the bryozoan-rich Sulphur Well Member, Lexington Limestone (Middle Ordovician), central Kentucky: Southeastern Geology, v. 26, p. 199–219.

Ettensohn, F.R., and Kulp. M.A., 1995, Structural-tectonic control on Middle–Late Ordovician deposition of the Lexington Limestone, central Kentucky, in Cooper, J.D., Droser, M.L., and Finney, S.C., eds., Ordovician odyssey: Short papers for the Seventh International Symposium on the Ordovician System: Fullerton, Calif., Pacific Section, Society for Sedimentary Geology (SEPM), p. 261–264.

Federal Writers' Project, 1939, Kentucky: A guide to the Bluegrass State: New York, Harcourt, Brace & Co., 489 p.

Fenneman, N.M., 1938, Physiography of the eastern United States: New York, McGraw-Hill, p. 427–431.

Fisher, I.S., 1968, Interrelation of mineralogy and texture within an Ordovician Lexington Limestone section in central Kentucky: Journal of Sedimentary Petrology, v. 38, p. 775–784.

Gilreath, J.A., Potter, P.E., and Losonsky, G., 1989, Guide to interpretation of structural features associated with the Kentucky River Fault System along U.S. Highway 27 near Camp Nelson, Kentucky: Kentucky Geological Survey, ser. 11, Map and Chart 1, 1 sheet.

Greb, S.F., and Dever, G.R., Jr., 1997, Lexington Fault System, in Greb, S.F., Dever, G.R., Jr., and Anderson, W.H., Economic geology of the Inner Blue Grass region (guidebook and roadlog for Kentucky Society of Professional Geologists 1997 field trip): Kentucky Society of Professional Geologists, p. 28.

Harrison, L.H., and Klotter, J.C., 1997, A new history of Kentucky: Lexington, University Press of Kentucky, 501 p.

Hickman, J.B., 2011, Structural Evolution of an Intracratonic Rift System; Mississippi Valley Graben, Rough Creek Graben, and Rome Trough of Kentucky, USA [PhD Dissertation]: University of Kentucky, 186 p., 31 plates.

Hopper, W.D., 1992, Karst geology of the Bluegrass (Speleofest guidebook): Blue Grass Grotto of the National Speleological Society, p. 12-17.

Hughes, S.L., 1992, Camp Dick Robinson, in Kleber, J.E., ed., The Kentucky Encyclopedia: Lexington, University Press of Kentucky, p. 157–158.

Kulp, M.A., 1995, Paleoenvironmental interpretation of the Brannon Member, Middle–Upper Ordovician Lexington Limestone, central Bluegrass Region of Kentucky: Lexington, University of Kentucky, master's thesis, 222 p.

Kuhnhenn, G.L., Grabowski, G.J., Jr., and Dever, G.R., Jr., 1981, Paleoenvironmental interpretation of the Middle Ordovician High Bridge Group in central Kentucky, in Roberts, T.G., ed., GSA Cincinnati '81 field trip guidebooks, v. 1: Stratigraphy, sedimentology: Falls Church, Va., American Geological Institute, p. 1–30.

McFarlan, A.C., 1943, Geology of Kentucky: Lexington, University of Kentucky, 531 p.

Murray-Wooley, C., and Raitz, K., 1992, Rock fences of the Bluegrass: Lexington, University Press of Kentucky, 220 p.

Noe, K.W., 2001, Perryville—This grand havoc of battle: Lexington, University Press of Kentucky, 512 p.

Sears, R., 1992, Camp Nelson, in Kleber, J.E., ed., The Kentucky Encyclopedia: Lexington, University Press of Kentucky, p. 158.

Thrailkill, J., Spangler, L.E., Hopper, W.M., McCann, M.R., Troester, j.W., and Gouzie, D.R., 1982, Groundwater in the Inner Bluegrass karst region, Kentucky: University of Kentucky, Kentucky Water Resources Research Institute, Research Report 136, 108 p.

Wolcott, D.E., 1969, Geologic map of the Little Hickman quadrangle, central Kentucky: U.S. Geological Survey Geologic Quadrangle Map GQ-792, scale 1:24,000.

Acknowledgements

62nd Highway Geology Symposium Local Committee

Henry Mathis – Local Chairman	H.C. Nutting a /Terracon Co.
Tony Beckham	Kentucky Transportation Cabinet
Mike Blevins	Kentucky Transportation Cabinet
Doug Smith	Portland Cement Association
Mike Gragg	Florence & Hutcheson
Craig Lee	Soil and Materials Engineers, Inc.
William Andrews	Kentucky Geological Survey
Dr. John Kiefer	Kentucky Geological Survey-Retired
Mike Lynch	Kentucky Geological Survey
Larry Rhodes	Rhodes Consulting Group
Marwan Ryan	Lexington – Fayette Urban County Government
Richard Wilson	Horn and Associates
Mark Litkenhus	Stantec
Marsha Wilson	Lexington Convention and Visitors Bureau
Lauren Cosentino	Kentucky Transportation Center
Martha Horseman	Kentucky Transportation Center

Symposium Sponsors

Gannett Fleming, Inc.
Golder Associates Inc.
Michael Baker Jr., Inc.
Zonge International Inc.
Hi-Tech Rockfall
S&ME
Stantec
Florence & Hutcheson
Geobrugg North America

Symposium Exhibitors

AIS Construction
AMS
Applied Geomechanics
Baroid Industrial Drilling Products
Carmeuse Lime and Stone
Central Mine Equipment
Gannett Fleming, Inc.
Geobrugg North America
Geokon, Inc.
Golder Associates Inc.
Hayward Baker Inc.
HI-TECH Rockfall
Horn & Associates, Inc.
Janod, Inc.
Maccaferri Inc
Michael Baker Jr., Inc.
Monotube Pile Corporation
Mountain Management / Igor Paramassi
Pacific Blasting & Demolition LTD.
Soil Nail Launcher, Inc.
TenCate Mirafi
Terracon Consultants
Williams Form Engineering
Zonge International Inc.

Special Thanks to:

Bluegrass Tours
Camp Nelson Heritage Park
Darby Dan Farm
Keeneland Race Course
Kentucky Geological Survey
Woodford Reserve Distillery